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RESEARCH PAPER

Behavior of High Strength Concrete Columns Intersected by Normal Strength Concrete Beams

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A B S T R A C T:

Under axial compression stress, ten column specimens were investigated, one of which was an isolated column specimen, while the others were beam-column specimens with varying concrete strengths intersected with normal strength concrete. The column axial strength and behaviour were studied with the following parameters: different ratios of column concrete strength to beam concrete strength (f'_{cc}/f'_{cb}) (1, 1.58 and 1.89), different confinement ratios (ties) at the column location that intersected with the beam (0, 0.0048, and 0.0098), and the effects of confining beam-column joint by beams on two sides of the column. The experimental results demonstrated that the strength of the inner beam-column joint depends on the confining of the beam-column joint by ties. The ties change the failure location from the beam-column joint to the upper and lower column. The existence of ties overcomes the problem of normal concrete strength between high strength concrete (HSC). The proposed method is compared with ACI and CSA equations to show the efficiency of the models. ACI model gave significantly overestimated results.

KEY WORDS: high strength concrete, interior beam, isolated column, and beam-column joint.
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1.INTRODUCTION :

High-strength concrete (HSC) is becoming increasingly frequently used in columns; this tendency in engineering dates back to earlier times. The reason is to decrease the section size of columns at lower stories of medium to high rise buildings. This provides more clear spaces among the columns an efficient space on each floor, especially at lower stories (i.e., ground floor or basement floors). The axial load at the joint of the column, at the lower story, gives a very high value relative to the existence of bending or shear forces. Therefore, the effect of axial stress is dominant in the existing stresses The use of HSC is expensive; for example, Kurdistan region-Iraq, for a similar situation of casting, the concrete with cylinder compressive strength reaches 100 MPa may be reached about five times than normal strength concrete (NSC), 30 MPa. Normally, the volume of concrete of the columns in each story reaches about 10-20% of the volume of concrete of floor beams and slabs. This means that the column size is a small quantity of concrete material relative to the skeleton frame of the specified floor. Therefore practically, column cast with HSC and beam-slab floor cast with NSC. However, the placement of the NSC slab-beam floor between HSC concrete columns may lead to failure in the HSC columns or the NSC slab-beam. Therefore, this weak location in between the columns may limit the capacity of HSC columns. When is normal strength concrete present in the joint, and to what extent may the strength of HSC be sufficient?

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The ACI code (ACI Committee 318, 2019) mandates the use of puddled HSC in the slab, vertical dowels and spirals through the joint, or effective concrete strength is used for the concrete in the joint when the column concrete strength, f'_{cc}, exceeds 1.4 times the slab concrete strength, f'_{cs}. The effective concrete compressive strength, f'_{ce}, is given for columns laterally supported on four sides by slabs.

$$f'_{\mathrm{ce}} = 0.35 f'_{\mathrm{cs}} + 0.75 f'_{\mathrm{cc}} \quad (1)$$

When using Equation 1, the design's $(f'_{\mathrm{cc}}/f'_{\mathrm{cb}})$ ratio must not be more than 2.5.

The CSA standard (CSA, 2004) specifies that when the column concrete's reported compressive strength is greater than the slab's, the column load transmission through floor systems is investigated. For an interior column, the effective concrete compressive strength is given as

$$f'_{\mathrm{ce}} = 1.05 f'_{\mathrm{cs}} + 0.25 f'_{\mathrm{cc}} \leqslant f'_{\mathrm{cc}} \quad (2)$$

A few researchers studied this problem, common in the construction of high-rise buildings (cast of upper and lower columns with HSC and slab-beam with NSC).

Bianchini et al., (1960) tested slab-column specimens and isolated-sandwich columns. The tested samples comprised slab columns (interior, edge, corner) and isolated-sandwich columns. The authors showed how the ratio of the column concrete strength to the floor concrete strength (f'_{cc}/f'_{cb}) and the degree of confinement from the surrounding slab affect the axial capacity of the slab-column connections. Additionally, they recommended restricting the ratio of (f'_{cc}/f'_{cb}) in the design expressions for the effective concrete strength to 1.4. Based on the findings of Bianchini et al., the ACI Code equation (ACI 318-89) was developed.

Gamble and Klinar, (1991); Kayani, (1992); Ospina and Alexander, (1998) tested slab-column specimens (interior, edge) and isolated columns to broaden the range of column concrete strength to slab concrete strength ratio and the range of column concrete strengths. They concluded that when the concrete strength of the column is extremely high, the provisions of the 1989 ACI Code (ACI 318-89) overestimated the effective concrete strength. Additionally, they provided design equations for the effective column concrete strength for the interior and edge slab-column joints.

McHarg et al. (2000) experimentally studied the effects of confinement from the surrounding slab, the concentration of slab flexural reinforcement in the column region and the placement of steel fiber-reinforced concrete (SFRC) in the slab near the column. They concluded that the effective axial compressive strength and ductility increased by surrounding slab confinement of interior columns. They also demonstrated the benefits of placing SFRC slab concrete close to the column and evenly spaced slab top steel bars.

Shah et al. (2005); and Shah and Ribakov, (2005) found that the strength of an interior slab-column joint is greatly influenced by the surrounding slab's ability to restrain the joint, the joint aspect ratio (h/c), the ratio of slab reinforcement, the intensity of the slab loading, and the concrete strengths of the column and slab.

Shin et al. (2017) discovered that the strength of an interior slab-column joint is significantly affected by the surrounding slab's capacity to restrain the joint, the intensity of the slab loading, and the concrete strengths of the column and slab. The researchers found that the ACI Code (2014) approach provides overestimates predictions of interior slab-column specimens when the slab is loaded and the ratio of (f'_{cc}/f'_{cs}) ranges from 1.40 to 3.50.

Lee and Yoon (2010) did trials, and Stanisław Urban and Gołdyn (2015) cast a portion of slab-beam, under the top and above the bottom column, with the same HSC type of the column. In addition, the remaining portion of the slab-beam, of the same floor, is cast with NSC. Nevertheless, this procedure of casting two types of concrete on the same floor and at the same time is difficult in practice, during the casting and leveling of concrete for a slab-beam, a portion of the floor, around the column.

Most of the previous literature examined slab-column specimens. As it is known, slab reinforcement is different from beam reinforcement. Slab not confined with transverse reinforcement, while beam wrapped laterally with stirrups. These stirrups induced more strength for the core concrete than if not used stirrups. Although, in addition, all the studies used two types of concrete strength, they also recommended doing more experimental studies to take new variables into account and confirm the existing data. On the other hand, all previous studies on this subject do not take confinement effect of the joint by transverse reinforcement (ties of the column in the joint). Depending on these gaps in the literature review, this study involves the two main variables, the first is the degree of joint confinement using ties and the second is having three different concrete strengths, NSC, medium strength concrete (MSC) and HSC of column intersected by NSC floor beams.

2.EXPERIMENTAL PROGRAM:

The experimental program consists of ten beam-column joints, one of these specimens is cast without beams (isolated column) that is used as a control column; the rest are plane interior beam-column specimens, to show the effect of different strengths of the concrete column on intersected beams with NSC. The beam-column specimens consist of reinforced concrete stub columns, having cross-sectional dimensions of 150 x 150 mm and extending 500 mm above and below the 150 mm depth of the beam. As a result, the depth of beam to column dimension, h/c, has a constant aspect ratio of 1.0. The beam of all specimens was cast with NSC. The target column concrete strength at the top and bottom of the beam were variables taken as NSC, MSC and HSC. Samples with different concrete strength were cast in two stages: the lower and upper stub column, and the intersect beam. Different concrete strengths were separated from one another by thin wood before being cast in the same shape. After casting, the thin wood was removed, when the concrete was in the fresh state seen in **Figure 2**. All the beam-column specimens were cast from the same batch concrete (each with its strength) at the same period to get the same properties of concrete.

The forms were removed two days after casting and then cured at ambient temperature up to 56 days. Another parameter was studied, the number of ties (0,2 and 4), (i.e.; 0, 0.0048 and 0.0098) confined the concrete in the portion of the column was cast with NSC. The ready concrete mix used for this study was from the 77 company. **Figure 1** show the longitudinal column reinforcement as 4-Ø10 mm and the ties reinforcement as 5- Ø6 mm @ 500 mm. The reinforcement *fy*, *fu*, elongation and area properties were 310 MPa, 414 MPa, 33%, 66.5 mm^2: 608 MPa, 675 MPa, 5%, 27.5 mm^2 for Ø10 mm and Ø6 mm, respectively, as shown in **Table 1**. All beam-column specimens have the same arrangement of longitudinal bars, transverse reinforcement and 20 clear concrete covers were used. Group two (G2) consists of beam-column specimens without ties in the beam-column intersection joints. The top and bottom columns of each specimen are cast with cylinder concrete compressive strength (f_c') of NSC, MSC, and HSC for BC2, BC3, and BC4, respectively. The overall height of the column is 1150 mm. In all ten specimens have been cast with cylinder compressive strength (f_c') of NSC. The used ratios of column concrete strength to beam concrete strength (f_{cc}'/f_{cb}') were 1, 1.58, and 1.89, as shown in **Table-2**. The other two groups (G3 and G4) were cast with the same properties as G2, but G3 has two ties of Ø6 mm to confine the concrete at the intersection joint between the beams and columns. G4 has four ties- Ø6 mm to confine the concrete. Further, specimen G1 which has the same properties as BC6, except that the specimen consists of isolated columns without beams.

A universal computerized testing device with a maximum capacity of 2,500 kN was used to load the test specimens. The loading was done by load increment at a rate of 1.1 kN/sec. The end parts of each column were covered with 100 mm-high carbon fiber reinforced polymer CFRP sheets in order to prevent early failure at the extremities of the column (where they meet the apparatus). In addition, a capping steel plate box of 5 mm thick plate was used, as shown in **Figure 4**. On the top and bottom of each column specimen, an 8 mm thick rubber capping was employed to guarantee distribution loading from the machine to the column surface.

According to **Figure 3 and 4**, all columns were subjected to pure axial (concentric) force through two rollers at the top and bottom of the column. The inclusion of a roller was to create pin-to-pin contacts between the specimen is one of the test setup's key components. The force is transferred to the column at the top and bottom through two rollers fixed above the top column and under the bottom column. The roller transfers the force to pure axial through the column, which is similar to the actual column at the lower story of the high-rise building.

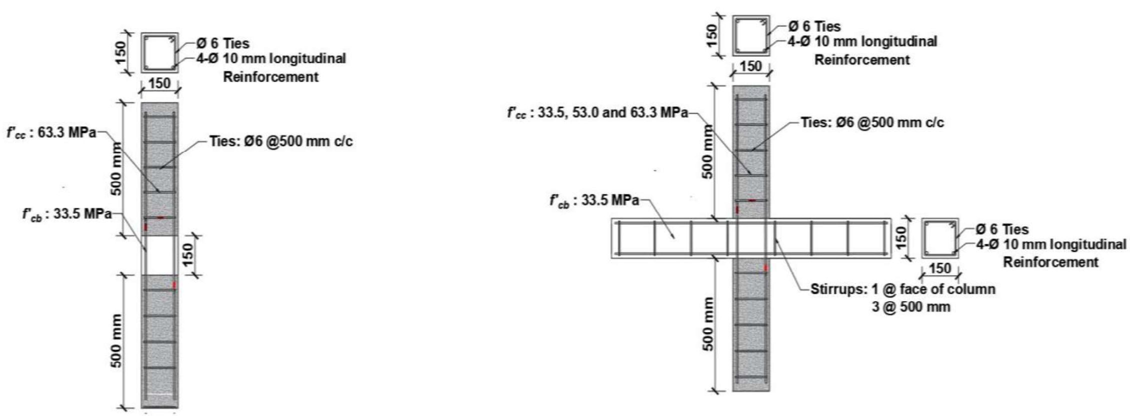
Table-1 Properties of steel bars

No.	Nominal Diameter (mm)	Diameters (mm)	Area (mm^2)	f_y (MPa)	f_u (MPa)	Elongation (%)
1	6	5.9	27.5	608	675	5
2	10	9.2	66.5	310	414	33

Table 2 Detail of beam-column specimens, with studied program.

Group	specimen	Target concrete strength			Ties in joint	ρ_t *
		f'_{cb}	f'_{cc}	f'_{cc}/f'_{cb}		
G1	IC1	NSC	HSC	1.89	0	0
G2	BC2	NSC	NSC	1	0	0
	BC3		MSC	1.58		
	BC4		HSC	1.89		
G4	BC5	NSC	NSC	1	2	0.0048
	BC6		MSC	1.58		
	BC7		HSC	1.89		
G4	BC8	NSC	NSC	1	4	0.0098
	BC9		MSC	1.58		
	BC10		HSC	1.89		

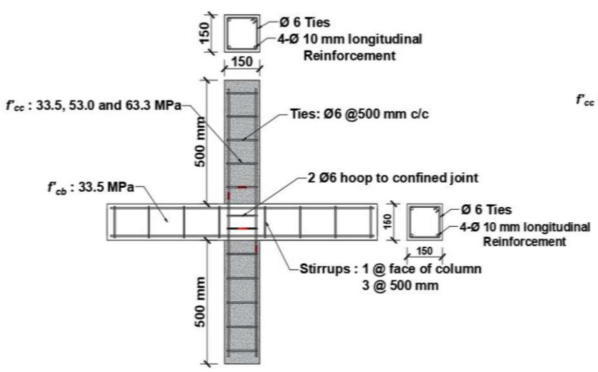
* Transverse steel ratio in the joint



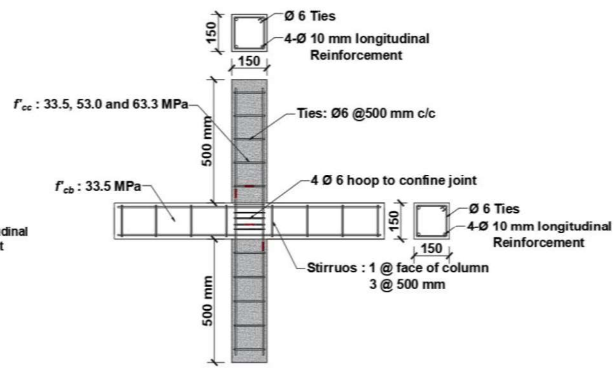
a-isolated column, IC1 **b**- Specimen without ties, G2

Figure 1 Detail of the beam-column joint with reinforcements.

Figure 1 continued



c- Specimen with two ties, G3



d- Specimen with four ties, G4

Figure 2 Beam-column specimen connection casting configuration

Figure 3 Testing machine

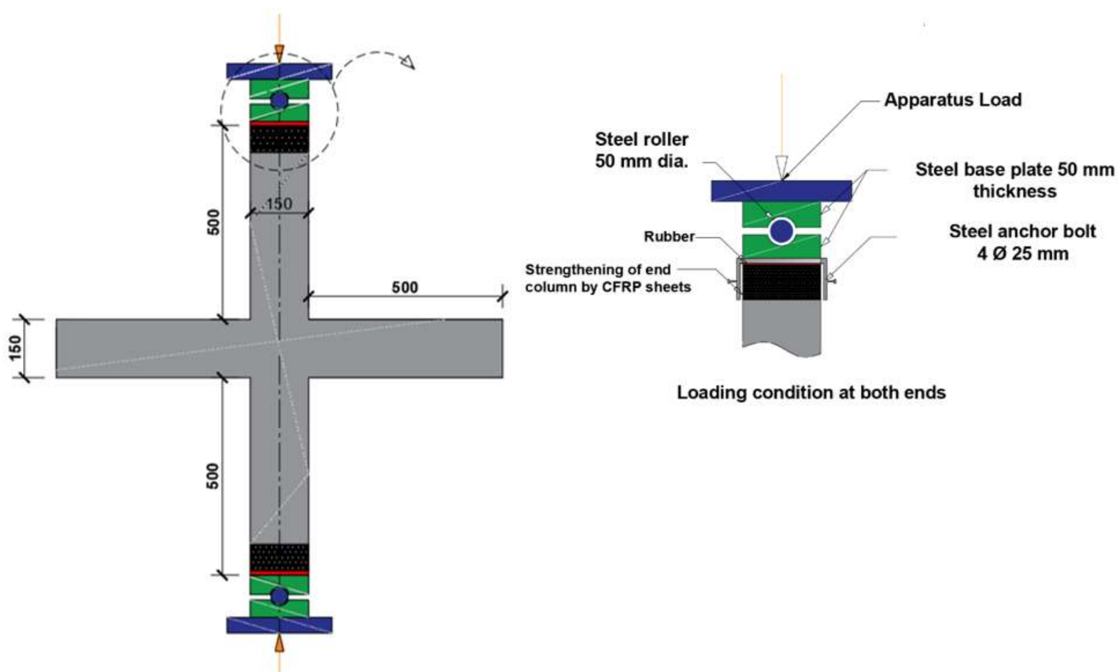


Figure 4 Sketch of loading condition at both ends of beam-column specimen

3.EXPERIMENTAL RESULTS AND DISCUSSION:

This section presents the test results for the isolated concrete columns and beam-columns specimens. All specimens reached the maximum carrying capacity. The loading failure for each specimen is showed in **Table 3**. In the following sections, the behaviour and failure modes of the isolated concrete column and beam-column specimen, the effect of different concrete strength for beam and column on the maximum carrying capacity, and the effect of the ties inside beam-column joint on the location of failure are discussed.

Table 3 Summary of test results

Group	specimens	f'_{cc} (MPa)	f'_{cc} (MPa)	f'_{cc}/f'_{cb}	Ties in joint	P $_{test}$ (kN)	% Effect of Concrete strength	% Effect of ties	Location of failure
G1	IC1	33.5	63.3	1.89	0	874		0	JF
G2	BC2		33.5	1.00	0	635	0	0	CF
	BC3		53.0	1.58		750	18.1	0	JF
	BC4		63.3	1.89		930	46.6	0	JF
G3	BC5		33.5	1.00	0.0048	683	0	7.6	CF
	BC6		53.0	1.58		1090	59.6	45.3	CJF
	BC7		63.3	1.89		998	46.1	7.3	CJF
G4	BC8		33.5	1.00	0.0098	704	0	10.9	CF
	BC9		53.0	1.58		1048	48.9	39.7	CF
	BC10		63.3	1.89		1140	61.9	22.6	CF

CF: Column failure, failure occurred at top or bottom column.
JF: Joint failure, failure occurred at intersection beam-column joint.
CJF: Column joint failure, failure occurred at the horizontal line at intersection of the joint

3.1Cracks, Cracks Propagation and Method of Failure:

This section takes the influence of (f'_{cc}/f'_{cb}), confining joint by reinforced ties, and the existence of beam around the joint on the behaviour and mode of failure.

All beam-column specimens generally exhibited no beam, column, or joint cracks and exhibited evident deflection until a significant proportion of loading reached roughly 95% of the specimen's failure load. This means that abruptly failed with little or no warning, sudden failure. This is one of the very dangerous failures of building and is not preferred by the engineers, especially the column that are subjected to pure axial load like columns at the ground or the basement, i.e., lower columns of a high-rise building. In addition, the final failure was due to the crushing of concrete at the intersection of the beam-column region, near this joint, above or below the joint. The failure accompanies by the buckling of longitudinal steel bars of the column

3.2Effect of Confining Joint by Ties on Location of Failure:

The column material strength changed in G2 specimens without using ties at the intersection joint between the beam and the column, from NSC to MSC to HSC. The type of failure location changed from column failure (CF) to joint failure (JF). The reason for the column failure in BC2 is related to that portion of the beam beside the column (the joint), making a wrapping for concrete, then the failure transferred to the column. In addition, all the materials of beams and columns have the same strength. In BC3 and BC4, with MSC and HSC columns, the concrete at the column is stronger than the joint, therefore the failure occurred at the joint (JF), despite the existence of beams confinement beside the joint, this means that this confinement is not enough to prevent joint failure as shown in **Table 3** and **Figure 5 (b, c and d)**.

In G3, the joint of the specimens was made from NSC and confined with two ties (ρ_t = 0.0048) these two ties have an important role in transferring the failure location from the joint (NSC) to the columns part MSC that attach to the NSC joint, so the final failure occurred done in the column and then transferred to the joint. This means that the ties made the joint stronger than G1 (without ties, i.e., without confinement), as shown in **Figure 5 (e, f, and g)**.

In G4, the joint of the specimens (NSC), was confined with four steel ties (ρ_t = 0.0098).

These ties significantly transferred the failure position from the joint (NSC) to the column (HSC). Also, the same effect of the ties happened in the specimens with columns made from NSC (BC8), MSC (BC9) or HSC (BC10) material. The effect is more pronounced (than two ties, G3) to transfer the failure position to the column part adjacent to the intersection joint, as shown in **Figure 5 (h, i, and j).**

In general, in joints without lateral steel confinement, the failure occurred in the joint region. Nevertheless, with this type of confinement (ties) in the joint, the joint made from NSC transferred to be stronger, then the final failure transferred from joint location the column (that made from HSC). This means that if the column is made from MSC or HSC, the weak cast point (NSC floor slab-beam) can be overcome by confining the joint with steel ties (ρ_t = 0.0048) if the ratio of the material strength of the column to the floor (f'_{cc}/f'_{cb}) increased from 1 to 1.58 and 1.89.

The authors intend to use column material to achieve approximately 100 MPa. The reason of use concrete column strength of 63.3 MPa as a maximum value is due to the limitation of the universal testing machine available in the laboratory

a-IC1

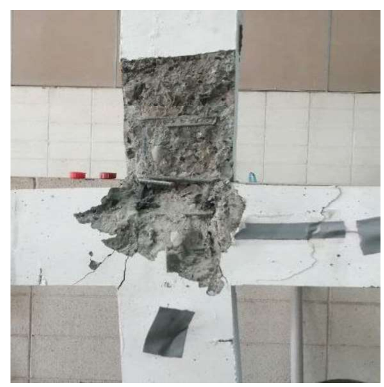
b-BC2

c-BC3

d-BC4

e-BC5

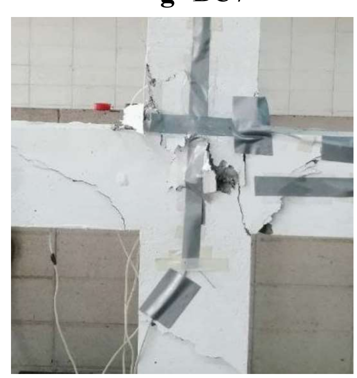
f-BC6



g- BC7

h-BC8

i-BC9



j-BC10

Figure 5 Failure mode of specimens

3.3Maximum Carrying Capacities:

3.3.1Effects of the Column Strength Grade

The specimens IC1 and BC4 consist of HSC columns and NSC beams (if they exist) and an NSC joint, while IC1 exhibited a 6.1% decrease in axial strength relative to specimen BC4. This shows that the intersection joint of the specimen of BC4 has to confine the effect of the joint from the surrounding beam. A similar observation done by Bianchini et al. (1960), the effective concrete joint strength improves as the number of sides of the specimen being constrained by the overhanging floor concrete grows, even for corner columns, using slab-column specimens with unloaded slabs.

Table 3 and **Figure 7** show the effect of different concrete strengths of the column intersected by the NSC beam on the maximum carrying capacity of the specimen. The specimens without stirrups BC3 (MSC column) and BC4 (HSC column) gave an ultimate load more than specimens BC2 (NSC column) by 18.1% and 46.6%, while the material strength increased 58% and 89%, respectively. This means that increasing the column's material strength is not accompanied by a parallel or reasonable equivalent increase in the strength of the specimens. A similar behaviour was observed in specimens with two ties of the joint reached to 59.6% and 46.1%, and four ties of the joint were 48.9% and 61.9%, respectively, with an increase in material strength of the column at 58% and 89%.

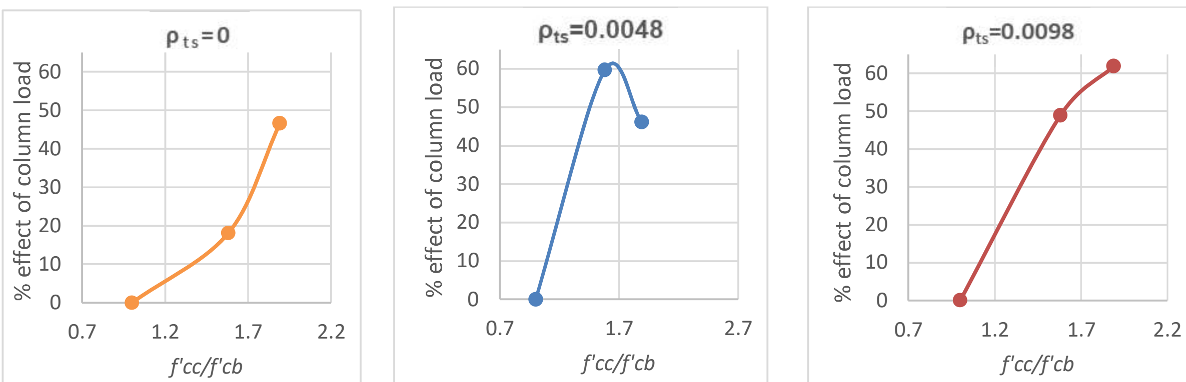


Figure 7 Relation between $f'_{cc}/f'_{cb}$with percentage effect of G2, G3 and G4, respectively.

3.3.2Effect of the Joint Confinement:

The effect of ties on the specimen's carry capacity is shown in a separate column in **Table 3**. The specimens BC5 (two ties) and BC8 (four ties) have increased in specimen strength reach to 7.6% and 10.9%, respectively, relative to BC2 (without ties). The column of all three specimens, BC2, BC5, and BC8, have the same material strength, as shown in **Figure 6**.

This specimens with column concrete strength of 53.0 MPa (BC3, BC6, and BC9), increasing the number of ties (0 to 2, or 4), the specimens (BC6 and BC9) exhibited 45.3% and 39.7%, respectively, in column strength relative to BC3. The practice shows that the specimen with two ties is more efficient than a joint reinforced with four ties, as shown in **Figure 6**.

The specimens BC4, BC7, and BC10 have the same concrete strength of column 63.3 MPa, with increasing the ties from 0 to 2, or 4 got 7.3% and 22.6%, respectively, relative to BC4.

In general, as the confinement ratio of a joint increases from 0 to 0.0048 or 0.0098, specimens with a *(f'_{cc}/f'_{cb})* ratio of 1.58 are more efficient than those with 1.89 and 1.0 ratios.

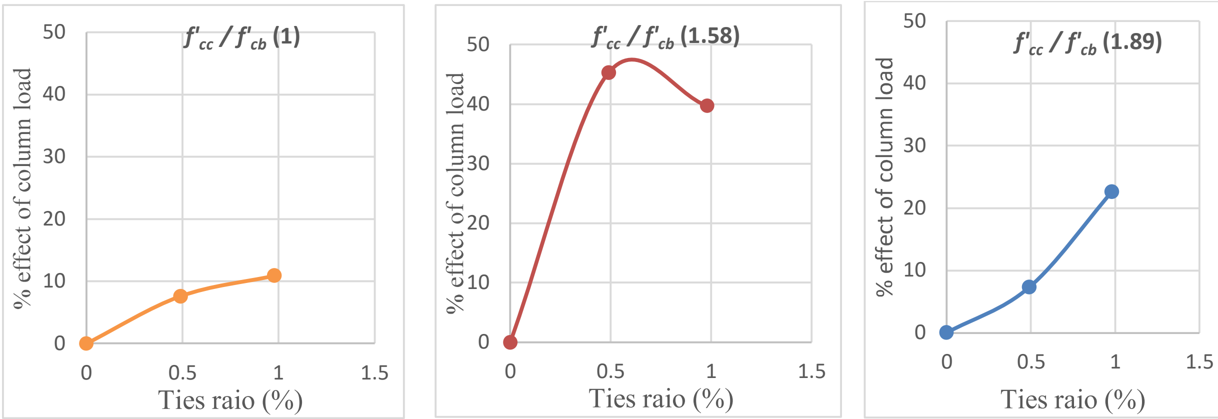


Figure 6 Relation between ties ratios with percentage effect of column load.

3.4Theoretical Analysis:

All the existing theoretical models on this subject consider specimens constructed from the HSC column joined with NSC flat plate slabs. The study focuses on specimens made from the HSC column with NSC beams. In this study, the design equations of ACI code Eq. (1), and Eq. (4), and CSA code as in Eq. (2), Eq. (5), and Eq. (6), were compared with a proposal model Eq (3), Eq (4), Eq (6), and Eq (7). These equations were proposed to be used in columns subjected to axial loads. In addition, the ACI and CSA equations were modified to incorporate the effect of different strengths of the HSC column joined with NSC beams, as shown in **Table 4**.

The proposed equation considers another variable not taken in ACI and CSA codes. The confinement of the joint by ties takes as a new parameter in the following proposed equation:

$$f'_{ce} = f'_{cc} \quad \text{if} \quad f'_{cc}/f'_{cb} <= 1 \qquad \ldots\ldots\ldots\ldots\ldots\ldots(3)$$

$$f'_{ce} = 0.8 f'_{cs} + 0.45 f'_{cc} \quad \text{if} \quad f'_{cc}/f'_{cb} > 1 \qquad \ldots\ldots\ldots\ldots(4)$$

In addition, the ACI code equation of axial load is:

$$P_n = 0.85.f'_{ce}.\left(A_g - A_{st}\right) + A_{st}.f_y \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Tthe CSA Code equations of axial load are:

$$P_n = \alpha.f'_{ce}.\left(A_g - A_{st}\right) + A_{st}.f_y \ldots\ldots\ldots\ldots\ldots\ldots(5)$$

$$\alpha = 0.85 - 0.0015.f'_{cc} > 0.67 \ldots\ldots\ldots\ldots\ldots\ldots(6)$$

The proposal equations of axial load are:

$$P_n = \alpha.f'_{ce}.\left(A_g - A_{st}\right) + A_{st}.f_y + 30000.f_{yt}.\rho_t \ldots\ldots\ldots\ldots(7)$$

$$\alpha = 0.75 - 0.0015.f'_{cc} > 0.65 \ldots\ldots\ldots\ldots\ldots\ldots(8)$$

The values of excremental carrying capacity to theoretical carrying capacity of the specimens (P_{EXP}/P_{Theo}) were used to assess ACI, CSA, and the proposal equations, as seen in **Table 4**. The values of mean, standard deviation (SD), coefficient of variation (COV), and the number of samples that have P_{EXP} more than P_{Theo} taken as a safe value (more than one) was used as a comparative point among these methods. ACI code equations show overestimated values of

(P_{EXP}/P_{Theo}) reached 0.88 as a mean value, 13.2% for COV and one sample has a value of more than 1, while the CSA code gave 1.03 for the mean, 12% for COV and 6 samples have values more than 1.

The proposed equations take the effect of steel ties confinement into account that not taken in previous studies. The application of the equations (3), (4), (6), and (7), resulted in better value than the previous mention methods, as the ratios of *(P_{EXP}/P_{Theo})*, have a 1.07 as the mean value, 9% as COV with 8 specimens being at the safer side (P_{EXP} larger than P_{Theo}), as shown in **Figure 8**.

Table 4 Comparative among ACI, CSA, and proposal models.

			ACI			CSA			Proposal		
specimen	f'_{cc}/f'_{cb}	P EXP	f'_{ce}	P_n	$\frac{P_{EXP}}{P_{Theo}}$	f'_{ce}	P_n	$\frac{P_{EXP}}{P_{Theo}}$	f'_{ce}	P_n	$\frac{P_{EXP}}{P_{Theo}}$
IC1	1.89	874	59.2	1201	0.73	51.0	939	0.93	55.29	888	0.985
BC2	1.00	635	33.5	716	0.89	33.5	678	0.94	33.50	604	1.052
BC3	1.58	750	51.5	1055	0.71	48.4	912	0.82	50.65	838	0.895
BC4	1.89	930	59.2	1201	0.77	51.0	939	0.99	55.29	888	1.048
BC5	1.00	683	33.5	716	0.95	33.5	678	1.01	33.50	647	1.055
BC6	1.58	1090	51.5	1055	1.03	48.4	912	1.20	50.65	881	1.237
BC7	1.89	998	59.2	1201	0.83	51.0	939	1.06	55.29	931	1.071
BC8	1.00	704	33.5	716	0.98	33.5	678	1.04	33.50	691	1.018
BC9	1.58	1048	51.5	1055	0.99	48.4	912	1.15	50.65	925	1.133
BC10	1.89	1140	59.2	1201	0.95	51.0	938	1.21	55.29	975	1.169
	Mean				0.88			1.03			1.07
	Standard deviation				0.12			0.12			0.096
	Coefficient of variation				13.2			12.06			9.00

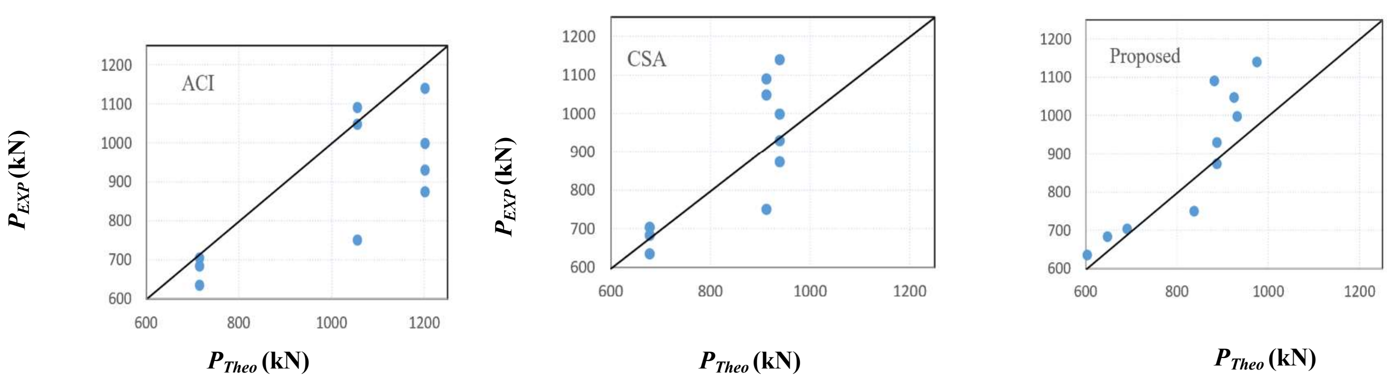


Figure 8 Experimental versus theoretical axial load capacity

4.CONCLUSION:

The following conclusions could be drawn on the basis of the experimental results:

1) The Confinement of the beam-column intersection by beams on two sides increased the axial capacity of the column by 6.1%.

2) The specimens with the same concrete strength for beams and columns. the increase in ties ratios from 0 to 0.0048 or 0.0098, the failure location has not changed.

3) The confinement (ties) in the specimens' joint transfers the final failure from the joint location to the upper or lower HSC column despite the joint being made from NSC.

4) The column's axial capacity increased by 45.3% and 39.7% by adding two ties (ρ_t=0.0048) and four ties (ρ_t=0.0098) at the intersection, respectively, for specimens with column concrete strength 53 MPa and beam concrete strength 33 MPa. In addition, the specimens with concrete column strength 63.3 MPa the increases were 7.3% and 22.6%, respectively. This means that the ties in the specimen's joint with MSC are more efficient than HSC.

5) ACI318-19 code equations show the overestimated values of (P_{EXP}/P_{Theo}). The proposed equations consider the effect of steel ties confinement, which was not taken in the existing models and gave more approach to the experimental results.

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