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Effect of External Confining Pressure on Concrete Columns' Strength

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Abstract: Numerous methodologies and techniques have been devised to strengthen columns, encompassing concrete and steel jackets, as well as wrapping with carbon or E-glass fibers. In this study, it is employed on a previously proposed technique by authors (Al-Tuhami, 2000, Al-Tuhami & Sakr, 2000) to strengthen columns. This approach has demonstrated its efficacy in augmenting the load capacity of columns and ameliorating the performance of reinforced concrete columns through testing 8 samples with multiple variables such as external confinement force value, concrete strength, load centricity, and spacing between steel plates. The empirical results unequivocally indicate a substantial improvement in the ability to withstand elevated vertical loads when concrete columns are strengthened with externally encircling steel angles and horizontal stirrups. To achieve more favorable outcomes, it became evident that the connection between beams and columns necessitates strengthened. Moreover, the condensation on the weaker parts of the column on top and bottom warrants attention to optimize results through an increase in the quantity of horizontal stirrups at both top and bottom of the columns. Finite element models, utilizing ABACOUS, were developed to investigate the behavior of these strengthened columns. These models were validated and refined through meticulous comparison with experimental results, demonstrating a good agreement between the two approaches.

Keywords: Column, Steel jacket, Load capacity, Steel angles, Strengthening, Confining pressure.

1. Introduction

The enhancement of load-bearing capacity in concrete elements, particularly columns, constitutes a pivotal endeavor in the preservation of historic and archaeological structures. Such augmentation facilitates the optimal utilization of these edifices and paves the way for potential functional transformations. To achieve the enhancement of load-bearing capacity of columns, there are a wide range of techniques collectively aim to elevate column strength, enhance stiffness, and augment ductility. One of the most widely used techniques is concrete column jacket where it can adapt to columns of diverse shapes and dimensions. Moreover, these methods often necessitate neither specialized labor nor costly materials. The additional reinforced concrete section typically comprises supplementary vertical bars, stirrups, and shear connectors. Al-Enin et al. [1] identified two primary factors that can diminish the ultimate load capacity of a strengthened column such as an increase in the spacing between transverse hoops and an increase of the cross-section rectangularity ratio. Soliman et al. [2] conducted comparative analyses, demonstrating the superior efficacy of spiral stirrups in circular jackets compared to rectangular stirrups in rectangular jackets in enhancing column strength, stiffness, and ductility.

One effective technique for enhancing the load-bearing capacity of concrete columns involves the application of fiber-reinforced polymer (FRP) jackets. These jackets, typically composed of glass or carbon fibers, are wrapped around the original column, resulting in a substantial increase in strength and durability. Wange et al. [3] conducted investigations on square and rectangular columns reinforced with six layers of GFRP sheets. Their findings revealed a remarkable augmentation in ultimate load capacity, with increases of 100% and 35% respectively for square and rectangular columns. Mahfouz et al. [4] similarly explored the impact of GFRP and CFRP jackets on circular and rectangular columns. Their results demonstrated a substantial enhancement in ultimate load capacity, reaching up to 253% and 258% for circular columns and 126% and 200% for rectangular columns, respectively. However, they observed a diminution in load capacity when the rectangularity ratio of the columns increased. Amin et al. [5] presented a case study of an abandoned three-story school building, focusing on the application of CFRP jacketing to strengthen reinforced concrete columns. The study's outcomes unequivocally indicated a significant improvement in the behavior of these columns as the number of CFRP layers increased. This enhancement is attributed to the confinement effect exerted by the FRP jacket.

The technique of strengthening concrete columns with steel jackets offers several distinct advantages over alternative methods. Notably, it results in a negligible increase in weight and a minimal expansion of the column's cross-section. Moreover, this technique enhances the ductility and stiffness of the original column while significantly augmenting its shear strength, particularly when subjected to lateral loads. Jirsa [6] posits that a thin rectangular steel jacket can serve as a highly effective retrofit measure for reinforced concrete columns exhibiting inadequate shear resistance. Lisantono et al. [7] have conducted extensive research on the application of steel jackets to strengthen reinforced concrete columns. Their investigations have contributed significantly to the understanding and refinement of this technique. A technique proposed by EL Tuhami [8] involves the application of steel jackets with embedded horizontal steel plates, which are welded to the corners under external pressure. This method, known as active confinement, has been demonstrated to significantly enhance the ductility, durability, stiffness, ultimate compressive capacity, and shear resistance of concrete columns. EL Tuhami et al. [8] revealed that increasing the area of the corner steel angles in the steel jackets leads to a substantial augmentation of the ultimate capacity, reaching up to 112%. Furthermore, EL Tuhami et al. [9] observed a notable increase in the ultimate load capacity of strengthened columns, ranging from 16% to 86% compared to the original columns. Furthermore, El Tuhami [9] highlighted the positive correlation between the applied confining pressure and the resulting increase in ultimate capacity and ductility. Additionally, the study demonstrated that augmenting the number of steel pattern plates further enhances the ultimate load capacity and ductility of the columns. The main objective of the present research is to generate a finite element model using the Abaqus program that can provide accurate results for columns strengthened by this technique and achieve a high-accuracy mathematical formula to calculate the column's failure load after strengthening by this technique.

2. EXPERIMENTAL PROGRAM

2.1 Test Parameters and Specimen Designation

The present research program is centered on the behavioral analysis of reinforced concrete columns strengthened with pressurized steel jackets. The employed strengthening technique involves the application of steel jackets affixed to the original column's corners through an external pressure mechanism. Specifically, four steel angles were strategically positioned along the column's corners, followed by the application of external pressure in two directions perpendicular to the direction of the imposed vertical loads. Subsequently, steel batten plates were welded horizontally across the corner angles to apply external pressure. The experimental program encompassed a total of eight rectangular, short, strengthened concrete columns specimens, as detailed in Table 1. These specimens served as half-scale models of typical prototype columns commonly encountered in residential structures, measuring 150 x 300 x 1200 mm as in Figure 1. The primary parameters investigated across the three experimental groups (A, B, and C) are as follows:

1. External confinement force value.
2. Load eccentricity applied on the column.
3. Concrete strength.
4. The Spacing between steel plates.

Table 1: Details of the tested specimens.

Group	Specimen number	Dimensions, mm (L*W*H)	F_{cu} N/mm^2	Column reinforcement: Main steel	Column reinforcement: stirrups	Load type	Angles dimensions mm	Horizontal plates dimensions	External Confining Torque N.m
A	1- control	150*300*1200 mm	20	Vertical bars 6ө10	horizontal stirrups 9ө8 @1200 mm	Concentric vertical load.	NA	NA	NA
	2						4-angle 40*40*4 mm	4*12 – horizontal plates 24 (100*50*5) + 24 (250*50*5)	100
	3								150
	4							4*16 – horizontal plates 32 (100*50*5) + 32 (250*50*5)	200
B	5- control	150*300*1200 mm	20	Vertical bars 6ө10	horizontal stirrups 9ө8 @1200 mm	Ecc = 50 mm	NA	NA	NA
	6						4-angle 40*40*4 mm	4*12 – horizontal plates 24 (100*50*5) + 24 (250*50*5)	100
C	7- control	150*300*1200 mm	50	Vertical bars 6ө10	horizontal stirrups 9ө8 @1200 mm	Concentric vertical load.	NA	NA	NA
	8						4-angle 40*40*4 mm	4*12 – horizontal plates 24 (100*50*5) + 24 (250*50*5)	200

All angles and steel plates are steel 37

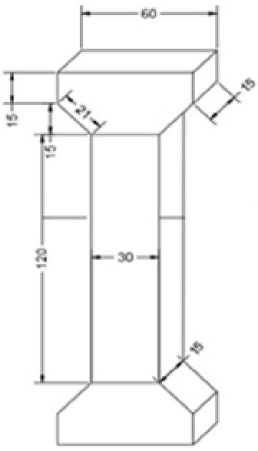


Figure 1: Column dimensions.

2.2 Concrete Mix

The ordinary Portland cement, sand, and coarse aggregate were used in concrete mix, in addition to water were mixed to generate the concrete required to cast columns. The target concrete mixes achieved maximum compressive strength with an average of 20 and 50 MPa as shown in Table 1.

2.3 Steel Reinforcement

The experimental program utilized three distinct diameters of steel bars: 10 mm and 12 mm high-tensile steel bars, and 8 mm mild steel bars. The vertical reinforcement consisted of six 10 mm diameter bars, encircled by nine 8 mm diameter horizontal stirrups. At the top and bottom heads, two primary 12 mm diameter bars were encompassed by four 10 mm diameter horizontal stirrups, as illustrated in Figure 2. The steel cages employed in the strengthening techniques comprised mild steel angles and batten plates.

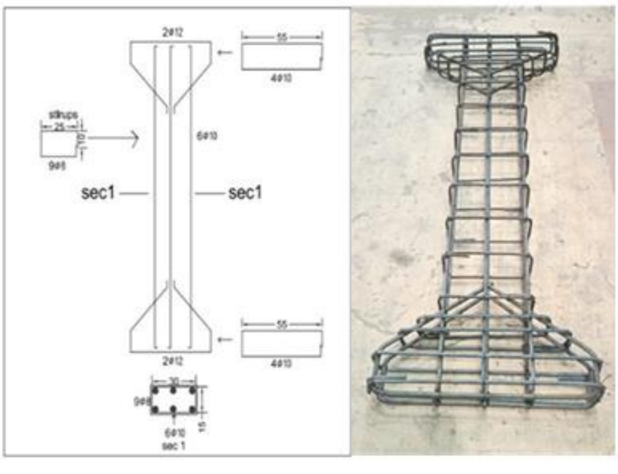


Figure 2: Reinforcement details of tested columns.

2.4 Casting and Strengthening

2.4.1 Wooden Mold

The molds for columns were made of 25 mm thick plywood sheets with very smooth faces and had the dimensions of the tested columns as shown in Figure 3. Six cubes of 15 cm size were also cast to determine the compressive strength of the concrete used in columns. Standard steel molds were used for the casting of these control specimens.

2.4.2 Casting Process

Before casting, the specimen molds were tightly assembled and checked for dimensional accuracy. The molds were cleaned well with an air jet and well-greased. The required cover was ensured by placing cover blocks made of cement mortar at intervals of about 20 mm center to center between stirrups and the mold. Concrete with F_{cu} = 20 and 50 N/mm^2 was used to cast the specimens in the horizontal direction.

Following the partial filling of the molds to approximately half the column's width, the concrete was subjected to both internal and external vibration. Subsequently, the upper half of the mold was filled with concrete and subjected to similar vibration, as depicted in Figure 4. Upon completion of vibration, the top surface was meticulously smoothed through trawling.

In conjunction with the eight columns being cast simultaneously, control specimens comprising six cubes were also fabricated for each concrete mix.

Figure 3: Columns wooden molds.

Figure 4: Casted specimen.

2.4.3 Strengthening Columns Before Testing

After 28 days of demolding the columns from casting, the concrete columns were ready for strengthening. Columns were strengthened as follows; four angles 40*40*4 mm were placed on the column corners with a length equal to 118 cm as shown in Figure 5 (i.e., shorter than the column height). Strengthening forms consisting of four angles with opening and to the direction were placed above the original angles. After that, third rows were placed in the two directions through openings and pressed with a torque wrench to achieve the desired torque specification. Then, batten plates with dimensions of 100 x 50 x 5 and 250 x 50 x 5 mm were welded on the four sides of the column through the corner angles at equal spacing and were intensified at the top and bottom. The steelhead (as in Figure 6) around the column head was fixed to prevent local failure and damage in the column head as it was not surrounded by the steel jacket like other parts of the column. Figure 7 shows that the plates were welded together with the corner angles.

Figure 5: Applying external pressure.

Figure 6: Columns steel head.

Figure 7: Welding horizontal plates.

2.5 Test Set-Up and Instrumentation

All columns were subjected to pure axial compressive loads within a controlled testing environment. The compression testing machines employed in these experiments were located within the Concrete Laboratory of the American University in Cairo. The columns were meticulously positioned and aligned within the machine's heads, ensuring that their centerlines coincided with the machine base's centerline except the two samples in group B were eccentric from the centerline with 5 cm.

2.5.1 Instrumentation and Control

A load cell was employed to monitor and record the applied axial loads. The corresponding relative displacements were measured using 100 mm Linear Voltage Differential Transducers (LVDTs), as illustrated in Figure 8.

Figure 8: Experimental test instrumentation.

2.5.2 Testing Procedure

The initial readings of the strains of the concrete were recorded by computer software (LAB VIEW) without any compression load. Also, a previous history of jack base displacement was entered into the software program. Then, the columns started to be loaded, and when the machine jack displacement reached the saved step in the computer program which was measured by (LVDT) attached to the machine jack, the operator of the machine closed the valves of the hydraulic oil, then, the machine stopped loading the column and computer software began to record the readings of the (LVDT). After finishing recording all data the machine operator opened the hydraulic oil valves and then the machine continued loading the column until the jack base displacement reached the next step saved in the computer program.

The tests were terminated when the load suddenly dropped to a low fraction of the maximum load reading, resulting in a complete failure of the specimen.

3. EXPERIMENTAL TEST RESULTS

The six standard cubes were tested to determine the compressive strength of the concrete and compared with the design values as summarized in Table 2 which give acceptable results. Test results include the records of ultimate load capacity for the original columns and the strengthened columns as summarized in Table 3.

Table 2: Compressive strength of standard tested cubes

Cube number	Tested Concrete Strength (Kg/cm^2)	Average Concrete Strength (Kg/cm^2)	Designed Concrete Strength (N/mm^2)
CU1	198	195.6	20
CU2	207		
CU3	182		
CU4	491	499.3	50
CU5	518		
CU6	489		

Table 3: Experimental test results.

Group	Specimen number	External confining torque N.m	Exp. column-load, kN	Enhance ratio Pu/Pu control	Confining pressure (δ_L) N/mm2
A	C1-control	NA	1030	100%	0.00
	C2	100	1396	136%	0.48
	C3	150	1475	143%	0.71
	C4	200	1604	156%	0.95
B	C5-control	NA	704	100%	NA
	C6	100	960	136%	0.48
C	C7-control	NA	1410	100%	NA
	C8	200	1998	142%	0.95

Figures 9 to 11 describe the relationship between vertical load and corresponding vertical displacement. Figure 9 shows the comparison between the control specimen without strengthening and the columns with strengthening in group A (F_{cu} = 20 MPa) in which the applied load was a concentric vertical load.

Figure 9 illustrates that the influence of external pressure increases the column's ability to resist vertical loads by approximately 36% to 56% of its capacity, and it also increases the ductility of the columns. Also, noting the elevation in the value of external pressure and reducing the distances between the outer battens increases the bearing capacity of columns.

Figure 10 shows the comparison between the specimens in group B (F_{cu} = 20 MPa). It was clear that the effect of lateral pressure increases the column's bearing capacity to bear eccentric vertical loads by about 36% of its capacity. However, there was no significant effect of confining pressure on increasing the ductility of the column.

Figure 11 shows the comparison between the specimens in group C (Fcu = 50 MPa). It was found that in the case of high-strength concrete, there is a significant effect of external pressure on increasing the capacity of the column by about 42% of its capacity, as well as the apparent increase in the ductility of the column due to high-strength concrete characteristics.

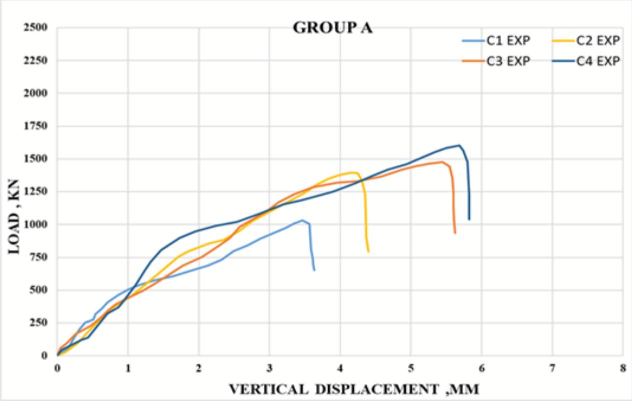


Figure 9: Vertical load versus vertical displacement of group (A).

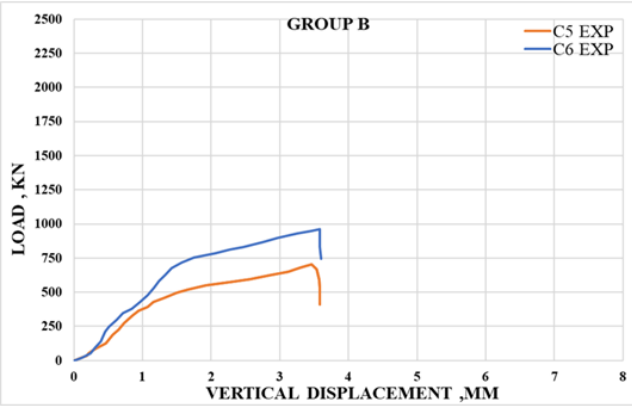


Figure 10: Vertical load versus vertical displacement of group (B).

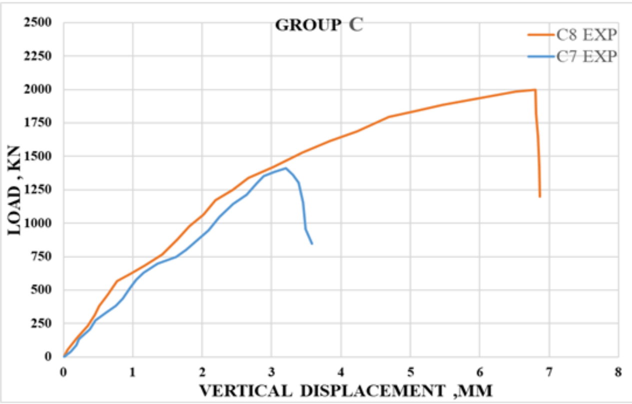


Figure 11: Vertical load versus vertical displacement of group (C).

4. FINITE ELEMENT MODELING AND VERIFICATION

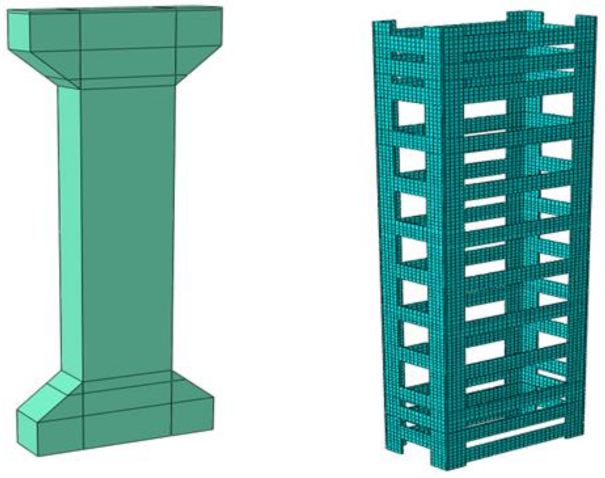
To gain a deeper understanding of the behavioral characteristics of concrete columns subjected to external confining pressure, a comprehensive finite element model was developed. The ABAQUS software, renowned for its efficacy in modeling transient dynamic events and highly nonlinear problems involving evolving contact conditions, was employed for this analysis. Drawing upon the previously delineated geometrical, structural specifications, and material properties of the experimental column models, a total of eight finite element models were meticulously constructed. These models served as virtual representations of the physical specimens, enabling detailed simulations and analyses of their behavior under various loading conditions.

4.1 Element Description

To construct the finite element model, a 3D FE mesh of concrete columns, reinforcement bars, steel angles, and steel plates were created using two main types of elements, solid element, and truss element (wire element).

4.1.1 Solid Element

Concrete columns and external steel cage were modeled using (C3D8R) brick element or solid element. Each node of these elements has three transitional degrees of freedom and three rotational degrees of freedom. This element was chosen as it can define the boundaries of the RC columns, steel angles, and plate properties as shown in Figures 12 (a, b). Additionally, it follows the constitutive law integration accurately and is very suitable for nonlinear static and dynamic analysis and allowing for finite strain and rotation in large-displacement analysis. The Part option in the Abaqus model tree defines all element types used in developing the finite element model.



(a) Modeling concrete part. **(b)** Modeling steel box part.
Figure 12: Solid element.

4.1.2 Truss Element

Truss elements are rods that can carry only tensile or compressive forces. They have no resistance to bending; therefore, they are useful for modeling reinforcement within other elements. (T3D2) element was selected in modeling reinforcement bars (see Figure 13) which were modeled as embedded elements in concrete blocks.

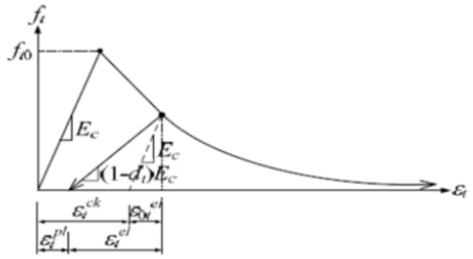
Figure 13: Truss element.

4.2 Material Modeling

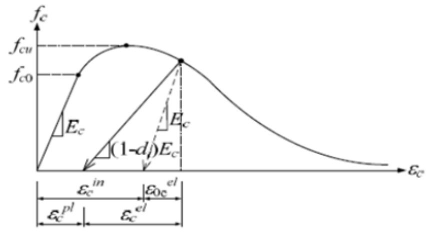
To model the RC columns, two different material models from ABAQUS [10] were used. These models include the concrete-damaged plasticity model (CDP) and the elastic-plastic model.

4.2.1 Concrete Damaged Plasticity Model

The concrete damaged plasticity model available in ABAQUS was used to model concrete due to the ability of this model to simulate concrete's plastic properties and consider the behavior of concrete softening in tension and compression [11] as shown in Figure 14. Table 3 presents the concrete elastic properties while Table 4 shows concrete damaged plasticity model parameters used in the analysis.



(a) Tensile behavior associated with tension stiffening.



(b) Compression behavior associated with compression hardening.

Figure 14: Concrete damaged plasticity model [11].

4.2.2 Elastic-Plastic Model

The elastic-plastic material model in ABAQUS was used to represent the behavior of steel bars and steel plates. Table 6 lists mechanical properties values used in finite element modeling to represent steel material.

Table 4: Elastic properties of concrete.

Parameter	Concrete 20 MPa	Concrete 50 MPa
Density, kg/m^3	2200	2200
Modulus of elasticity (Ec)	20164 MPa	29368 MPa
Poisson's ratio (υ)	0.18	0.2

Table 5: Concrete damaged plasticity parameters [10].

Parameter	Concrete 20 MPa	Concrete 50 MPa
Dilation angle	38	37
Eccentricity	0.12	0.12
f_{b0}/f_{c0}	1.36	1.36
K	0.67	0.67
Viscosity parameter	0.00001	0.00001
Ultimate compressive stress	20 MPa	50.1 MPa
Ultimate tensile stress	2.1 MPa	5.18MPa

Table 6: Mechanical properties of steel.

Parameter	High Tensile Steel	Mild Steel	Steel Plates and Angles
Density, kg/m3	7860	7860	7860
Modulus of elasticity (Es), MPa	210000	200000	200000
Poisson's ratio (υ)	0.3	0.3	0.3
Yield strength, MPa	574	280	300
Ultimate strength, MPa	663	375	435

4.3 Boundary Condition

Within the Abaqus model tree, boundary conditions were strategically applied through the "load" option, selecting the "create boundary conditions" function. As depicted in Figure 15, the base of the finite element models was restrained from vertical translation to emulate the experimental setup, wherein the columns were supported from below. Furthermore, the top and bottom heads of the columns were prevented from horizontal translation, as illustrated in Figure 16, mirroring the experimental conditions where the columns were supported by steel heads.

4.4 Loading

External confining pressure was simulated as a horizontal pressure acting in steel angles in both directions as a predefined field. Vertical load applied in steel plate over the column head and increased till failure as shown in Figure 17.

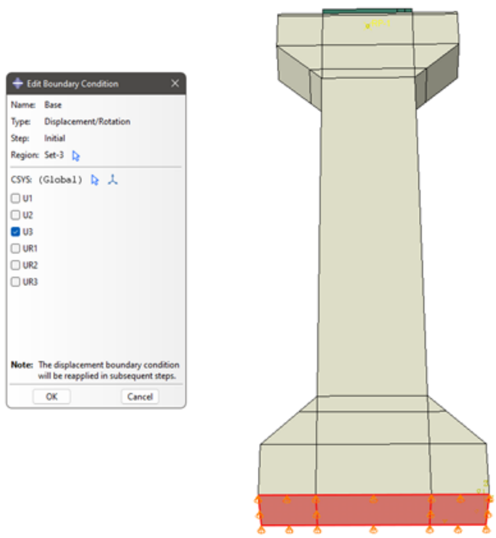


Figure 15: Boundary condition of column base.

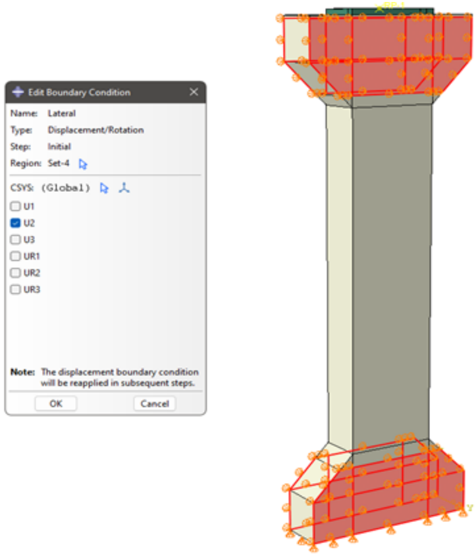


Figure 16: Boundary condition of column head.

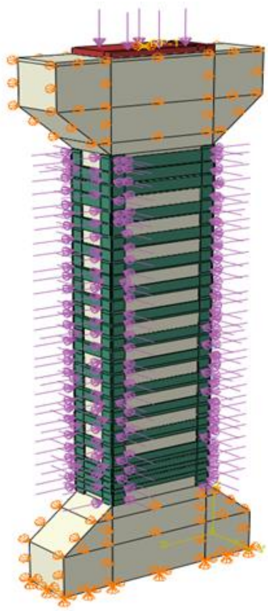


Figure 17: Loading of finite element model.

4.5 Finite Element Results

Figure 18 describes the relationship between vertical load and corresponding vertical displacement for columns in Group A.

The results of the analysis unequivocally demonstrate that the application of external lateral pressure significantly enhances the load-bearing capacity of concrete columns, approximating a 62% increase in their original capacity. Concomitantly, this technique leads to a notable augmentation in the column's ductility.

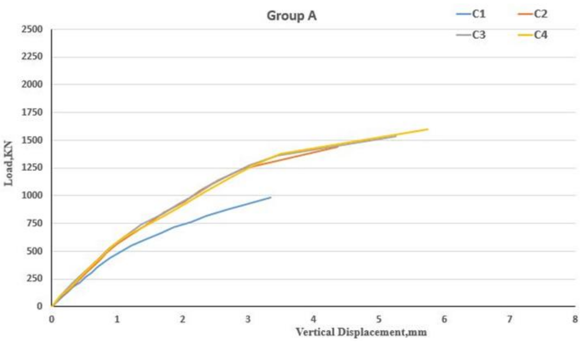


Figure 18: Vertical load versus vertical displacement of group (A).

Figure 19: Shows the comparison between the specimens in group B (Fcu = 20 MPa). The analysis revealed that the effect of external lateral pressure increases the column's bearing capacity to bear eccentric vertical loads by about 39% of its capacity. However, there was no significant effect of confining pressure on increasing the ductility of the column.

Figure 20: Presents a comparative analysis between the specimens in group C (F_{cu} = 50 MPa). It indicates that in the case of high-strength concrete, there is a significant effect of external pressure on increasing the capacity of the column by about 42% of its capacity, as well as increasing the ductility of the column.

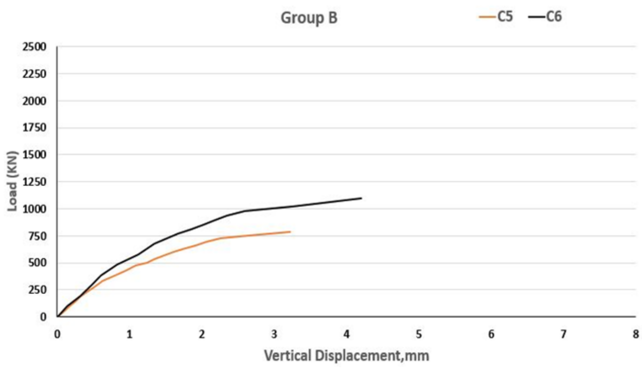


Figure 19: Vertical load versus vertical displacement of group (B).

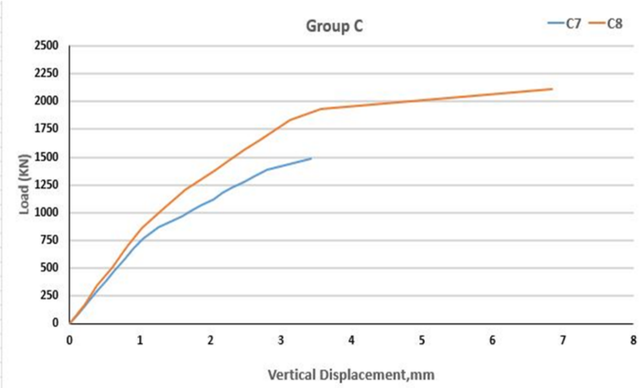


Figure 20: Vertical load versus vertical displacement of group (c).

5. COMPARISON BETWEEN EXPERIMENTAL RESULTS AND FINITE ELEMENT RESULTS

5.1 Ultimate Load Capacity

Experimental and finite element results of column carrying capacity for eight models were summarized in Table 7. There is a good agreement between the experimental and FE results with a variation of 15%.

Table 7: Comparison between experimental and FE results.

Model	EXP.		FEM.		Accuracy $P_{u\,FEM}/P_{u\,EXP}$
	V.Disp, mm	P_u, kN	V.Disp, mm	P_u, kN	
C1	3.464	1030.37	3.34745	981.387	95%
C2	4.1535	1396.89	4.37663	1440.39	103%
C3	5.448	1475.8	5.25569	1533.11	104%
C4	5.685	1603.87	5.75212	1595.86	100%
C5	3.45999	704.75	3.22475	788.154	112%
C6	3.5789748	960.22	4.1992	1100.26	115%
C7	3.20832	1409.96	3.42516	1486.37	105%
C8	6.80448	1998.07	6.84395	2115.49	106%

5.2 Vertical Load versus Vertical Displacement Relationship

Figures 21 to 28 displayed the relationship between axial load and corresponding vertical displacement for both experimental and F.E column models.

Figure 21 to 28 provides a clear view that the relationship between load versus displacement in experimental and finite elements modeling is the same. Therefore, the generated model can be used to study other relevant factors.

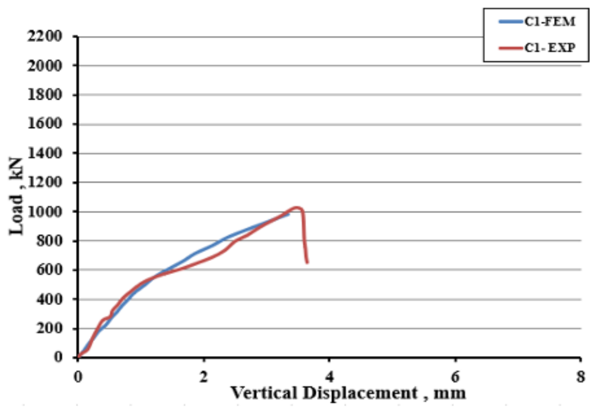


Figure 21: Load–displacement curve of C1.

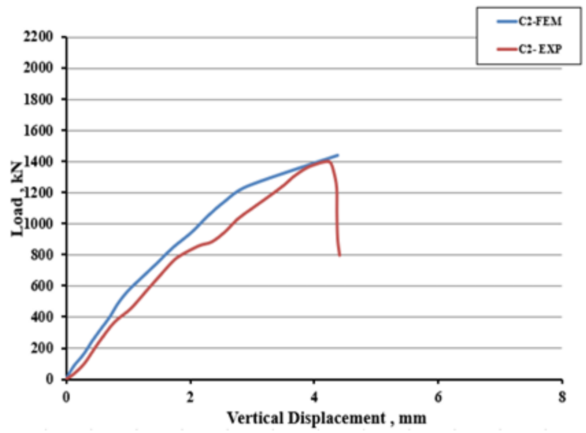


Figure 22: Load–displacement curve of C2.

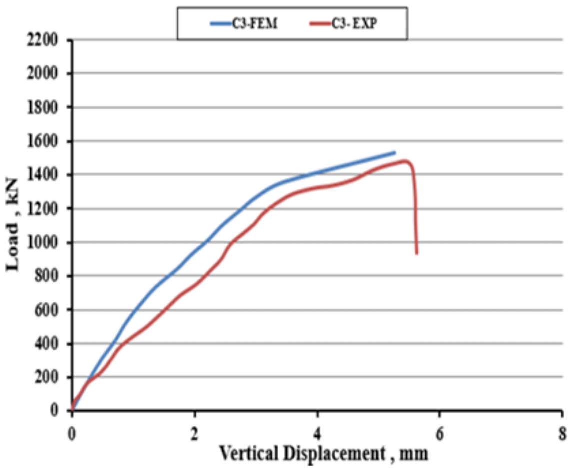


Figure 23: Load–displacement curve of C3.

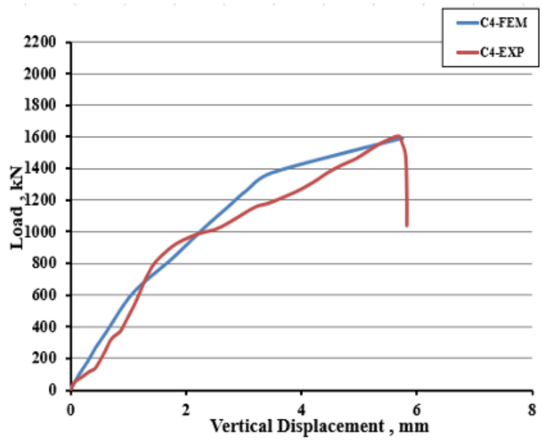


Figure 24: Load–displacement curve of C4.

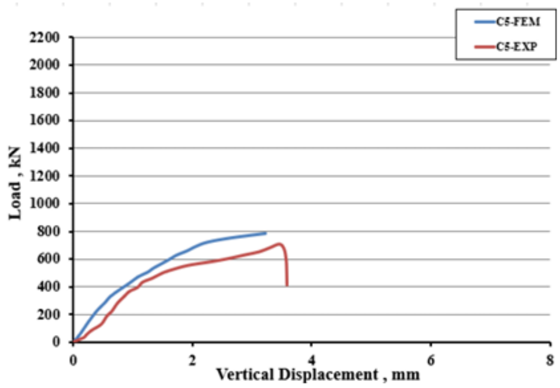


Figure 25: Load–displacement curve of C5.

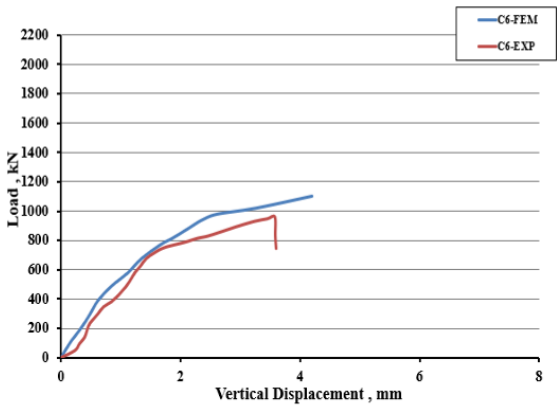


Figure 26: Load–displacement curve of C6.

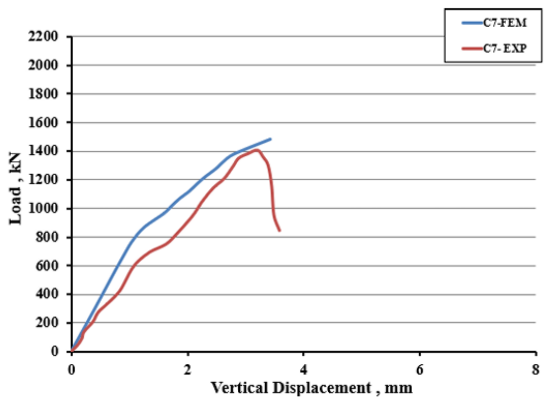


Figure 27: Load–displacement curve of C7.

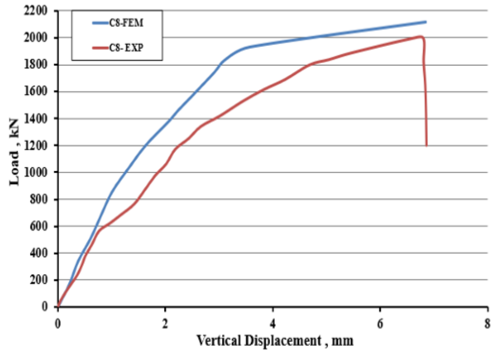
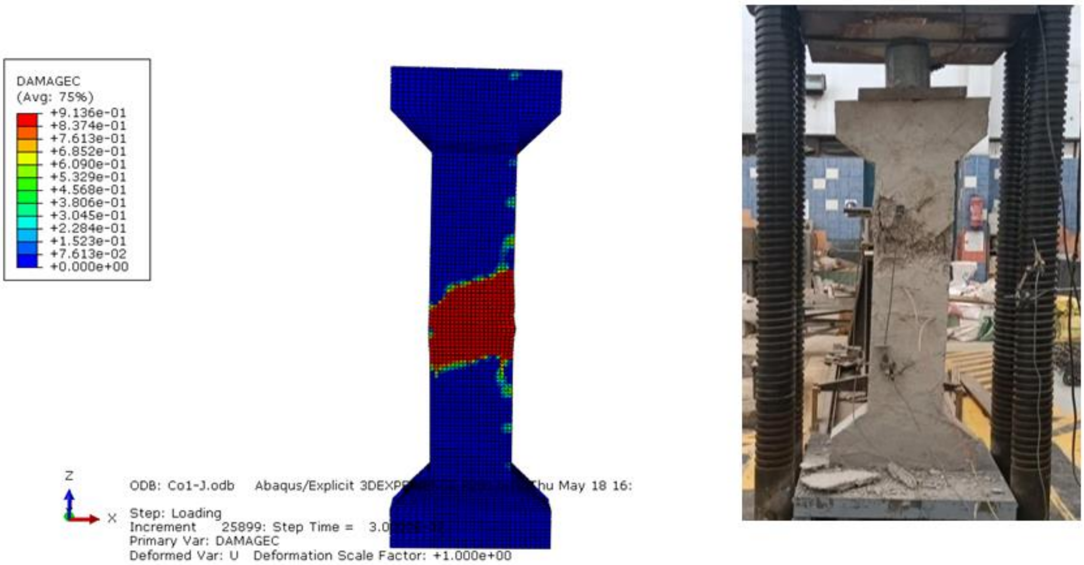


Figure 28: Load–displacement curve of C8.

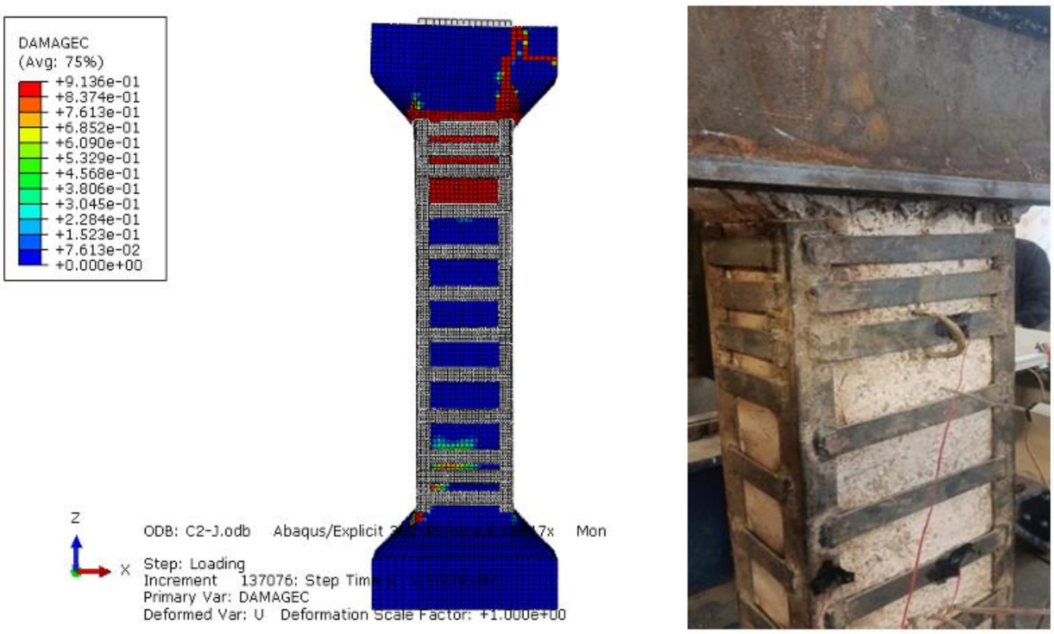
5.3 Failure Pattern (creak pattern)

Figures 29 to 32 represent a comparison between failure modes and crack patterns of column samples as tested and the corresponding FE models, in which the same modes of failure happened.



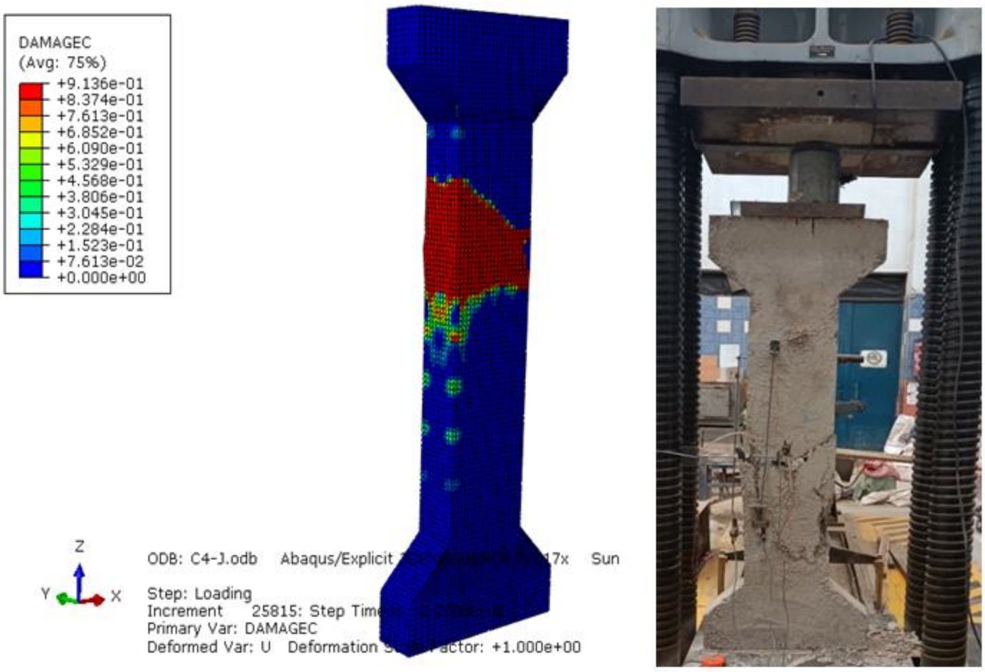
(a) Compression damage of FE model. **(b)** Crack pattern of tested specimen.

Figure 29 Comparison between the crack pattern of the experimental model and its corresponding FE model of C1.



(a) Compression damage of FE model. **(b)** Crack pattern of tested specimen.

Figure 30: Comparison between the crack pattern of the experimental model and its corresponding FE model of C2.



(a) Compression damage of FE model. **(b)** Crack pattern of tested specimen.

Figure 32: Comparison between the crack pattern of the experimental model and its corresponding FE model of C5.

6. GOVERNING EQUATIONS AND DESIGN PROCEDURE

The subsequent procedure is grounded upon empirical equations and established relationships pertaining to the resistance of steel and concrete sections. The analysis commences with the recognition that the un-strengthened column possesses a predetermined ultimate load capacity, calculated as follows:

$$\mathbf{P_u = factor * \delta_C * A_C + factor * \delta_s * As}$$

Where,
Pu = ultimate column load capacity
δC = concrete compressive strength
δs = steel strength
δC = concrete compressive strength
Ac = column section area
As = steel section area

The effect of internal steel can be assumed as a percentage of the total column's capacity. So, the factor of safety of concrete section capacity was adjusted to be $\mathbf{0.75*\delta_C*A_C}$. This value is equal to the minimum factor of safety gained from laboratory test results of Sample C5.

Also, it can be assumed that the external confining pressure converted to concrete strength improves column load bearing capacity.

$$\mathbf{P_t = (0.75*\delta_C*A_C + \delta_L*X*A_C)} \quad \mathbf{(1)}$$

Where,
Pt = target ultimate retrofitted column load
δC = Concrete compressive strength
X = constant factor
Ac = Column section area
δL = required applied lateral confining pressure

Equation 1 was solved for the required lateral pressure δ_L which is to be applied to the column by the proposed procedure. This pressure is attained by applying torque on the threaded rods surrounding the column resulting in tension in the rod. To calculate the required lateral pressure δ_L the following equations can be used.

$$\mathbf{Mt = 0.2*Tr*db} \quad \mathbf{(2)}$$

Where,
Mt = torque applied on threaded rods
db = rod diameter
Tr = tension on the threaded rods

Table 8 shows the results of substitution in Eq 2.

Table 8: Substitution in Equation 2.

M_t N.m	M_t N.mm	T_r N	d_b mm
100	100000	25000	20
150	150000	37500	20
200	200000	50000	20

$$\mathbf{Tr = \delta L * L/2 * dp} \quad \mathbf{(3)}$$

Where,
Tr = tension on threaded rods
db = Maximum distance between threaded rods
L = column Length
δL= lateral pressure generated at the bottom of the corners

Table 9 shows the results of substitution in Eq 3.

Table 9: Substitution in Equation 3.

T_r N	L/2 mm	d_p mm	δ_L N/mm^2
25000	150	350	0.476195
37500	150	350	0.714285
50000	150	350	0.95238

To calculate the required constant factor (X) it was assumed through the following equations.

For ordinary concrete strength

$$X= 25* M* L/b * (Lh/Lv) \quad (4)$$

For high-strength concrete

$$X= 10* L/b *(L_h/L_v)$$

Where,

M = reduction factor

(M=1 in case of internal ,steel reinforcement,

M= 0.6 in case of without internal steel reinforcement)

L = column length

b = column width

L_v = distance between center line of horizontal plate and the start of next plate

L_h = width of the horizontal plate

Table 10 shows the results of substitution in Eq 4,

Table 10: Substitution in Equation 4.

Column	L/b	M	L_h/L_v	X
C1				
C2	2	1	0.47	23.5
C3	2	1	0.47	23.5
C4	2	1	0.61	30.5
C5				
C6	2	1	0.47	23.5
C7				
C8	2	-	0.47	9.4

Table 11 shows the results of substitution in Eq 1 which presented adequate results with the experimental columns capacity with a range of ±20%.

Table 11: Substitution in Equation 1.

Column	δ_C N/mm^2	δ_L N/mm^2	X	A_C mm^2	P_t from Equation N	P_t from tests N	Percentage of Equation Validity
C1							
C2	20	0.476	23.5	45000	1178370	1395000	118%
C3	20	0.714	23.5	45000	1430055	1475000	103%
C4	20	0.952	30.5	45000	1981620	1604000	81%
C5							
C6	20	0.476	23.5	45000	1178370	960000	81%
C7							
C8	50	0.952	9.4	45000	2090196	1998000	96%

7. CONCLUSION

The present investigation unequivocally demonstrates the efficacy of strengthening reinforced concrete columns through the application of steel jackets welded under external pressure. This technique has been proven to significantly augment the ultimate load-bearing capacity, enhance strength, ductility, and stiffness. The following recapitulates the key conclusions derived from this study:

1. For columns constructed with ordinary concrete (Fcu=20 N/mm^2), the application of the proposed strengthening technique resulted in a substantial augmentation of the ultimate load-bearing capacity, ranging from approximately 36% to 56%. Concomitantly, a notable enhancement in the column's ductility was observed.
2. In the context of columns fabricated from high-strength concrete (Fcu=50 N/mm^2), the strengthening technique yielded an impressive increase in ultimate load capacity, reaching 42%. Moreover, the column's ductility was significantly improved.
3. Columns subjected to eccentric loading, the strengthened columns demonstrated a notable increase in ultimate load capacity, approximating 36%. However, the application of confining pressure in these cases did not exert a significant influence on the column's ductility.
4. The empirical findings revealed a direct correlation between the reduction in the distance between horizontal battens and an augmentation of the column's ultimate load capacity. Similarly, an increase in external confining pressure was observed to positively impact the column's ultimate load capacity.
5. The developed finite element model serves as a valuable tool for analyzing columns and predicting their capacity following the application of strengthening techniques. The model's accuracy was validated through its congruence with experimental results, demonstrating a difference of within ±15%.
6. The equations proposed within this study can be effectively employed to calculate the column's capacity after strengthening, as their predictive capabilities align well with the experimental outcomes, exhibiting a difference of within ±20%.
7. It was observed that a localized failure occurred at the interface between the column head and the

strengthened section, attributable to the concentration of stresses within that region. To mitigate this issue and enhance the column's load-bearing capacity, it is imperative to strengthen the contact point between the column and the concrete slab or beam in all structures.

REFERENCES

[1] Abou EL-Enin, A., “Strengthening of Eccentrically Loaded Columns”, Bulletin of Ain Shams University, 1985.

[2] Soliman, M, I., “Repair of Distressed Reinforced concrete Columns”, Technical Report, Ain Shams University, 1988.

[3] Wang, Jizhong, Shiwei Lu, and Junlong Yang. "Behavior of eccentrically loaded rectangular RC columns wrapped with CFRP jackets under different preloading levels." Journal of Building Engineering 34 (2021): 101943.

[4] Mahfouz I., Sarkani S., and Rizk T., “Repairing and Retrofitting of Circular and Square Reinforced Concrete Columns Using Fiber Reinforced Plastic (FRP)”, The Arabic Conference for Repair and Rehabilitation of Structures, pp. 437-451, 1998.

[5] Amin, N.M., and Ahmad, N.A., “Performance of reinforced concrete columns strengthened with carbon fiber reinforced polymer (CFRP)-a case study of an abandoned school project”, IOP Conference Series: Materials Science and Engineering, Vol. 933, No. 1, p. 012014, IOP Publishing, 2020.

[6] Jirsa “Rehabilitation of shear critical concrete columns by use of rectangular steel jackets”, in ACI Structural Journal, Jan 1999.

[7] Lisantono, A., Budiman, A. and Sidauruk, P.H.B., “Experimental investigation of reinforced concrete column embedded with the angle steel shapes”, Procedia Engineering, 125, pp.879-884, 2015.

[8] EL Tuhami A. A. and Saker T. A., “An Innovative Technique for Strengthening Reinforced Concrete Elements Using Mechanical External Prestressing”, Proceeding of the Arab Conference for Structural Engineering, Cairo, pp. 423-433, 2003.

[9] A. Al-Tuhami & T. A. Sakr “Retrofit of reinforced concrete columns using global lateral external pressure” Composites in Constructions, Bruno et al (eds), 2003, Editoriale Bios

[10] Abaqus Documentation User’s Guide: ABAQUS User’s Guide: Dassault Systemes, Simulia Corp, Providence, 2021.

[11] Hu, Hsuam-Teh, and William C. Schnobrich. “Constitutive Modeling of Concrete by Using Nonassociated Plasticity”, Journal of Materials in Civil Engineering 1, no. 4, P.P:199-216, 19.