Visual Abstract

Interaction diagram for RC column strengthened by steel angles and strips

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ABSTRACT. This paper presents an analytical model to construct the interaction diagrams (normal force and moment) for the RC column strengthened using the steel jacket technique. The proposed model is defined using the strain distribution block by determining the location of the neutral axis in the concrete section. The proposed analytical formulation is verified by experimental results performed by previous researches and numerical models using the nonlinear program ANSYS. The factors affecting the capacity of the strengthened column are taken into consideration, such as the amount of loads resisted by the steel cage, steel strips spacing, and the effect of concrete confinement. The results of the proposed model are in good agreement with the results from the experimental and numerical work used in verification. A practical design formula has been presented for strengthened columns.

KEYWORDS. Reinforced concrete; Strengthening; Steel angles; Strips; Eccentricity; Interaction diagrams.

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INTRODUCTION

RC columns usually require strengthening to increase their capacities to sustain loads. For RC columns strengthened by steel angles and strips, four steel angles are fixed at the RC column corner and steel strips spaced at a suitable spacing and welded to the angles to form the steel jacket in this technique. Grouting is used to fill the small gaps between the steel cage and the concrete column. This strengthening system requires a limited area around the column section when compared with concrete jackets.

Many researchers studied the behavior and efficiency of the strengthened column using the steel jacket under purely axial loads [1–5]. They studied the strengthening parameters as the size of the angles and strips, concrete strength, steel strips spacing, and direct or indirect loading on steel angles.

Analytical models for determining the capacity of the strengthened column using the steel jacket under axial load were also carried out [4, 6–9]. They discussed the factors affecting the capacity of the strengthening columns as the amount of load resisted by the steel angles, the effect of steel strips spacing, the effect of direct and indirect loading on the steel cage, and the effect of concrete confinement.

For studying the strengthened column using steel jacketing under the eccentric load and slender column, some researchers conducted experimental and numerical studies to determine the capacity and mode failure of these columns [10,11]. Others

construct the interaction diagram N-M using the experimental and numerical investigation to predict the capacity of the strengthened column for different eccentricities [12–15].
In this study, a practical analytical formulation is presented to construct the interaction diagram for columns strengthened using steel jackets. The proposed formulation is practical for determining the capacity of the strengthened columns under different eccentricities. The proposed model is verified using previous experimental work results done by [10, 12, 14, 16]. Finite element models using the ANSYS program were also used to validate the proposed design formula.

STUDYING THE BEHAVIOR OF STRENGTHENED COLUMNS USING STEEL JACKETING

In order to study the behavior of concrete columns strengthened using steel jacketing (steel angles and strips), two main factors would be discussed. The first factor is the amount of load that the steel jacket can resist. The second factor is the improvement in concrete strength due to the confinement caused by the steel jacket. The majority of the researches conducted is carried out in their formulation of the ultimate load capacity of the strengthened column based on two basic modes of failure, failure caused by yielding of steel angles and failure caused by yielding in steel strips as follows.

Failure in the strengthened column due to yield in angles.

In case of failure due to yielding in angles, local buckling occurs in angles. Subsequently, the steel jacket is no longer able to confine the column. This behavior is based on the assumption that there are three points between every two strips, The two points at each strip are assumed to be hinged, while the point in the middle of the distance between strips is considered as a weak point in the angles as shown in Fig. (1).

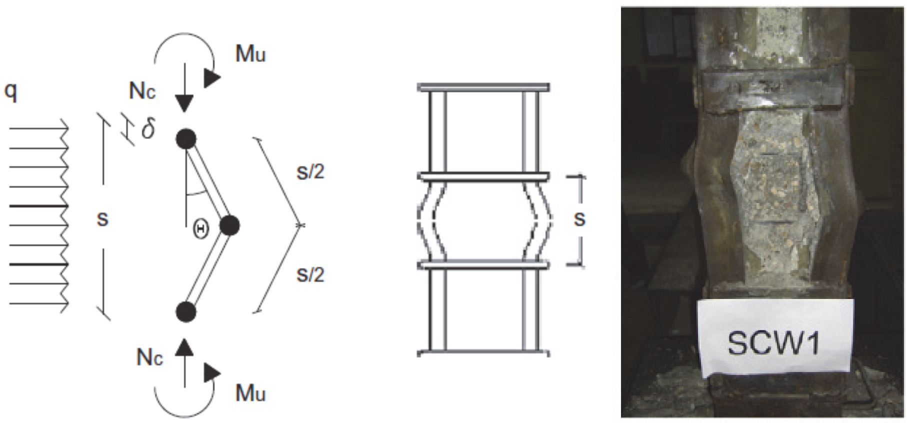

Figure 1: (a) The equilibrium model for the local buckling of the steel angles failure prediction. (b) Failure of the strengthened column due to local buckling in steel angles experimental work of Tarabia and Albakry [8].

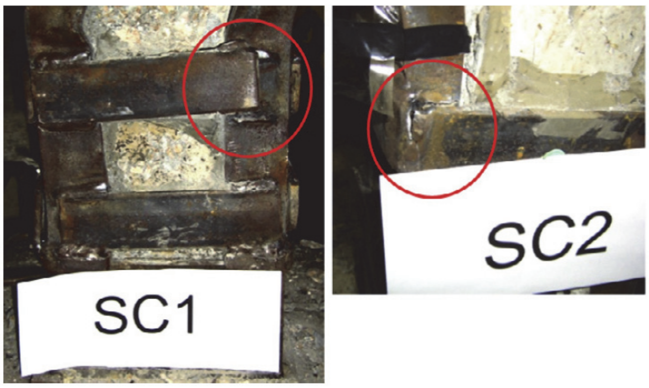

Figure 2: Failure of the strengthened column due to failure in strips experimental work Tarabia and Albakry [8]

Failure in the strengthened column due to yield in strips.

In this case, failure caused by yielding in strips due to the compression load on the column which leads to lateral strain in the concrete column and the steel jacket and causes an elongation in the strips subsequently the steel jacket isn't able to confine the column and the failure of the strengthened column occurred as shown in Fig. (2).

ANALYTICAL MODELS FOR LOAD-CARRYING CAPACITY:

Below are some of the analytical expressions for determining the ultimate load for the strengthened column using steel angles and strips, which would be used to produce the proposed formula. Fig. (3) shows the dimensions of the strengthened column using steel angles and strips used in the equations.

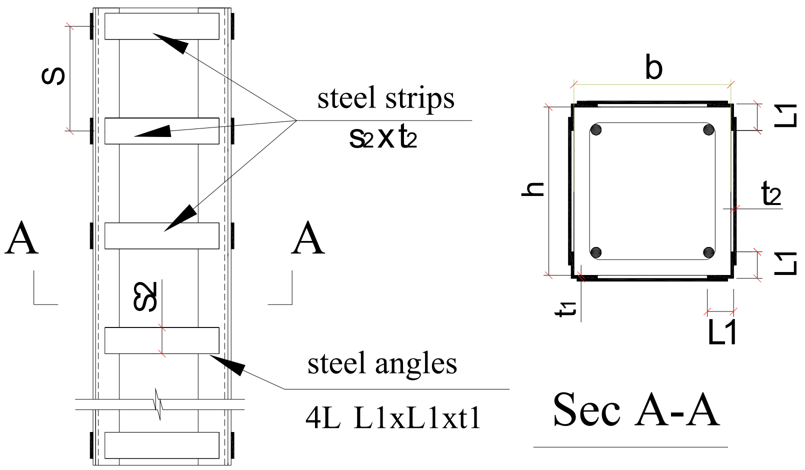

Figure 3: Main dimensions of the strengthened column using steel angles and strips.

Eurocode (2008) [17]:

Some researchers consider the column strengthened by steel angle and strips as a composite column. According to Eurocode, the ultimate load of composite columns can be expressed by the following equation:

$$p_{EC} = 0.85 \cdot b \cdot d \cdot f_c / \gamma_c + A_s \cdot f_{ys} / \gamma_s + 2.5 \cdot b \cdot d \cdot f_L / \gamma_a \tag{1}$$

where γ_c, γ_s, and γ_a are the reduction factors for concrete, reinforcement, and structural steel strength at the ultimate limit states in practical design. However, for real comparison with the experimental work, these factors can be dispensed with.

There are two main differences between the composite column behavior and the column strengthened by steel angle and strips, the first difference is the behavior of the composite action between the steel jacket and the concrete column. The second difference is the improvement of the concrete properties and strength due to the confinement of steel jacketing on the column, which would be addressed in the following researches.

Calderon et al. [6]

Calderon et al. [6] proposed a design formulation for determining the ultimate load carried by the strengthened column using steel angles and strips. The formula is based on the failure mode analysis observed in a numerical and experimental study presented in his research. The proposed design equation is expressed by the following equation:

$$p_{ca} = 0.85 \cdot b \cdot d \cdot f_c + A_s \cdot f_{ys} + 2.5 \cdot b \cdot d \cdot f_L + N_L \tag{2}$$

The factors N_L (axial load carried by steel angles) and f_L (confinement pressure) are calculated by two possible failure modes: failure due to material yielding of strips or yielding in angles as formed in the following equation.

$$f_L = f_{ystrip} \cdot \frac{t_2 . s_2}{s . b} . e^{\left(1-1.5\frac{s}{b}\right)} \quad (3)$$

where: (f_L) when failure due to yield in strips

$$f_L = \left[\frac{16.M_p \sqrt{2}}{f_c} . \frac{1}{b\left(s - s_2\right)} \right] \quad (4)$$

where: (f_L) when failure due to yield in angles.

Badalamenti et al. [7]

Badalamenti et al. [7] proposed a design equation for determining the ultimate load that is carried by the RC column strengthened with steel angles and strips based on the effect of concrete confinement and load carried by the steel angles. The formula is expressed as the following equation:

$$p_{\text{campione}} = b \cdot d \cdot f_{cc} + A_s \cdot f_{ys} + n_a \cdot \left(8 \cdot L_1 \cdot t_1\right) \cdot f_{yL} \quad (5)$$

where, f_{cc} = compressive strength of confined concrete; n_a = Maximum axial force in angles; n_a and fcc are calculated by using the following formula.

$$f_{cc} = f_{co}\left(1 + 4.74\frac{f_l}{f_{co}}\right)^{0.87} \quad (6)$$

$$n_a = \frac{\sqrt{t_1 \cdot f_{yl}\left(t_1 \cdot f_{yl} \cdot L_1^2 - \frac{q_{max} \cdot s^2}{3}\right)}}{2L_1 \cdot t_1 \cdot f_{yl}} \leq 1 \quad (7)$$

Campione [18]

Campione [18] proposed an equation for calculating the capacity of the strengthened column using steel jacketing. To determine the confinement pressure, it is assumed that the confinement pressure is reduced in steel strips suddenly while it remains constant along the steel angle. The effect of concrete confinement and composite action between the concrete column and steel jacketing is taken into consideration, as the following equation:

$$p_{ult} = n.b.d.f_c + n_a.8.L_1.t_1.f_{ys} + A_L.f_{yL} \quad (8)$$

where, n is the dimensionless load capacity of confined concrete core and n_a is the maximum axial force available indirectly loaded angles in the dimensionless as the following equation:

$$f_{cc} = \frac{f_{cc}}{f_{cd}}\left(1 + 1.42 \cdot \omega_s \cdot e^{\left(-1.5 \cdot \frac{s}{b}\right)}\right)^{0.87} \quad (9)$$

$$n_a = \sqrt{1 - 0.63 \cdot \frac{1}{t_1} \cdot \frac{s}{\mathrm{b}} \cdot \frac{1}{\frac{L_1}{s \cdot \mathrm{t}} \cdot \frac{\left(0.5 \cdot b - L_1\right)}{s_2 \cdot t_2}} \cdot e^{\left(-1.5 \cdot \frac{s}{b}\right)}} \quad (10)$$

Tarabia A. M. and Albakry H. F. [8]

Tarabia A. M. and Albakry H. F. [8] proposed an equation for determining the carried load by a column strengthened by steel angles and strips, compared to that proposed equation in Calderon et al. [6] only with different in determining confinement effect on concrete core and load carried by steel jacketing, as the following equation.

$$F_L = \frac{N_c}{b^2} \cdot \frac{\upsilon}{\left(1 - \upsilon + \dfrac{b \cdot s \cdot E_c}{2 \cdot s_2 \cdot t_{2.} \cdot E_s}\right)} \tag{11}$$

The load carried by the steel jacketing when axial compression of the column happens called direct loading, in this case, the steel jacket resists load with the concrete column from the beginning the load carried by the strengthened column, as the following equation.

$$p_{ult} = 2 \cdot L_1 \cdot t_1 \cdot f_{yL} \tag{12}$$

Campione et al. [14]

Campione et al. [14] proposed a design formula to define a plane fiber-section model of the column cross-section and take into consideration the frictional action along the column-angle face. The proposed formula is calibrated and validated by experimental results. The simple analytical stress-block procedure to derive continuous and simplified axial force bending moment domains is illustrated as a method for the hand-verification of reinforced cross-sections. The stress-strain laws assumed for the materials, the equilibrium equations of a reinforced cross-section written in the following form.

$$N_u = \alpha \cdot \beta \cdot b \cdot x_c \cdot f_{cc} + A_s^{\backslash} \cdot \sigma_s^{\backslash} + A_a^{\backslash} \cdot \sigma_a^{\backslash} - A_s \cdot \sigma_s - A_a \cdot \sigma_a \tag{13}$$

$$\begin{aligned} M_u = {} & \alpha \cdot \beta \cdot b \cdot x_c \cdot f_{cc} \cdot \left(d - \frac{\beta \cdot x_c}{2}\right) + A_s^{\backslash} \cdot \sigma_s^{\backslash} \cdot (d - \delta) + A_a^{\backslash} \cdot \sigma_a^{\backslash} \cdot \left(d + t - \frac{l}{4}\right) + \\ & + A_a \cdot \sigma_a \cdot \left(\delta + 1 - \frac{l}{4}\right) - N_u \left(\frac{d - \delta}{2}\right) \end{aligned} \tag{14}$$

where

x_c = concrete block neutral axis from compression zone;

σ'_s and σ_s = steel stress for top and bottom reinforcement;

A'_s and A_s = steel bars areas respectively;

σ'_a and σ_a = steel stresses for top and bottom angles.

Salman and Sherrawi [15]

Salman and Sherrawi [15] performed a nonlinear numerical analysis in order to determine the carried load of the high-strength column with steel angles in the corner of the column. Their numerical model takes into consideration the confinement effect of the concrete column due to existing steel angles and local buckling of it. They proposed a numerical method to predict the load capacity of the composite column at failure and study the efficiency of the steel angles in confining the concrete core. According to the numerical models, the column carried a large load after concrete cover spalling as discussed in their research.

PROPOSED ANALYTICAL MODEL

A numerical method will be discussed to construct the interaction diagram (M-N) for the strengthened column using steel angles and strips as shown in Fig. (4). The axial load is plotted versus the bending moment M till failure. This method is based on the stress-strain compatibility procedure [17]. The effect of confinement on the concrete core, the carried load by the steel jacket, the reduction in compression load in the steel jacket, and the steel jacket parameters are taken into consideration. The formulation is produced using the main four points from A to D as following.

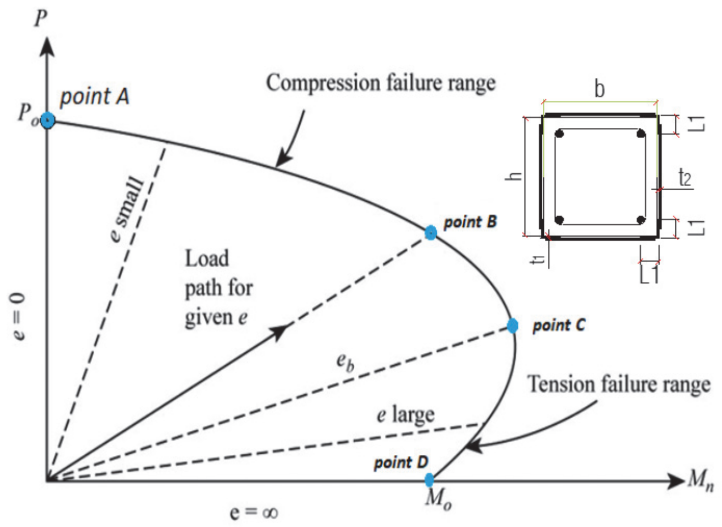

Figure 4: Main points used to plot the interaction diagrams for the strengthened column using steel angles and strips.

Point A (pure compression load:
Point A will be plotted as a point referring to pure compression failure. The maximum load-carrying capacity of the strengthened column takes into consideration the main parameters such as the concrete strength, the amount of steel reinforcement in the column, the steel yield stress, the effect of confinement on the concrete core, the carried load by steel jacket, the dimension of the steel angles and strips, and the composite action between the concrete column and steel jacket. The design model of Campione, [18] will be used to calculate the ultimate carried load capacity of the strengthened column with the equation's parameters shown in the previous section as following :

$$P_u = \alpha \cdot f_{cc} \cdot b \cdot h + n_a \cdot A_{s\ angles} \cdot f_{y\ angles} + A_s \cdot f_{ys} \tag{15}$$

Point B (compression failure assumed):
Point B will be plotted as a point referring to compression failure with a minimum eccentricity of the strengthened column. The compression failure is assumed to occur when the depth of the neutral axis is greater than its depth at the balanced position, Fig. (5).

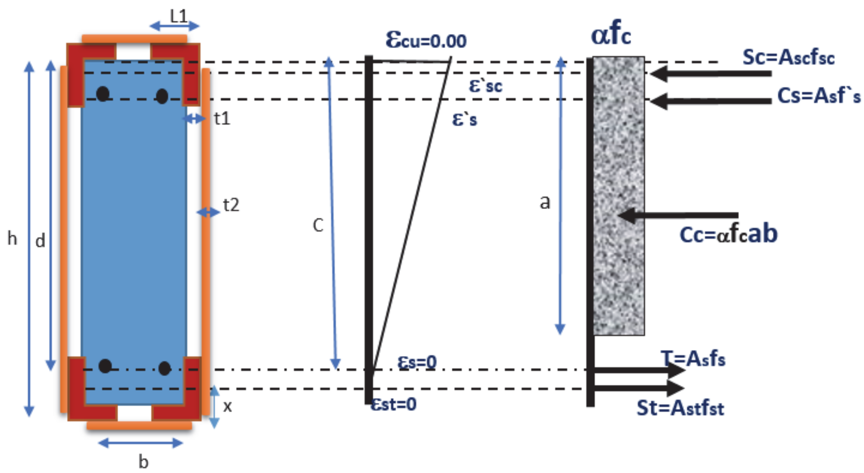

Figure 5: Stress-strain distributions of Point B which are used to plot the interaction diagrams for the strengthened columns.

In this case, the tension steel stress in the steel angles and reinforcement is below the yield stress, and for simplicity, the neutral axis position is chosen at the tension steel location (c=d). Thus, the developed force in the tension steel is equal to zero. The ultimate load and moment will be illustrated as follows:

$$P_u = \alpha \cdot f_{cc} \cdot b \cdot \mathrm{a} + A_{sc}' \cdot f_{sc}' + A_s' \cdot f_{ys}' \tag{16}$$

$$M_u = C_c \cdot \left(\frac{h}{2} - \frac{a}{2}\right) + C_s \cdot \left(\frac{h}{2} - d'\right) + S_c \cdot \left(\frac{h}{2} - x\right) \tag{17}$$

Point C (balanced failure assumed)

Point C refers to the balanced failure of the column section. The failure of the balanced section occurs when the concrete reaches its maximum strain simultaneous with the yield strain in steel Fig. (6). By definition, the point at the balanced section, the strain in the tension steel equals (εs). Thus, the stress in the tension steel equals (fy). The ultimate load and moment will be illustrated as follows:

$$P_u = C_c + S_c + C_s - S_T - T_s = \alpha \cdot f_{cc} \cdot b \cdot \mathrm{a}_b \tag{18}$$

$$M_u = C_c \cdot \left(\frac{h}{2} - \frac{\mathrm{a}_b}{2}\right) + C_s \cdot \left(\frac{h}{2} - d'\right) + S_c \cdot \left(\frac{h}{2} - x\right) \cdot 2 \tag{19}$$

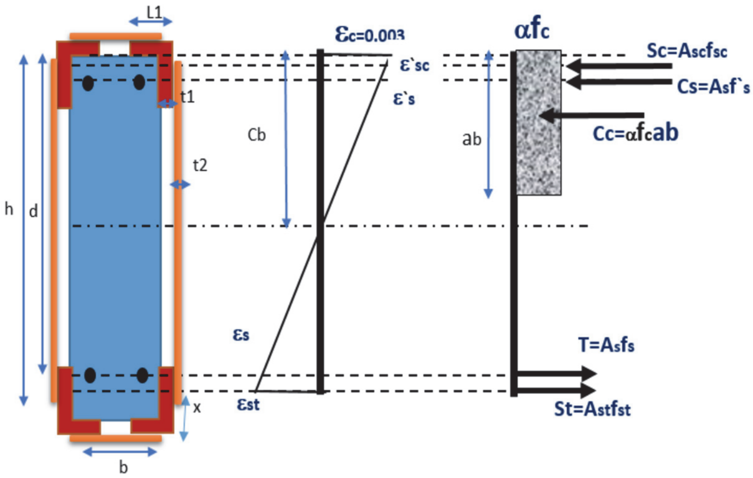

Figure 6: Stress-strain distributions of Point C which are used to plot the interaction diagrams for the strengthened column.

Point D (Pure bending)

In the case of a column subjected to pure bending or infinity eccentricity, the axial load is considered to be zero Fig. (7). The locating of the neutral axis must be performed by applying the equilibrium equation as following:

$$C_c + S_c + C_s = S_T + T_s \tag{20}$$

$$a \cdot f_{cc} \cdot b \cdot (0.8c) + \varepsilon_c \cdot E_s \cdot \frac{c - d'}{c} + \varepsilon_c \cdot E_s \cdot \frac{c - x}{c} = A_s \cdot f_{ys} + A_{st} \cdot f_{yst} \tag{21}$$

VERIFICATION OF THE ANALYTICAL PROPOSED FORMULA

In order to verify the results of the proposed model in this study, some experimental researches work and numerical models have been concerned with stresses, strains, and deflections for the strengthened column to verify the proposed formula. In this study, strengthened columns are modeled and studied using the commercial finite element software (ANSYS -Version 19.2).

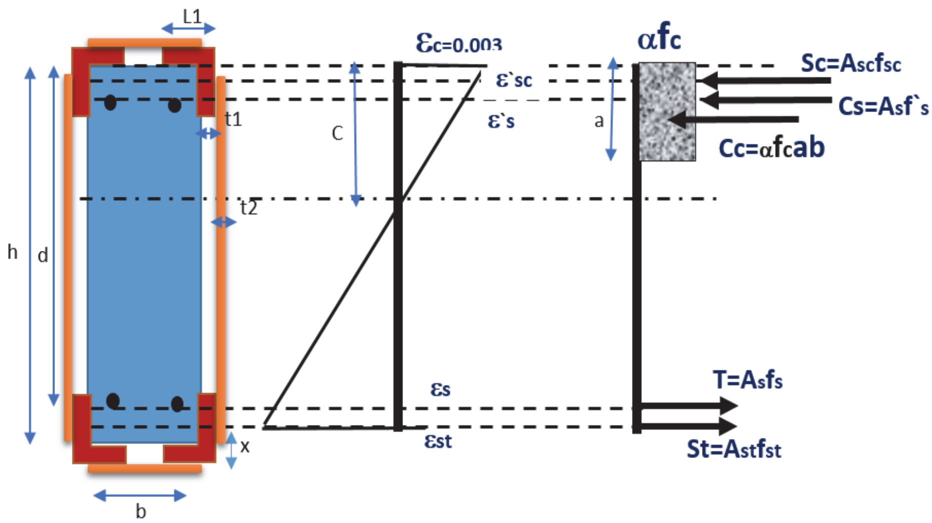

Figure 7: Stress-strain distributions of Point C which are used to plot the interaction diagrams for the strengthened column.

Verification using finite element models.

In order to verify the proposed model finite element models established in this paper using ANSYS software, the experimental researches work used in Verification has been concerned with stresses, strains, and deflections for the strengthened column. The methods for implementing the test, and the quality of the materials, and the configuration of the specimens. Solid element 65 is used to define concrete in 3D, link element 180 is used for steel reinforcement, solid 185 is defined for steel angles and strips, and steel plates for load distribution are defined as solid element 45. The components of the model, elements used, and boundary conditions are shown in Fig. 8.

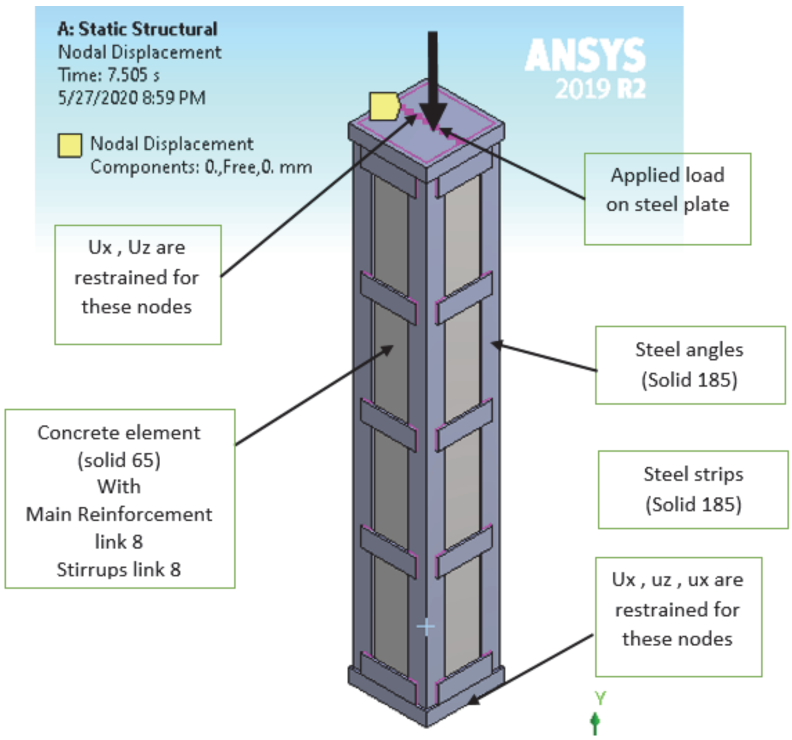

Figure 8: The numerical model's component used in the verification.

Verification using previous experimental work.

Experimental work results of previous researches will be used to validate the proposed analytical model. The first experimental work was taken from Montuori et al.[16], their specimens were (E-R1, D-R1, A-R1, and B-R1a) the second experimental work was taken from Elsamny et al.[12] their specimens were (Cl T3, C2 T5, C3 T7, C4 T3, C5 T5 , C6 T7, C7 T3, C8 T5, and C9 T7) . The third experimental work was taken from Ezz-Eldeen . [10] their specimens were (CS22e1 ,CS22e2, CS22e3 and CS22e4) . The fourth experimental work was taken from compaine et al. [14] their specimens were (RCAEX1 ,RCAEY1 ,RCBEX1 and RCBEY1). Tab. 1 shows the specimen's details and experimental work results. The results of the finite element models will be discussed in the next section.

Ref.	specimen	column section mm	fc' MPa	steel bars mm	steel angles	strips mm	fya MPa	e mm	N EXP kN	Failure load N FEM kN	N design kN	Comparison $\frac{N\,design}{N\,Exp}$	$\frac{N\,design}{N\,FEM}$
Montuori et al.[16]	E-R1	150×150x500	26.4	4 φ 16 mm	4 L 30×2	15x3@125 mm	353	50	745	782.7	630	0.85	0.80
	D-R1	150×150x500	26.4	4 φ 16 mm	4 L 30×2	15x3@125 mm	353	75	556	583.5	530	0.95	0.91
	A-R1	150×150x500	26.4	8 φ 10 mm	4 L 30×2	15x3@125 mm	353	50	717	752.6	620	0.86	0.82
	B-R1a	150×150x500	26.4	8 φ 10 mm	4 L 30×2	15x3@125 mm	353	75	524	550.1	520	0.99	0.95
Elsamny et al.[12]	Cl T3	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@490 mm	320	10	390	409.5	325	0.83	0.79
	C2 T5	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@245 mm	320	10	360	378	310	0.86	0.82
	C3 T7	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@164 mm	320	10	340	357	305	0.90	0.85
	C4 T3	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@490 mm	320	20	290	304.5	235	0.81	0.77
	C5 T5	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@245 mm	320	20	250	262.5	225	0.90	0.86
	C6 T7	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@164 mm	320	20	250	262.5	215	0.86	0.82
	C7 T3	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@490 mm	320	30	255	267.8	185	0.73	0.69
	C8 T5	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@245 mm	320	30	210	220.5	170	0.81	0.77
	C9 T7	120×120x1000	15	4 φ 8 mm	4 L 20×2	20x2@164 mm	320	30	210	220.5	160	0.76	0.73
Ezz-Eldeen . [10]	CS22e1	120×160x1000	28	4 φ 8 mm	4 L 20×2	20x2@250 mm	380	10	643	675.2	575	0.89	0.85
	CS22e2	120×160x1000	28	4 φ 8mm	4 L 20×2	20x2@250 mm	380	20	552	579.6	510	0.92	0.88
	CS22e3	120×160x1000	28	4 φ 8 mm	4 L 20×2	20x2@250 mm	380	30	474	497.7	455	0.96	0.91
	CS22e4	120×160x1000	28	4 φ 8mm	4 L 20×2	20x2@250 mm	380	40	420	441	410	0.98	0.93
compaine et al. [14]	RCAEX1	220x300x820	12.7	6 φ 12 mm	4 L 50×5	40x4@136 mm	275	65	1048	1100	1150	1.10	1.05
	RCAEY1	220x300x820	12.7	6 φ 12 mm	4 L 50×5	40x4@136 mm	275	55	1205	1266	1175	0.97	0.93
	RCBEX1	220x300x820	24	6 φ 12 mm	4 L 50×5	40x4@136 mm	275	65	1370	1439	1250	0.91	0.87
	RCBEY1	220x300x820	24	6 φ 12 mm	4 L 50×5	40x4@136 mm	275	55	1476	1550	1350	0.91	0.87

Table 1: specimens details and experimental work results and numerical models results.

RESULTS AND DISCUSSION

The comparison between the result of the proposed interaction diagram, experimental and numerical models is shown in Tab. 1. It can be seen that the results obtained using the proposed interaction diagram give a difference from 2 % to 30 % with an average difference of 12 % as illustrated in Tab. 1. It can be seen that the value of the experimental and numerical is bigger than the value of the proposed model, which is considered as an advantage of the proposed method as a conservative design approach. It is also noticed that the results of the strengthened column with big eccentricity have a small difference with the proposed model results than the column with small eccentricity. Fig. from (9-12) shows the comparison between the proposed interaction diagram and the experimental work of the researches used in verification.

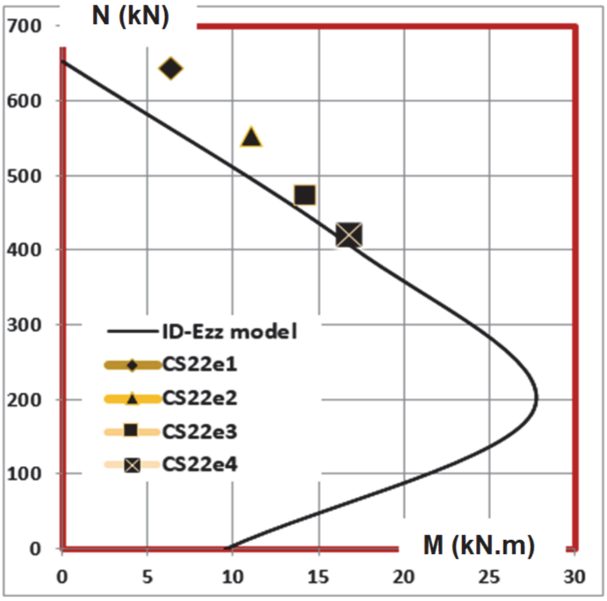

Figure 9: Comparison between the proposed I-D and the experimental work of Ezz-Eldeen, [10]).

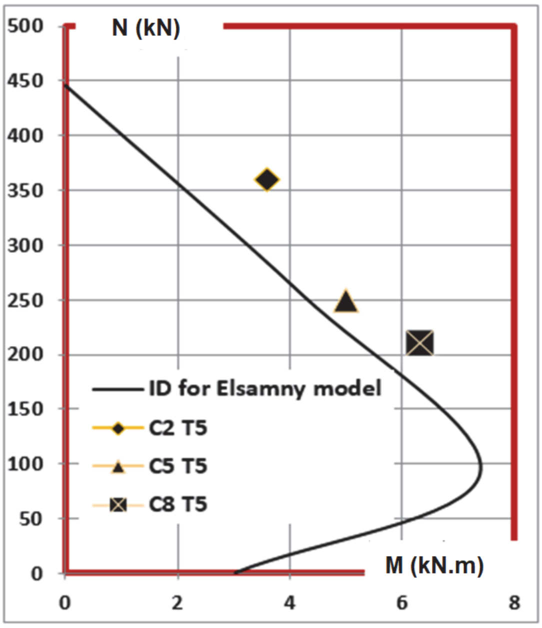

Figure 10: Comparison between the proposed I-D and the experimental work of Elsamny et al.[12])

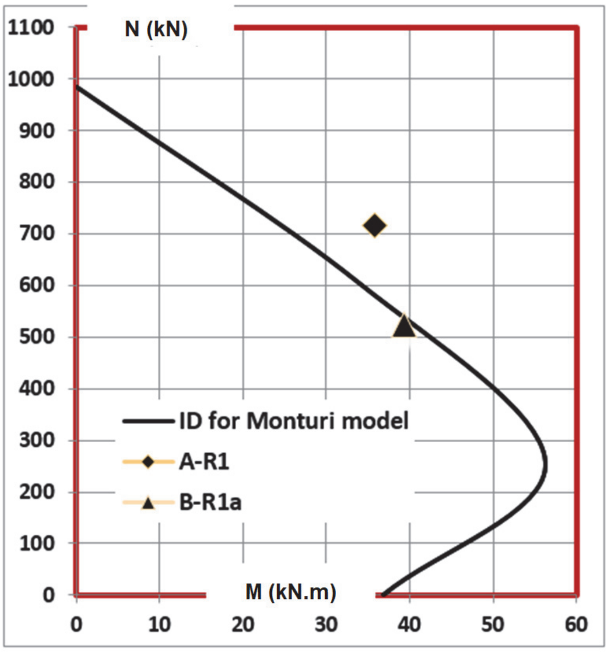

Figure 11: Comparison between the proposed I-D and the experimental work of (Montuori, and Rizzano [16]).

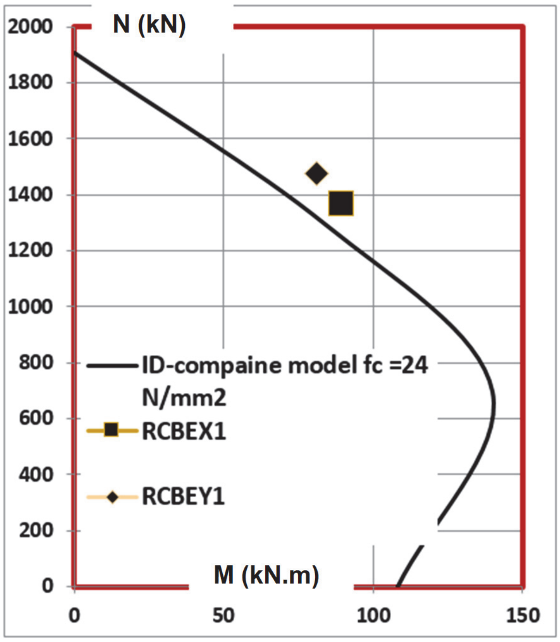

Figure 12: Comparison between the proposed I-D and the experimental work of compaine et al. [14].

As shown in Fig. (9-12), the interaction diagram constructed from the proposed model comparison with the experimental work results are in good agreement and gives a practical design method for determining the capacity of the strengthened column under the different cases of loading. It can be noticed that the first point (A) which refers to pure axial load is almost underestimation, while the other points have a good agreement with the experimental and numerical results. Fig. 13 shows the results of the numerical work using the finite element program.

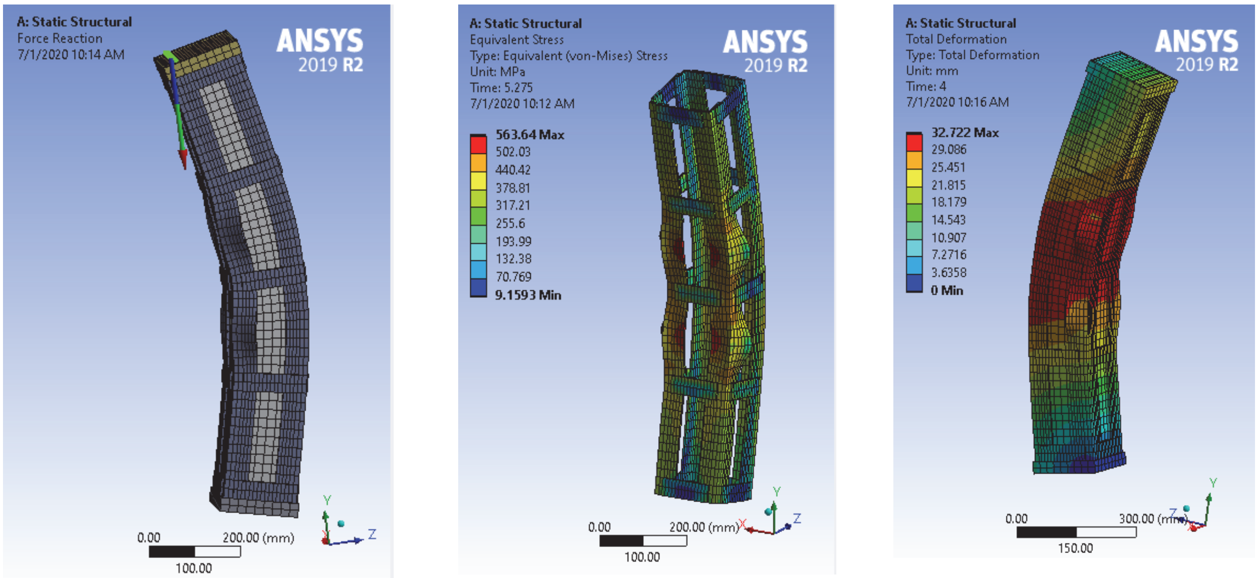

Figure 13: Deformation in strengthened column specimen (c1 t3) at failure comparison between numerical and experimental results for verification.

Conclusions

This paper presents an analytical design formulation to construct the interaction diagrams (N-M) for the RC column strengthened by steel angles and strips, and the following conclusions can be drawn within the scope of this study:

- The proposed analytical interaction diagram is efficient in determining the capacity of columns strengthened using steel jackets in different eccentricities.
- The proposed analytical formula takes into consideration the factors affecting the behavior and the capacity of the strengthened column as the amount of loads resisted by the steel angles, the effect of steel strips spaci ng, and the effect of concrete confinement.
- The proposed analytical formula was verified using experimental and numerical models with a good agreem ent with the difference from 2 % to 25 % with an average difference of 12 %.
- The finite element models using the ANSYS program give a variety in studying the parameters affecting the behavior of the strengthened column under different eccentricities and valuable for constructing and verific ation the proposed analytical formulation.

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