Experimental And Numerical Investigation Of The Behavior Of Castellated Steel Beam-Columns

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Abstract: Experimental investigation has been performed on twelve short and long castellated beam-column elements. The castellation process effectively increases the section resistance because of increasing the depth of the section without any additional weight. By increasing the depth of the beam, major axis bending resistance and stiffness are improved as the major axis moment of inertia (Ix) and the section modulus (Sx) are increased. It has been noticed that, researches that considered castellated elements subjected to axial and bending moment are limited compared to those considered elements subjected to bending moment only. In this research castellated beam-columns are studied under different load eccentricity to investigate the effect of castellation on beam-column strength. Behavior and strength of castellated beam-column are also investigated using the finite element analysis utilizing ANSYS software. The finite element results are found to be in a good agreement with their experimental counterparts.

Index Terms: Castellated steel beam-columns, Castellated steel beams, castellated steel columns, FEM modeling, flexural buckling, lateral torsional buckling, Web opening.

1 INTRODUCTION

castellated beams, columns and beam-columns have been used as structural members in steel structures. Castellated beams and columns are fabricated from a hot-rolled steel I-section, by cutting the section web on a half hexagonal line down its center. The two halves are moved across by a single spacing and then rejoined together by welding. This process increases the depth of the steel element and accordingly, the major axis bending resistance and stiffness, without any additional steel weight. This results in the possibility of utilizing castellated member in long spans with moderate loading conditions in roofs and floors. The fabrication process results in openings on the member web, which can be utilized to accommodate services. Accordingly, the overall height of the building can be significantly reduced, regardless of increasing the element depths, compared to using elements with solid webs, where services are only possible beneath the beams. Regardless of the increased cost due to the fabrication process, the advantages of using castellated steel elements outweigh the disadvantages. Mohebkhah and Showkati [1] Studied the effect of lateral bracing on castellated steel element capacities and concluded that the effect of bracing became reduced in the elastic buckling range, whereas this effect is enhanced with the increase of the laterally unbraced length. They developed a general equation to predict the castellated flexural members' stiffness and capacity. The developed formula accounted for the member slenderness. Nethercot and Kerdal [2] investigated the behavior of castellated steel beams without the provision of lateral supports. He developed empirical equations that could be used to estimate the co-efficiency of bending (Cb) for castellated-beams under various conditions. Ellobody [3] [4] studied the effect of combined lateral-torsional and distortional buckling modes on the behavior of castellated and cellular elements. In his study, he considered normal and high strength steel castellated elements and concluded that failure load of castellated steel elements was likely reduced due to the presence of web distortional buckling in the case of slender steel beams. He Also, found out that, utilizing high strength steel has contributed to an increase in the failure load of less slender castellated beams. He furthermore concluded that the Australian Standards equations predicting the behavior of steel beams under lateral-torsional buckling were conservative, however, when castellated elements were failing in web distortional buckling the equation was otherwise. Finally, when high strength beams by lateral-torsional buckling, the AS expressions were quite conservative. Abu-Sena et al. [5] Conducted a parametric study to predict the buckling behavior and the element capacity of beam-column elements. They considered the initial imperfection in the F.E.A and developed an analytical model to predict the interactive strength compared to the finite element non-linear analysis of EC3 design approach. Yuan et al. [6] Studied the critical buckling load of simply supported castellated columns. They considered the buckling about the major axis and concluded that the presence of the web openings resulted in shear deformations along the castellated steel column and contributed to reduction in the buckling capacity of the castellated steel column. They moreover concluded that web shear deformations couldn't be neglected, even if the second moment at the opening location's area was taken into consideration in predicting the buckling strength of the castellated steel column. Kumbhar and Jamdar [7] investigated the behavior of castellated elements with sinusoidal openings. They used these types of openings to study the possibility of optimization. They considered different sizes of web openings and found that when sinusoidal web opening sizes were equal to 0.55 times the total depth of the beam element, along with the strength of the element might reach the maximum. They also concluded that utilizing these types of openings resulted in further shear transfer area. Cheng [8] studied the critical buckling load (CBL) of cellular columns. They focused on the major buckling axis and considered the web shear deformations that have a clear influence on the buckling capacity of the cellular elements. They concluded that the effect of shear deformations on the cellular element increased along with the increase in the cross-section area of the tee section above the openings and the diameter of the web opening, however it decreased with the web thickness and the length of the cellular element. Shaikh and Autade [9] Studied the effect of web shear deformation on the critical buckling load of castellated beams.

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They found out that, the depth of the web openings had a significant effect on the shear deformations that might occur in the castellated steel element. They also presented an analytical solution to predict the critical buckling load of the castellated steel element and found out that the utilization of rounded holes avoided Vierendeel effect (to avoid stress concentration), and local failure of the beam. The provision of a plate below the concentrated load and reinforcement at the beam's weak sections could be used to enhance the beam's capacity. Sonck and Belis [10] studied the effect of residual stresses on the weak-axis flexural capacity of both castellated and cellular columns. They concluded that the weak axis flexural capacity was affected by the residual stress simulation. Wang et al. [11] used the finite element method to investigate the post-buckling behavior of the web of castellated beams when subjected to vertical shear. They concluded that the web thickness and the inclination angle of the web opening had a significant effect on the shear buckling factor. They moreover presented a simplified method to predict the shear buckling factor of castellated steel beam. Yuan et al. [12] investigated the transverse deflection of perforated beams. They considered two different boundary conditions and concluded that web shear effect could significantly increase the transverse deflection of castellated and cellular beams particularly when the beam has a short length and deep section. Elaiwi et al. [13] presented theoretical and numerical solutions for calculating the deflection of hexagonal castellated beams with simply supported boundary condition, considering the influence of shear deformations that might occur in the web. Elaiwi et al. [14] investigated the effect of the web opening on the lateral-torsional buckling resistance of castellated beams. They developed an analytical solution that could be used for the design and practicality. They concluded that plastic failure was the failure of short castellated beams' mode whereas lateral-torsional buckling was the mode of long beams' failure, accordingly, the longer the beam, the less important was the nonlinearity. Hadeed and Alshimmeri [15] studied the effect of castellation with and without strengthening on the structural behavior of castellated-beams and compared the results with the origin solid steel beam. They found out that, the load carrying capacity of the castellated steel beams was increased compared with the original solid beam, while mid-span deflection values at service load were decreased comparing with the origin solid steel beam. Liu et al. [16] studied octagonal web openings elements. They used high strength bolt to connect the two halves of the elements in site instead of utilizing the factory welding. They draw a comparison between bolted and welded web opening elements and found out that, the structural behavior of bolted elements was further improved than the welded elements. Considerable amount of research has investigated the behavior of castellated steel beams and/or columns. However, most of these studies were confined on the lateral-torsional buckling of castellated steel beams, types of openings (either circular or hexagonal web openings), the buckling behavior of castellated steel columns and the deflection of castellated beams. Nevertheless, no research seemed to have investigated the behavior of castellated steel beam-column elements. Therefore, this study will have the precedency in contributing towards the implementation of castellated beam-column elements into numerous structures such as office buildings, car parks, shopping centers, hospitals almost any structure with a suspended floor. This paper reports on short and long castellated steel beam, column and beam-column elements. Experimental study was developed to examine the behavior of castellated steel beam-column elements. Linear interaction diagrams were plotted to illustrate the effect of castellation on linear buckling behavior of castellated elements. Also, non-linear interaction diagrams were plotted to investigate the effect of castellation on the element capacity. This paper presents different castellation ratios (R), where R equals the ratio between castellated element and the original solid web element. Therefore, enhancement diagrams were plotted to investigate the best castellation ratio, enhancement diagrams plotted. The current study aims at investigating the effect of castellation on the beam-column elements strength. This study presents short and long beam-column elements with three different (e/d) ratios, where (e) is the load eccentricity and (d) is the section depth. The first (e/d) ratio is equal to zero so these elements represent the pure axial elements, the second (e/d) ratio is equal to 0.5 and the third (e/d) ratio is equal to 0.75. Also, two castellation ratios (R) were considered 1.3 and 1.5. The castellation ratio (R) is the ratio between castellated section depth to the original section depth. The results of experimental work performed in this research will be compared to the finite element non-linear analysis to validate the applicability of finite element as a numerical tool.

2 Experimental Program

Twelve specimens are tested to study the effect of castellation on the beam column elements behavior and strength. One of the commercially available IPE sections was chosen carefully with the standard section dimensions as shown in Fig 1.

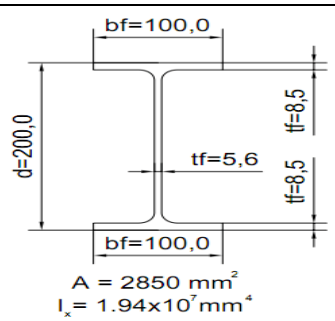


Fig. 1. *IPE200 Cross-section used for experimental work in this research.*

The experimental program consisted of 3 groups. Group I, was focused on the axial compression behavior of 4 short and slender I- section columns. The parameters considered were the effect of castellation ratio (R) and the slenderness ratio of the steel columns. The columns were instrumented to examine their behavior in terms of the load-axial displacement and load-lateral displacement responses. Groups II-III, were focused on the axial-flexural behavior of 8 short and slender I-section beam columns. The parameters considered were the effect of castellation ratio (R), the slenderness ratio of the steel sections and the eccentricity to section depth ratio (e/d ratio). Group II was focused on the structural behavior of 4 short and long I-beam column sections that had ratio of e/d = 0.5, while group III was focused on the structural behavior of 4 short and long I-section members that had ratio of e/d = 0.75. The I-sections were instrumented to examine their behavior in terms of the load-axial displacement and load-lateral displacement response. The identification of each specimen in further

sections of this research will indicate specimen length followed by castellation ratio followed by eccentricity to section depth. For example, L17-R1.3-e0.0 refers to specimen that has length of about 1700mm, castellation ratio equals to 1.3 and pure axial specimen, so e/d ratio equals to 0.0. The parameters of each group are shown in table 1.

2.1 Material Properties

Three tensile coupon tests are performed to specify the mechanical characteristics of tested steel specimens. The representative stress-strain curves of all tested specimens are shown in Fig 2. Mechanical properties of the tested steel specimens are shown in Table 2.

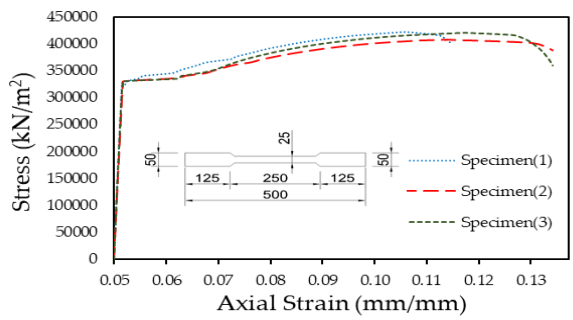


Fig. 2. *Tensile Stress-Strain Curve of Specimens.*

TABLE 1

TEST MATRIX OF CASTELLATED SPECIMENS TESTED IN GROUPS I, II AND III

Group	Specimen ID	L (mm)	$\lambda_y = \frac{L}{r_y}$	Castellation Ratio (R)	e/d
I	L17-R1.3-e0.0	1732.1	77	1.3	0.0
	L17-R1.5-e0.0	1847.5	83	1.5	0.0
	L30-R1.3-e0.0	2979.1	134	1.3	0.0
	L30-R1.5-e0.0	3233.2	144	1.5	0.0
II	L17-R1.3-e0.50	1732.1	77	1.3	0.5
	L17-R1.5-e0.50	1847.5	83	1.5	0.5
	L30-R1.3-e0.50	2979.1	134	1.3	0.5
	L30-R1.5-e0.50	3233.2	144	1.5	0.5
III	L17-R1.3-e0.75	1732.1	77	1.3	0.75
	L17-R1.5-e0.75	1847.5	83	1.5	0.75
	L30-R1.3-e0.75	2979.1	134	1.3	0.75
	L30-R1.5-e0.75	3233.2	144	1.5	0.75

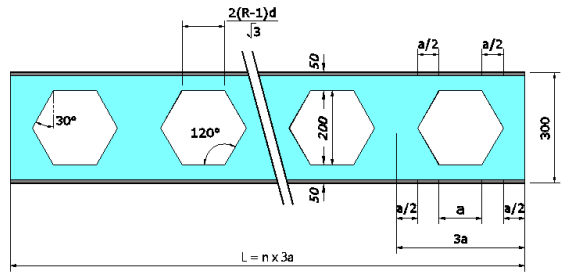
TABLE 2

Coupon	Width (mm)	Thick (mm)	Initial Length (mm)	Elastic Modulus E(Gpa)	Yield Strength (Mpa)	Ultimate Strength (Mpa)
Specimen (1)	21.7	6.1	221.6	186.4	331	421
Specimen (2)	23.9	6.1	221.4	189.6	321	407
Specimen (3)	23.8	5.85	211.2	213.8	303	420
Average	23.13	6.017	218	197	319	416

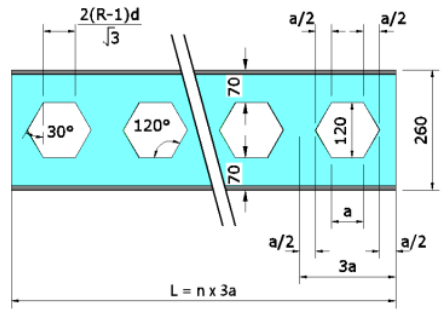
MATERIAL PROPERTIES AS PER THE COUPON TEST RESULTS

2.2 Castellated Beam Specimen Preparation

Section opening arrangement and dimensions, for R = 1.3 and R = 1.5 are shown in Fig 3. The steps of specimen fabrication from IPE200 to castellated element are shown in Fig 4. After fabrication, a loading plate was welded at one end of the specimen as shown in Fig 5.a. The boundary conditions for all test specimens were pin ended at both ends, the other end connections are shown in Fig 5.b.



a) Section dimensions R=1.5.



b) Section dimensions R=1.3.

Fig. 3. *Castellated element dimensions.*

2.3 Test Setup

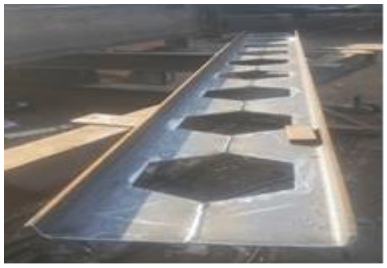
The end boundary conditions were pinned-pinned, so the specimen from one end was supported on the bearing plates by a simple connection to permit rotation, while, the upper end of the specimen was supported by using steel elements that also permitted rotation as shown in Fig 6. The specimen was permitted to move down, in the same time the specimen was restrained horizontally. Also, out of plan movement was restrained by using lateral support to ensure specimen out of plan stability.

a) Original IPE200 solid section.

b) Cutting path on the original section.

c) Cutting the specimen into two parts.

d) Alignment of the two parts.



f) Final fabricated castellated element.

Fig. 4. *Castellated beam fabrication process.*

a) Loading plate.

R=1.3 R=1.5

b) End connection for castellated elements.

Fig. 5. Specimen end preparation.

All specimens were tested under vertical load using one test setup as shown in Fig 7. Testing machine capacity was 1000 kn. Load cell position was based on e/d ratio. For column elements load cell was on the top center of the specimen while for beam column elements load cell was eccentric to satisfy the required e/d ratio. For group I, three LVDTs were used; one of them was mounted at the top loading plate to measure the vertical displacement, where the other two LVDTs were mounted at the mid-height to measure the lateral displacements as shown in Fig 8. For groups II and III four LVDTs were used; one of them was mounted at the top loading plate to measure the vertical displacement, where three LVDTs were mounted at the mid-height to measure the lateral displacements and the rotations as shown in Fig 9.

a) Lower end connection *b) Upper end connection.*

Fig. 6. *Specimen ends connection.*

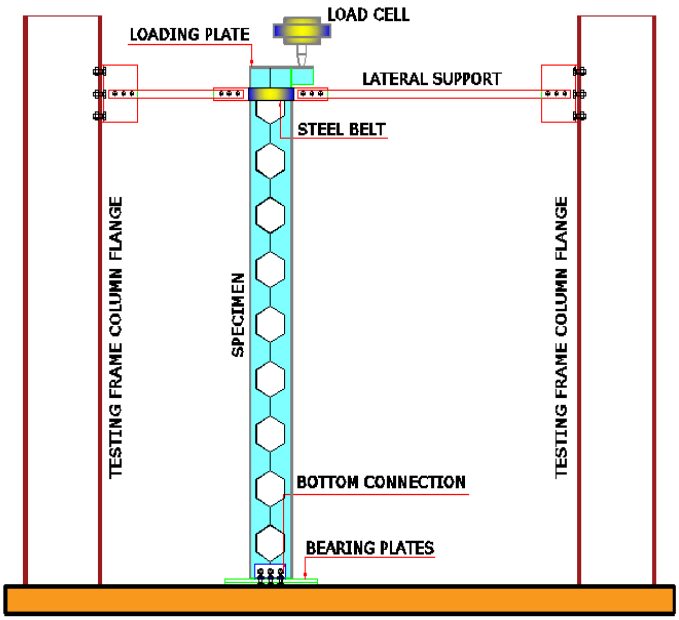
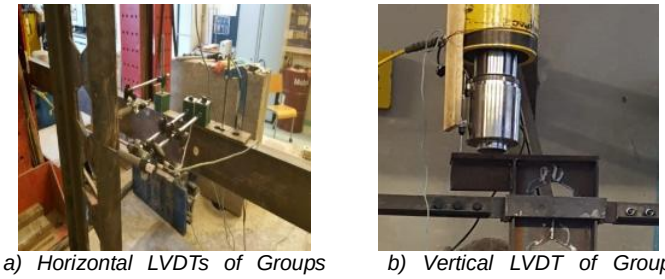


Fig. 7. Test setup.

a) Horizontal LVDTs of Group I *b) Vertical LVDT of Group I.*

Fig. 8*. Arrangement of LVDTs of Group I*



a) Horizontal LVDTs of Groups II&III *b) Vertical LVDT of Groups II&III.*

Fig. 9*. Arrangement of LVDTs of Groups II&III.*

3 Test Results

3.1 Failure Modes

Group I: pure axial loading

Four castellated columns were tested two of them were short and the others were long, by applying axial load at the top center of the specimens. The experimental tests showed that the failure mode of the short columns was the local buckling and web shear deformations, while the failure mode of long columns was overall flexural buckling as shown in Fig 10.

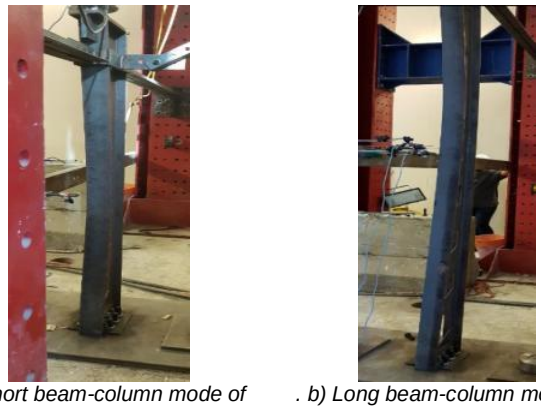
Groups II-III: beam columns members of (e/d=0.5 & e/d=0.75)

Eight castellated beam columns were tested four of them short

and the others are long, by applying load at the top loading plate. The experimental tests showed that the failure mode of the short and long beam column elements was lateral torsional buckling mode as shown in Fig 11.

a) Short column mode of failure *b) Long column mode of failure*

Fig. 10. *Failure mode in castellated columns (e = 0).*



a) Short beam-column mode of failure *. b) Long beam-column mode of failure*

Fig. 11. *Failure mode in castellated short & long beam-columns (e/d=0.5 & e/d=0.75).*

3.2 Ultimate Capacity and Load Deformation Relation

The load-axial displacement and the load-lateral displacement curves of the studied pure axial short columns is presented in Fig 12. It can be noticed that, column that has castellation ratio (R=1.3) can sustain axial load greater than that has castellation ratio (R=1.5) due to the increase in the section depth. That is means the effect of the local buckling in the element that has castellation ratio (R=1.5) is greater than that has castellation ratio (R=1.3). It can be concluded that, the effect of local buckling increases by increasing the castellation ratio (R). Also, the load-axial displacement curve and the load-lateral displacement curves of the studied pure axial long columns is presented in Fig 13. It can be noticed that; column that has castellation ratio (R=1.3) can sustain axial load greater than that has castellation ratio (R=1.5) due to the increase in section depth), this increase in section depth increases the effect of shear deformation that occurs at the openings locations specially in the first and last openings, means, the effect of shear deformation increases by increasing castellation ratio (R). The failure loads and corresponding displacement of pure axial columns are summarized in Table 3. The load-lateral displacement curves of the studied beam column elements are presented in Figs 14-15. It can be noticed that, short and long elements that have e/d = 0.50 can sustain load greater than others that have e/d = 0.75, so the increase in eccentricity of castellated beam-column elements decreases the axial load capacity. Fig 16 shows that beam column elements that have e/d = 0.75 can sustain bending moments greater than that have e /d =0.5, therefore, the increase in the eccentricity of castellated beam-column elements increases the moment capacity.

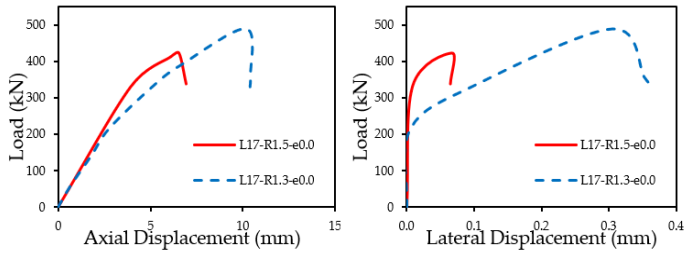


Fig .12. *Load deformation curves of pure axial short columns.*

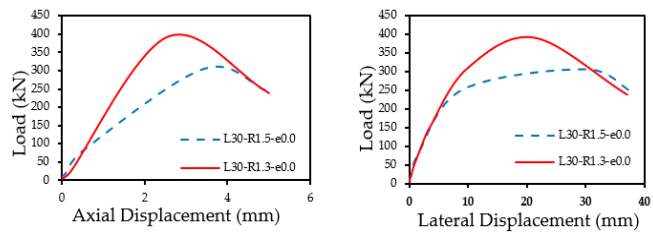


Fig. 13. *Load deformation curves of pure axial long columns.*

TABLE 3

ULTIMATE CAPACITY AND DISPLACEMENT AT FAILURE OF TESTED PURE AXIAL SPECIMENS

Group	Specimen ID	Ultimate compressive Load (kN)	Axial displacement (mm)	Lateral displacement (mm)
I	L17-R1.3-e0.0	490.11	10.2	0.31
I	L17-R1.5-e0.0	423.17	6.54	0.07
I	L30-R1.3-e0.0	392.7	2.56	21.19
I	L30-R1.5-e0.0	305.81	3.42	29.95

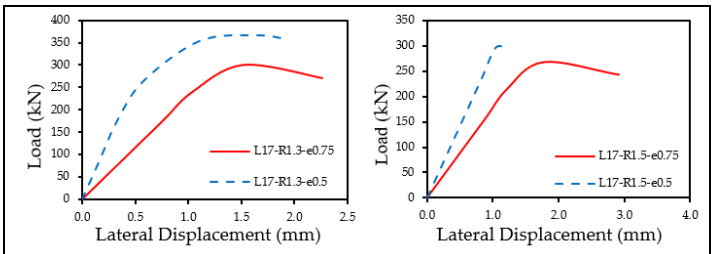


Fig. 14. *Load deformation curves of short beam-column specimens.*

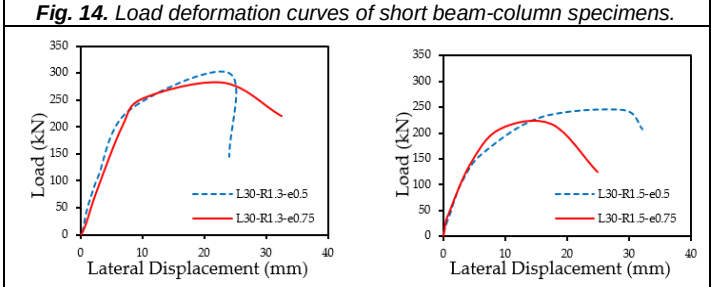


Fig. 15. *Load deformation curves of long beam-column specimens.*

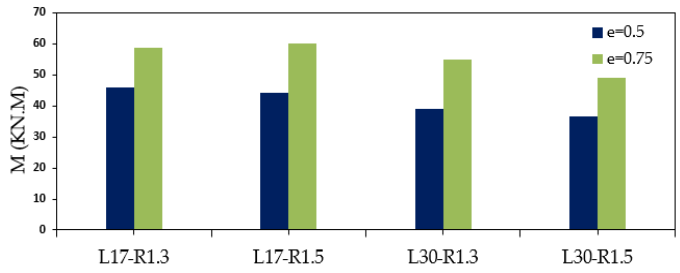


Fig. 16. *Moment capacity of beam column elements.*

4 Finite Element Simulation

4.1 Finite Element Model

Nonlinear buckling analysis using ANSYS.12.0 [17] is utilized to illustrate the effect of castellation upon castellated column and beam column ultimate failure load. The section dimensions shown in Fig 1 are adopted for this analysis. SHELL91 element is used to model both flanges and web, which is suitable for modeling thin to moderately-thick shell structures. It is an 8-nodes element that has six degrees of freedom at each node; translations in three directions, and rotations about the three directions. SHELL91 is well-suited for linear, large rotation, and/or large strain nonlinear applications.

4.2 Boundary Conditions and Load Application

All specimens were restrained laterally at both upper and lower end nodes. Therefore, the specimen was prevented from lateral translation at both ends. Axial load was applied to the top node of the element according to e/d ratio. To ensure the finite element model stability a mid-bottom point was restrained in x, y and z directions, top and bottom boundary conditions is shown in Fig 17. Figure 18 shows the finite element mesh for L17-R1.3-e0.0, L17-R1.3-e0.5 and L17-R1.3-e0.75 as examples.

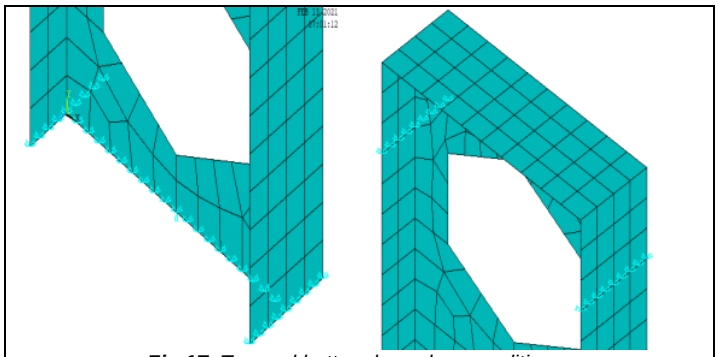
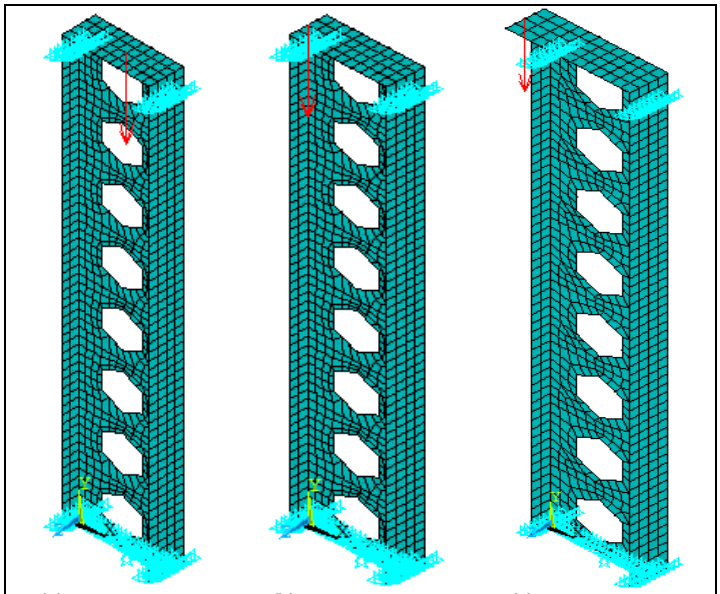


Fig 17. *Top and bottom boundary conditions.*



(a) L17-R1.3-e0.0 *(b) L17-R1.3-e0.50* *(c) L17-R1.3-e0.75*

Fig .18*. Created models for L17-R1.3 elements*

4.3 Mesh Convergence and Initial Imperfection

The finite element model results became converge when the increase in the mesh density has a neglectable effect. A total of three element sizes 10mm, 20mm and 30mm are evaluated. The results found that the mesh size of approximately 30mm for web and flanges provides adequate choice to be used in the modeling and save the computational time. Furthermore, initial geometric imperfections were provided in structural steel members as a result of the fabrication process.

4.4 Finite Element Validation

Figs 19 to 22 present comparisons between the load-displacement curves obtained from the tests and finite element analysis. Likely concluding that, the FEM can be used to predict castellated element behavior. The ultimate axial load of each specimen obtained from both experiments and finite element modeling are presented in Table 4. It can be noticed that the ratio of experimental to the finite element capacity for all specimens' ranges between 1.0 to 1.06.

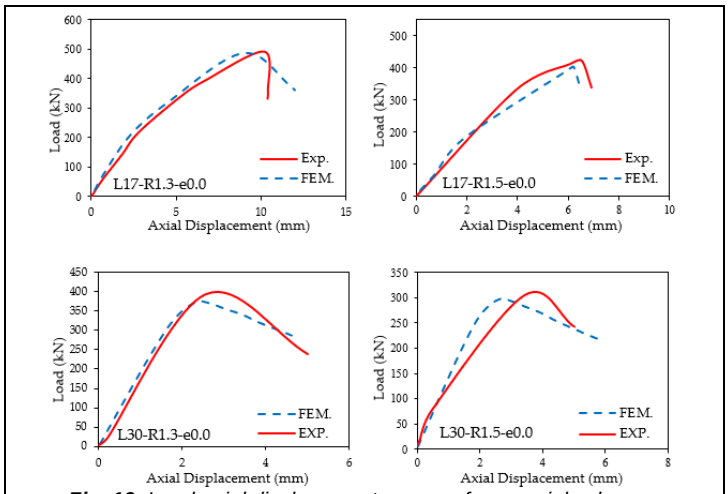


Fig. 19*. Load-axial displacement curves of pure axial columns.*

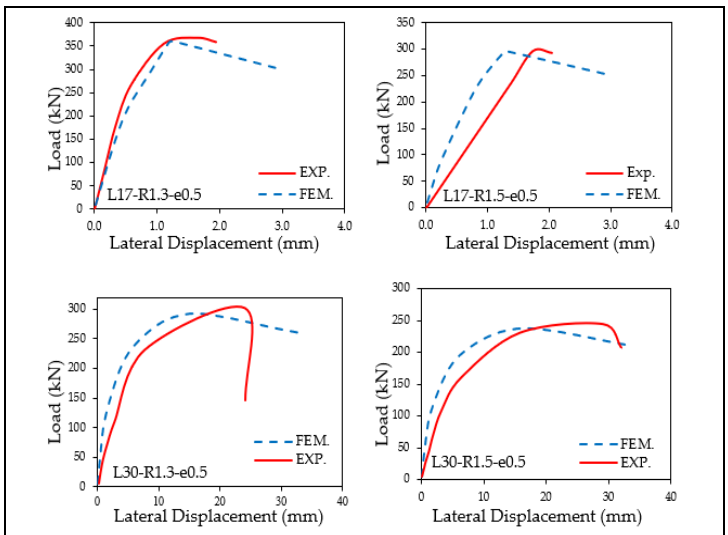


Fig. 20. *Load-lateral displacement curves of pure axial columns.*

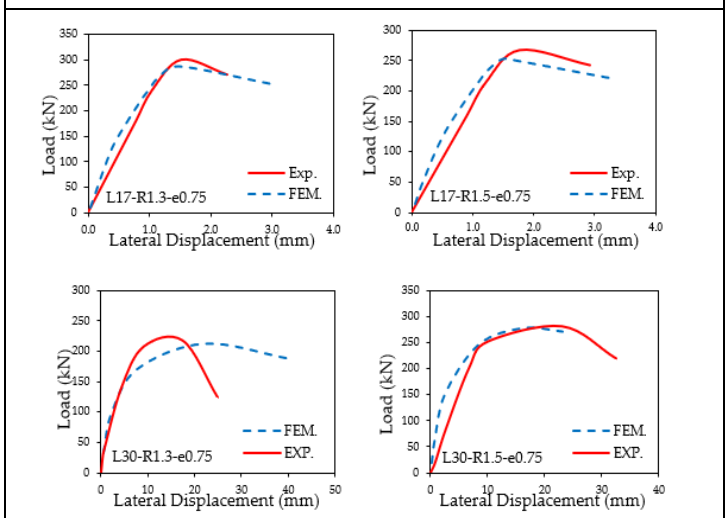


Fig. 21. *Load-lateral displacement curves of beam-column elements (e/d=0.5).*

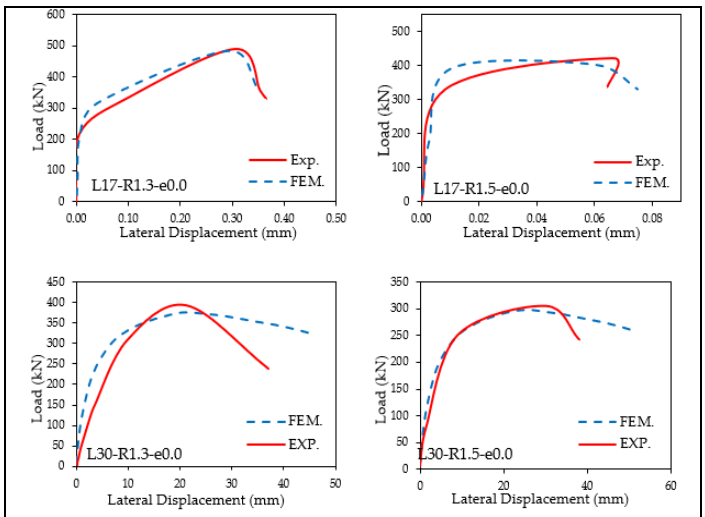


Fig. 22. *Load-lateral displacement curves of beam-column elements (e/d=0.75).*

Table 4

Comparison between Experimental and Finite Element Ultimate Capacities

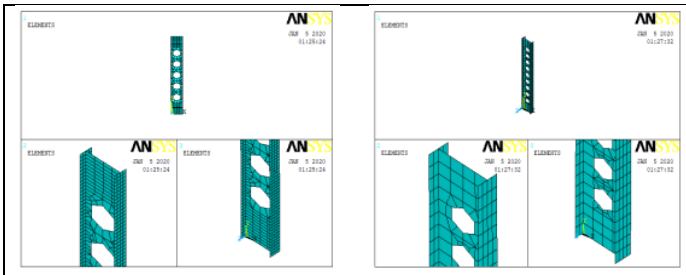
Group	Specimen ID	Ultimate compressive Load (kN.) Exp.	F.E.	Exp./F.E.
I	L17-R1.3-e0.0	490.11	485.12	1.01
	L17-R1.5-e0.0	423.17	402.71	1.05
	L30-R1.3-e0.0	392.7	376.68	1.04
	L30-R1.5-e0.0	305.81	297.82	1.03
II	L17-R1.3-e0.50	366.74	360.27	1.02
	L17-R1.5-e0.50	295.65	295.84	1.0
	L30-R1.3-e0.50	301.3	292.06	1.03
	L30-R1.5-e0.50	244.87	236.94	1.03
III	L17-R1.3-e0.75	300.17	286.88	1.05
	L17-R1.5-e0.75	267.44	253.19	1.06
	L30-R1.3-e0.75	282.11	278.5	1.01
	L30-R1.5-e0.75	217.8	211.88	1.03
Mean (μ)				1.03
Standard deviation (S)				0.017
Coefficient of variation (V) = S / μ				0.0165

5 Parametric Study

Castellated steel elements can be used as column, beam and beam column elements. For column elements, the existence of web openings causes local buckling of the column web at the first and the last web opening. A parametric study is conducted utilizing the verified finite element model to investigate the possibility of prevention of local buckling that may occur at the web to improve the axial capacity of the castellated column elements.

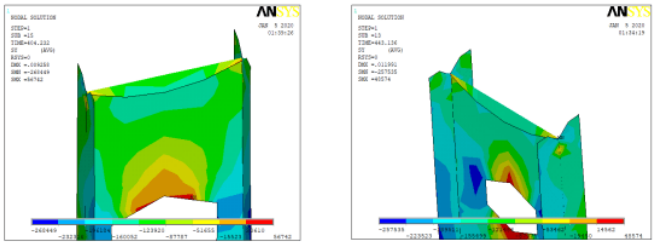
5.1. Enhancement of Axial Capacity of Castellated Columns

A parametric study to prevent the local buckling of the web plate of castellated steel columns to improve their axial capacity is conducted by using ANSYS.12.0. Two expansion ratios are used (R =1.3 & R = 1.5), also L17 elements are used for this study. In this study the upper and lower openings are closed, the study considered two stages, first stage with the same web thickness of IPE200 second stage, using a web plate of 8.0 mm thickness as shown in Fig 23. The mode of failure when the same web thickness is used (for R =1.3 and 1.5) is local buckling of web as shown in Fig 24, where the mode of failure when web plate of thickness = 8.0 mm is used (for R = 1.3&1.5) is the flexural buckling mode of failure as shown in Fig 25.



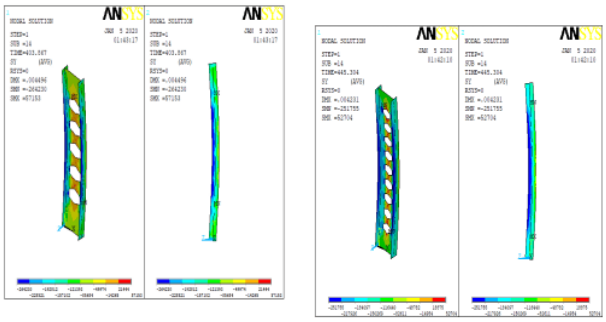
a) R=1.5 element. *b) R=1.3 element.*

Fig. 23. *Closed upper and lower web openings.*



a) R=1.5 element. *b) R=1.3 element.*

Fig. 24. *Local buckling for R=1.3 and R=1.5-web plate thickness = 5.6mm*



a) R=1.5 element. *b) R=1.3 element.*

Fig. 25. *Flexural buckling for R=1.3 and R=1.5-web plate thickness = 8.0mm.*

Table 5 illustrates the axial capacity for different castellated elements used in the study, for each castellation ratio three axial capacities are given: P_{no} for castellated column with all openings are opened, and P_n for castellated columns with upper and lower openings closed with a web plate thickness = 8.0mm, and with the same web plate thickness = 5.6mm. For R = 1.3 and R = 1.5 closing first and last openings increase the axial capacity, but for R = 1.5 the improvement is lower than for R = 1.3 due to the increase in web depth that led to increase in the effect of local buckling and the shear deformations that may occur in the flanges. Also, closing these opening does not improve the axial capacity to reach the axial capacity of the solid web elements as the web openings causes shear deformations in flanges that not found in the solid web elements as shown in Fig 26. Therefore, castellation does not improve the axial capacity of the elements due to the effect of local buckling in the web and shear deformations that occur in the flanges. However, to improve the behavior of the castellated axial elements closed openings may be used for the upper and the lower openings, these closed openings improve the axial capacity but do not lead to reach the axial capacity of solid web element.

TABLE 5

AXIAL CAPACITY FOR DIFFERENT AXIAL CASTELLATED COLUMN ELEMENTS USED IN THE STUDY.

Castellation Ratio (R)	Element	Axial Capacity (kN)	% of improvement $\frac{(P_n - P_{n_o})}{P_{n_o}} * 100$
R = 1.3	L17-R1.3-e0.0 (all openings are opened) - Pno	369.97 (Pno)	
	L17-R1.3-e0.0 Upper& Lower openings are closed with the same web thickness = 5.6mm	443.14 (Pn)	19.8 %
	L17-R1.3-e0.0 Upper& Lower openings are closed with web thickness = 8.0mm	445.3 (Pn)	20.36 %
R = 1.5	L17-R1.5-e0.0 (all openings are opened)- Pno	379.99 (Pno)	
	L17-R1.5-e0.0 Upper& Lower openings are closed with the same web thickness = 5.6mm	404.23 (Pn)	6.38 %
	L17-R1.5-e0.0 Upper& Lower openings are closed with web thickness = 8.0mm	403.23 (Pn)	6.11 %

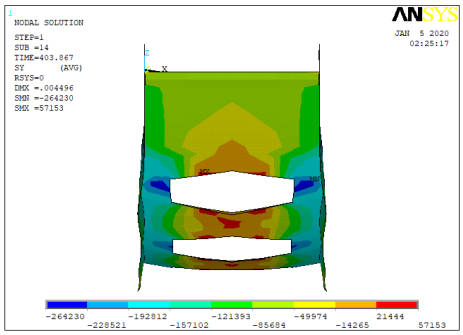


Fig. 26*. Flanges shear deformations.*

5.2. Beam and Beam-column Parametric Study

A parametric study to investigate the effect of castellation on beam and beam-column elements is carried out by using ANSYS.12.0. Four castellation ratios (R) are considered 1.2, 1.3, 1.4 and 1.5 in addition to solid web element (R=1). Four e/d ratio 0.5, 0.75, 1.5 and ∞ (beam element) are considered. All elements are considered buckle about their major axis so lateral supports are provided to ensure that λy < λx. Yield strength is considered = 240Mpa, Young`s modulus = 200 Gpa and tangent modulus for the steel was assumed equal to 0.5 percent of its elastic modulus. Fig 27 presents moment capacity versus castellation ratio (R) for different e/d ratio. Also, table 6 gives the moment capacity for different beam-columns considered on the current study, while table 7 gives the axial capacity for the same members.

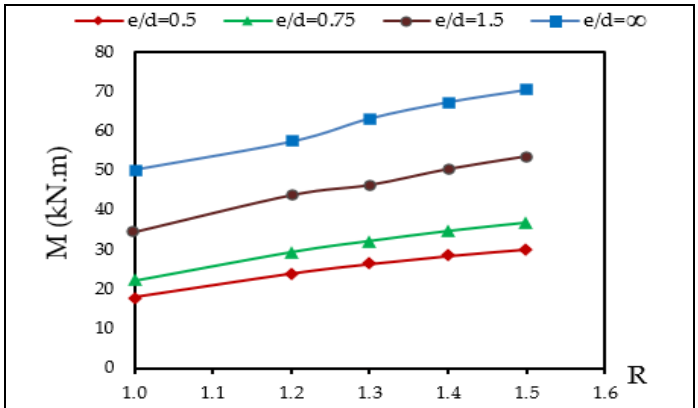


Fig. 27*. Moment capacity versus castellation ratio (R).*

TABLE 6

MOMENT CAPACITY FOR DIFFERENT BEAM AND BEAM COLUMN ELEMENT USED IN THE STUDY.

Element ID	Element Length (mm)	Pn (kN)	% of improvement $\frac{(P_n - P_{n(R=1)})}{P_{n(R=1)}} * 100$
L80-R1.0-e0.5	8036.71	201.92	
L80-R1.2-e0.5	8036.71	223	10.44%
L80-R1.3-e0.5	8015.99	226.32	12.08%
L80-R1.4-e0.5	8036.71	226.18	12.01%
L80-R1.5-e0.5	7967.43	223.52	10.7%
L80-R1.0-e0.75	8036.71	166.59	
L80-R1.2-e0.75	8036.71	182.75	9.7%
L80-R1.3-e0.75	8105.99	184.53	10.77%
L80-R1.4-e0.75	8036.71	184.5	10.75%
L80-R1.5-e0.75	7967.43	182.5	9.55%
L80-R1.0-e1.5	8036.71	128.44	
L80-R1.2-e1.5	8036.71	135.63	5.6%
L80-R1.3-e1.5	8105.99	132.55	3.2%
L80-R1.4-e1.5	8036.71	133.74	4.13%
L80-R1.5-e1.5	7967.43	132.75	3.36%

TABLE 7

AXIAL CAPACITY FOR DIFFERENT BEAM COLUMN ELEMENT USED IN THE STUDY.

Element ID	Element Length (mm)	Mn (kN.m)	% of improvement $\frac{(M_n - M_{n(R=1)})}{M_{n(R=1)}} * 100$
L80-R1.0-e0.5	8036.71	20.19	
L80-R1.2-e0.5	8036.71	26.76	32.54%
L80-R1.3-e0.5	8015.99	29.42	45.72%
L80-R1.4-e0.5	8036.71	31.67	56.86%
L80-R1.5-e0.5	7967.43	33.53	66.07%
L80-R1.0-e0.75	8036.71	24.99	
L80-R1.2-e0.75	8036.71	32.9	31.65%
L80-R1.3-e0.75	8105.99	35.98	43.98%
L80-R1.4-e0.75	8036.71	38.75	55.06%
L80-R1.5-e0.75	7967.43	41.06	64.31%
L80-R1.0-e1.5	8036.71	34.59	
L80-R1.2-e1.5	8036.71	43.83	26.71%
L80-R1.3-e1.5	8105.99	46.4	34.14%
L80-R1.4-e1.5	8036.71	50.42	45.76%
L80-R1.5-e1.5	7967.43	53.62	55.02%
L80-R1.0-e∞	8036.71	56.15	
L80-R1.2-e∞	8036.71	64.14	14.23%
L80-R1.3-e∞	8105.99	70.67	25.86%
L80-R1.4-e∞	8036.71	75.29	34.09%
L80-R1.5-e∞	7967.43	78.76	40.27%

From table 6, it can be noticed that, the increase in capacity due to castellation for elements that have e/d = 0.5 varies from 32.54 % to 66.07%, where varies from 31.65% to 64.31% for elements that have e/d = 0.75, also for elements that have e/d = 1.5 varies from 26.71% to 55.02 %. For beam elements the increase in capacity due to castellation varies from 14.23% to 40.27%. Also, it can be noticed that the increase in capacity due to castellation increases with increase in castellation ratio (R) due to the increase in depth that led to increase in moment of inertia and elastic section modulus. From table 7, it can be noticed that, the increase in the (e/d) ratio the decrease in the axial capacity, also, for elements that have (e/d) ratio = 0.5 and 0.75, the increase in castellation ratio the increase in the axial capacity except for elements that have castellation ratio (R) = 1.5 due to the large effect of shear deformation that may occur at the opening locations. For elements that have (e/d) ratio equals to 1.5, the effect of castellation ratio (R) on the axial capacity is minimal.

6 CONCLUSIONS

An experimental investigation and finite element non-linear analyses have been conducted on twelve short and long castellated steel column and beam column elements. The F.E.A results of ultimate capacities and deformed shapes were verified with experimental counterparts and were found to be in a good agreement. For short columns, castellation decreases ultimate capacity due to the effect of local buckling that occur in the web, also the increase in castellation ratio (R) decreases the ultimate capacity due to the increase in section depth that led to increase in the effect of local buckling. To improve ultimate capacity of castellated short column elements, the upper and lower openings are required to be closed. For long columns, castellation decreases the ultimate capacity due to the effect of shear deformation that occur in the flanges in the location of the openings, also the increase in castellation ratio (R) increases the shear deformation that led to less ultimate capacity of the element. So, castellation decreases the ultimate capacities of short and long columns. For beam and beam-column castellated elements, a parametric study was carried out to investigate the effect of castellation on the bending capacity. Four castellation ratios (R) were considered, in addition to solid web elements (R=1). Solid web elements were considered to investigate the improvement that occurs due to castellation rather than solid web elements. Four (e/d) ratios were considered to study the effect of load eccentricity on the element capacity, also, pure bending castellated elements were considered. All elements were considered to be restrained against out of plan buckling, therefore all elements buckled about major axes. The increase in capacity due to castellation for elements that have e/d = 0.5 varies from 32.54 % to 66.07%, where varies from 31.65% to 64.31% for elements that have e/d = 0.75, also for elements that have e/d = 1.5 varies from 26.71% to 55.02 %. For beam elements the increase in capacity due to castellation varies from 14.23% to 40.27%. Also, castellation is not beneficial in case of pure axial elements due to the effect of local buckling and shear deformations that occur at the opening locations especially at the first and last opening. Castellation increases beam-column and beam element capacity due to the increase in depth that leads to increase in section major moment of inertia and major section modulus. The increase in the (e/d) ratio decreases the castellated element capacity, so improvement decreases for pure bending castellated elements rather than beam column elements. Therefore, castellation improves the behavior and the strength of beam-column elements rather than column and beam elements. Structural designers have long been trying to develop minimum cost solutions, castellation provides a good benefit, the increase in the element capacity for beam and beam-column elements without any addition weight, therefore castellation can provide a useful solution for the designer to develop minimum cost. Therefore, for beam and beam-column castellated element, if the designer provides sufficient lateral supports to guarantee that, the major buckling will govern, castellation will be a benefit solution.

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