Check the bearing capacity of reinforced concrete columns using interactive diagrams according to TCVN 5574:2018, compare the results with British standard BS 8110:1997

Thach Som So Hoach[1,*]

[1] Faculty of Civil Engineering, Mien Tay Construction University;

[*]Corresponding author: mtthoach@mtu.edu.vn

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ABSTRACT

In the Etabs structural analysis software, there is a program to build interaction diagrams of columns and walls with any cross-section according to the standards of the most advanced countries: BS 8110:1997, ACI 318-2002, Eurocode 2 -2004, etc. Etabs allows model optimization and easy data entry, editing, and copying; execution time is significantly reduced compared to other calculation programs, especially for accurate, correct, and fast results.

In this article, the author builds interaction diagrams of reinforced concrete biaxially eccentrically loaded members with square and rectangular cross-sections according to TCVN 5574:2018 and interaction diagrams on Etabs according to British standard BS 8110:1997 (with material parameter conversion between the two standards TCVN 5574:2018 and BS 8110:1997). It is found that the results of the column interaction diagram between the two standards have insignificant deviation values.

Keywords: *BS8110:1997***,** *Interaction diagram, load-bearing capacity, reinforced concrete column, TCVN5574:2018*

1. INTRODUCTION

Currently, there are several methods for designing column reinforcement. For columns under plane eccentric compression, the longitudinal reinforcement area can be calculated using formulas from TCVN 5574:2018 [1], or from reinforced concrete design guides such as references [2,3], allowing for an accurate manual determination of the required longitudinal reinforcement. For columns under eccentrically inclined compression subjected simultaneously to the three internal force components, it is much more challenging to accurately design the required longitudinal reinforcement area. The commonly used approximate method for such columns is to convert the inclined eccentricity into an equivalent plane eccentricity, as introduced in [4]. However, after obtaining a preliminary calculation of the reinforcement area, it is necessary to arrange the reinforcement and verify the design using interaction diagrams in order to complete the column design process.

Nevertheless, after obtaining the preliminary reinforcement area, it is necessary to arrange the reinforcement and verify the results using an interaction diagram in order to complete the design process. The most reliable design method is the one based on the interaction diagram, because it is theoretically accurate and reflects the actual behavior of the cross-section (according to the assumptions stated in the code). The interaction diagram–based design method is the most comprehensive approach and can be applied to both columns and beams. However, its main drawback is that it requires processing a large amount of data, making manual computation impractical - programming tools are therefore necessary. At present, there are relatively few studies and references on column design using the interaction diagram approach according to TCVN 5574:2018 [5,6], and only a few commercial software programs, such as Ketcausoft and RDSuite, provide functions for checking the load-bearing capacity of columns.

The structural analysis software Etabs includes a module for generating interaction diagrams for columns and walls with arbitrary cross-sections, in accordance with most international design standards such as BS 8110:1997, ACI 318-2002, and Eurocode 2-2004, among others. Etabs allows for model optimization and provides convenient tools for data input, editing, and duplication. Compared to other calculation programs, it significantly reduces computation time while producing results that are accurate, reliable, and fast. However, Etabs does not yet support the Vietnamese design standard TCVN 5574:2018.

In this paper, the author develops interaction diagrams for biaxially eccentrically compressed columns with square and rectangular cross-sections according to TCVN 5574:2018, and also constructs corresponding interaction diagrams in Etabs based on the British Standard BS 8110:1997 (with material parameters converted between the two standards). A comparison of the results shows that the interaction diagrams of columns based on these two standards exhibit only negligible differences.

2. RESEARCH METHODOLOGY

2.1. Nonlinear Material Model According to TCVN 5574:2018 [1]

Compared with TCVN 5574:2012, the updated TCVN 5574:2018 introduces explicit stress–strain relationships for concrete and reinforcing steel materials. These relationships are simplified into bilinear (two-segment, MH2ĐT) and trilinear (three-segment, MH3ĐT) nonlinear material models, which can be conveniently applied in practical calculations.

In this study, the author presents and applies the trilinear model (MH3ĐT) for concrete and the bilinear model (MH2ĐT) for reinforcing steel, as illustrated in Figures 1 and 2.

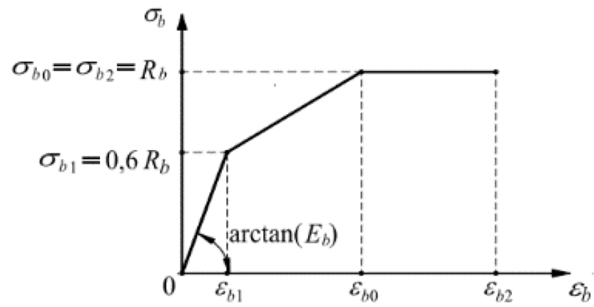

Figure 1. MH3ĐT for concrete material

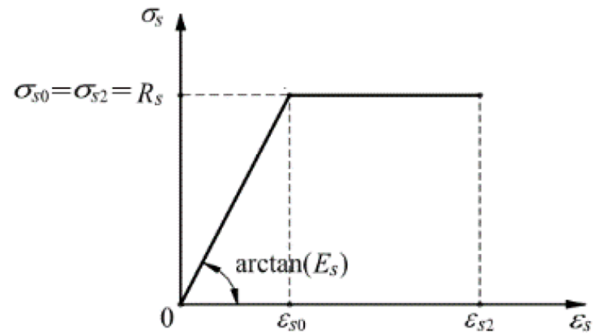

Figure 2. MH2ĐT for reinforcing steel

The trilinear model (MH3ĐT) of concrete is expressed by the following equations:

When $0 \le \varepsilon_b < \varepsilon_{b1}$: $\sigma_b = \varepsilon_b E_b$ (1)

When $\varepsilon_{b1} \le \varepsilon_b < \varepsilon_{b0}$:

$$\sigma_b = \left[\left(1-\frac{\sigma_{b1}}{R_b}\right)\left(\frac{\varepsilon_b-\varepsilon_{b1}}{\varepsilon_{b0}-\varepsilon_{b1}}\right)+\frac{\sigma_{b1}}{R_b}\right]R_b \quad (2)$$

When $\varepsilon_{b0} \le \varepsilon_b < \varepsilon_{b2}$: $\sigma_b = R_b$ (3)

Where: $\varepsilon_{b0} = 0.002; \sigma_{b1} = 0.6R_b; \varepsilon_{b1} = \sigma_{b1}/E_b;$ $\varepsilon_{b2} = 0.0035$ for normal-weight concrete whose compressive strength grade does not exceed B60.

The bilinear model (MH2ĐT) of reinforcing steel is expressed by the following equations:

When $0 \le \varepsilon_s < \varepsilon_{s0}$: $\sigma_s = \varepsilon_s E_s$ (4)

When $\varepsilon_{s0} \le \varepsilon_s < \varepsilon_{s2}$ $\sigma_s = R_s$ (5)

Where: $\varepsilon_{s0} = R_s / E_s$; $\varepsilon_{s2} = 0.025$.

2.2. Assumptions for Calculation [1]

- The cross-section remains plane before and after deformation. This assumption is used for calculating members subjected to bending and axial–bending loads. Based on this assumption, the strain at any point on the cross-section can be determined from the maximum strain of the concrete in compression and the reinforcing steel in tension.

- The tensile strength of concrete is neglected.

- Buckling and torsion of the column are ignored during the analysis.

2.3. Steps for Constructing the Interaction Diagram to Check the Load-Bearing Capacity of a Rectangular Reinforced Concrete Column [6]

+ Step 1: Input data: Cross-sectional dimensions of the column $C_x \times C_y$, reinforcement layout, material properties, and internal force values N, M_x, M_y. Select the compression vertex as one of the corners of the cross-section.

+ Step 2: Calculate the parameters:

$$\begin{aligned} e_{0x} &= \max(M_y / N; H / 600; C_x / 30; 1cm) \\ e_{0y} &= \max(M_x / N; H / 600; C_y / 30; 1cm) \end{aligned} \quad (6)$$

$$k_{bx} = \frac{0.15}{\varphi_{Lx}(0.3+\delta_{ex})}; \; k_{by} = \frac{0.15}{\varphi_{Ly}(0.3+\delta_{ey})} \quad (7)$$

Where $\varphi_{Lx}, \varphi_{Ly}$ - Factor accounting for the effects of long-term applied loads, taken not to exceed 2; $\delta_{ex} = e_{0x} / C_x$, $\delta_{ey} = e_{0y} / C_y$ Taken to be not less than 0.15 and not greater than 1.5.

Stiffness of the member:

$$\begin{aligned} D_x &= k_{bx} E_b I_{bx} + k_s E_s I_{sx} \\ D_y &= k_{by} E_b I_{by} + k_s E_s I_{sy} \end{aligned} \quad (8)$$

Where E_b, E_s corresponding, respectively, to the modulus of elasticity of concrete and that of reinforcing steel.

$I_{bx}, I_{by}, I_{sx}, I_{sy}$ these are, respectively, the moments of inertia about the x and y axes of the cross-sectional area, calculated separately for concrete and for all longitudinal reinforcing steel.

Nominal critical load:

$$N_{crx} = \frac{\pi^2 D_x}{L_{0x}}; \; N_{cry} = \frac{\pi^2 D_y}{L_{0y}} \quad (9)$$

Where L_{0x}, L_{0y} are the design lengths of the column in the x and y directions, respectively.

Factor accounting for the effect of longitudinal bending:

$$\eta_x = \frac{1}{1-\dfrac{N}{N_{crx}}}; \; \eta_y = \frac{1}{1-\dfrac{N}{N_{cry}}} \quad (10)$$

Moment considering the effect of longitudinal bending:

$$M_x^* = Ne_{0x}\eta_x; \; M_y^* = Ne_{0y}\eta_y \quad (11)$$

M_x^*, M_y^* these are, respectively, the bending moments about the x and y axes, taking into account the effect of longitudinal bending. Denote $\tan\alpha = M_x^* / M_y^*$.

Ultimate axial load:

$$N_u = \varphi\left[R_b\left(A_b - A_{s,tot}\right) + R_{sc}A_{s,tot}\right] \quad (12)$$

R_b, R_{sc} which are, respectively, the design compressive strengths of concrete and reinforcing steel.

$A_b, A_{s,tot}$ which are, respectively, the cross-sectional area of concrete and the total area of longitudinal reinforcing steel.

φ - Factor depending on the slenderness ratio of the column; for short-term loads, it is determined according to a linear relationship with $\varphi = 0.9$ when $L_0 / \max(C_x, C_y) = 10$ and $\varphi = 0.85$ when $L_0 / \max(C_x, C_y) = 20$.

+ Step 3: Subdivide the column elements: In this study, the author subdivides the column elements along the two directions into $n = 20$ parts, with each element having dimensions along the x and y directions of, respectively, $dx = C_x / 20$ and $dy = C_y / 20$. From this, the coordinates of each concrete element x_i, y_j and the coordinates of the reinforcing bars x_k, y_k can be easily determined.

+ Step 4: Assume the neutral axis is a straight line with the equation: $y = ax + b$ or $ax - y + b = 0$ having a slope of $a = -1/\tan\alpha$ and the coefficient b varies.

Distance from the compression vertex at coordinates (X, Y) to the neutral axis:

$$d = \frac{|aX - Y + b|}{\sqrt{a \; +}} \quad (13)$$

The distances from a concrete element at coordinates (x_i, y_j) and a reinforcing bar at coordinates (x_k, y_k) to the neutral axis are, respectively:

$$d_{ij} = \frac{|ax_i - y_j + b|}{\sqrt{a^2+1}};\ d_k = \frac{|ax_k - y_k + b|}{\sqrt{a^2+1}} \quad (14)$$

+ Step 5: From the results calculated in Step 4 and Section 2.1, for each position of the neutral axis, the strain of each element is determined based on the maximum strain of the concrete at the compression vertex:

$$\varepsilon_{ij} = \left(\frac{d_0 - d_{ij}}{d_0}\right)\varepsilon_{b2} \quad (15)$$

From Equation (15), the strains and stresses in the concrete elements and reinforcing bars can be determined (Figure 3) $\varepsilon_{bij}, \varepsilon_{sk}$, $\sigma_{bij}, \sigma_{sk}$ (neglecting the tensile stress of concrete in the calculations).

Step 6: Calculate the coordinates of the interaction diagram

$$N = \sum_{i=1}^{n}\sum_{j=1}^{n}\sigma_{bij}dxdy + \sum_{k=1}^{m}A_{sk}\left(f_{sk} - \sigma_{sk}\right) + \sum_{k=1}^{p}A_{sk}f_{sk} \quad (16)$$

$$M_x = \sum_{i=1}^{n}\sum_{j=1}^{n}\left(\sigma_{bij}dxdy\right)y_j + \sum_{k=1}^{m}A_{sk}\left(f_{sk} - \sigma_{sk}\right)y_k + \sum_{k=1}^{p}A_{sk}f_{sk}y_k \quad (17)$$

$$M_y = \sum_{i=1}^{n}\sum_{j=1}^{n}\left(\sigma_{bij}dxdy\right)x_i + \sum_{k=1}^{m}A_{sk}\left(f_{sk} - \sigma_{sk}\right)x_k + \sum_{k=1}^{p}A_{sk}f_{sk}x_k \quad (18)$$

$$M^* = \sqrt{M_x^2 + M_y^2} \quad (19)$$

Where i, j is the index identifying a concrete element under compression,, k is the index identifying the k-th reinforcing bar, m is the number of reinforcing bars in the compression zone, p is the number of reinforcing bars in the tension zone, f_{sk} is the stress in the reinforcing steel, σ_{sk} is the tensile stress of the reinforcing steel located in the compression zone.

+ Step 7: Represent the vertical plane of the interaction diagram with the origin at O(0,0). (This plane forms an angle α, with the horizontal plane *zOx*, where the horizontal axis represents the bending moment capacity and the vertical axis represents the axial load capacity.) Determine the position of the point *A(M, N)*, where *N* corresponds to the axial force in the column and the value of the bending moment $M = \sqrt{\left(M_x^*\right)^2 + \left(M_y^*\right)^2}$ with M_x^*, M_y^* is calculated according to Equation (11). The line OA intersects the interaction diagram at point B, and the load-carrying capacity factor is calculated as follows:

$$C_p = \frac{OA}{OB} \quad (20)$$

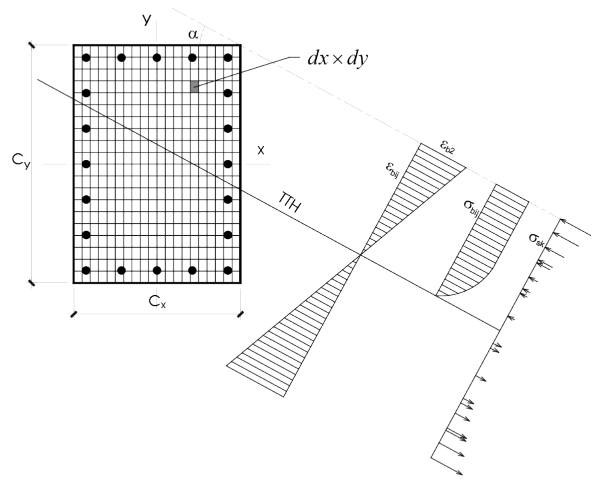

Figure 3. Stress–strain diagram of concrete and reinforcing steel elements

If $C_p \le 1$, the column is considered to have adequate load-carrying capacity; conversely, if $C_p > 1$, the column does not meet the load-carrying requirements and it is necessary to increase the reinforcement or the concrete cross-section.

2.4. Interaction diagram for checking the load-carrying capacity of a rectangular reinforced concrete column in ETABS

+ Material conversion according to TCVN 5574:2018 [7]:

- Concrete: characteristic strength of cubic specimens $f_{cu} = R_b \times 1.5 / 0.67$ (21)

- Reinforcing steel:

Yield strength of steel: $f_y = 1.15 \times R_s$ (22)

Tensile strength limit: $f_u = 1.5 \times f_y$ (23)

Here, R_b and R_s denote the design compressive strength of concrete and the design strength of reinforcing steel, respectively, expressed in MPa.

+ Input the model and the loads, as well as load combinations according to TCVN 2737:2023 [8].

+ Select the BS 8110:1997 standard to check the load-carrying capacity of the column.

+ Define the concrete and reinforcing steel materials according to the conversion formulas (21), (22), and (23).

+ Specify the reinforcement layout and the thickness of the concrete cover.

+ Run the analysis and review the results, including the load-carrying capacity check and the interaction diagram in Etabs.

3. EXAMPLE CALCULATION

Example calculation for checking the load-carrying capacity of the first-floor columns (including the column on axis 2B and the column on axis 6C) with a column height of 4.2 m, in a 7-story Administration Building (Mien Tay Construction University). Concrete of strength grade B20 is used, and the reinforcing steel is CB400-V.

Column 2B has a rectangular cross-section, with the reinforcement layout $20\phi25$, and concrete cover thickness $25mm$ as shown in Figure 4. Column 6C has a rectangular cross-section, with the reinforcement layout $14\phi20$, and concrete cover thickness $25mm$ as shown in Figure 5.

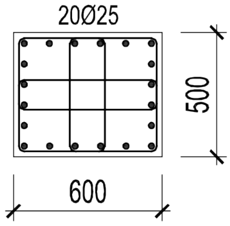

Figure 4. Longitudinal Reinforcement Layout – Column 2B

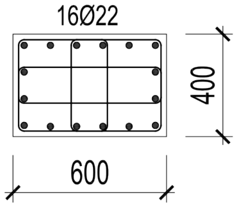

Figure 5. Longitudinal Reinforcement Layout – Column 6C

Proceed to perform the load-bearing capacity check for the two columns using 13 load combinations (COMB1 to COMB13).

3.1. Load-carrying capacity check of the column according to TCVN 5574:2018

The calculation steps are as presented in Section 2.3. Using VBA programming in Excel, the results are shown in Table 1 and Table 2, corresponding to column 2B and column 6C, respectively, at the column base cross-section.

Table 1: Calculation results of the load-carrying capacity factor C_p for column 2B

Comb	N (kN)	M2 (kNm)	M3 (kNm)	Cp
COMB1	3688.543	98.344	13.182	0.645
COMB2	3046.016	54.353	131.218	0.591
COMB3	2989.896	99.516	150.080	0.610
COMB4	2597.086	314.399	7.250	0.645
COMB5	3432.897	156.768	11.653	0.651
COMB6	3713.698	78.054	113.448	0.681
COMB7	3663.189	118.701	139.720	0.712
COMB8	3309.660	312.096	11.173	0.755
COMB9	4061.890	111.955	15.135	0.714
COMB10	3655.588	73.682	127.895	0.684
COMB11	3645.414	119.661	153.430	0.722
COMB12	3206.658	333.729	10.572	0.757
COMB13	4042.469	137.439	14.975	0.731

Table 2: Calculation results of the load-carrying capacity factor C_p for column 6C

Comb	N (kN)	M2 (kNm)	M3 (kNm)	Cp
COMB1	2538.712	50.003	23.232	0.657
COMB2	2431.523	15.220	88.144	0.722
COMB3	1984.925	72.144	53.661	0.576
COMB4	2468.474	181.494	15.607	0.815
COMB5	1951.130	265.054	18.768	0.811
COMB6	2739.743	24.442	87.075	0.786
COMB7	2337.804	75.674	40.550	0.651
COMB8	2772.999	152.601	21.791	0.849
COMB9	2307.389	249.293	24.636	0.870
COMB10	2734.942	20.970	93.569	0.802
COMB11	2330.207	78.987	47.909	0.659
COMB12	2771.893	175.744	21.032	0.881
COMB13	2254.549	270.805	24.193	0.891

It can be observed that columns 2B and 6C both satisfy the load-carrying capacity requirements

since $C_p < 1$. Column 2B has the most unfavorable internal force combination at COMB12, with the maximum load-carrying capacity factor $C_p = 0.757$ while column 6C has the most unfavorable combination at COMB13, with the maximum factor $C_p = 0.891$. The interaction diagrams corresponding to these most critical combinations are shown in Figures 6 and 7.

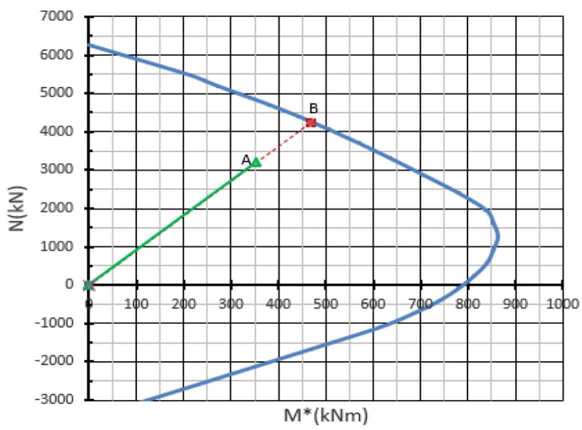

Figure 6. Interaction diagram of column 2B (COMB12)

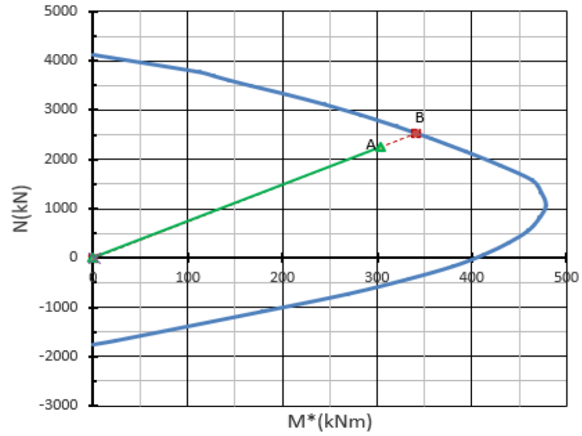

Figure 7. Interaction diagram of column 6C (COMB13)

3.2. The column's load-bearing capacity is analyzed in accordance with BS 8110:1997, with material parameters converted based on TCVN standards, using Etabs software.

The calculation procedure is described in detail in Section 2.4.

The interaction diagrams generated from the Etabs analysis at the base sections of columns 2B and 6C are illustrated in Figures 8 and 9, respectively.

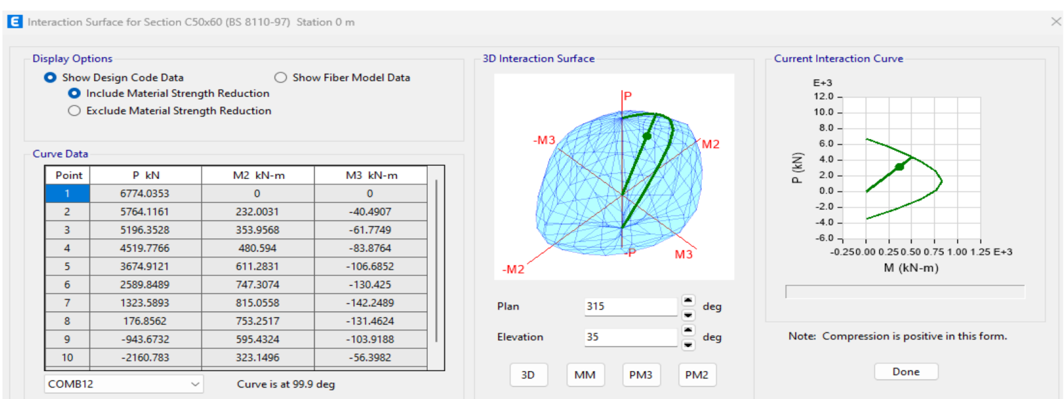

Figure 8. Interaction diagram of column 2B (COMB12)

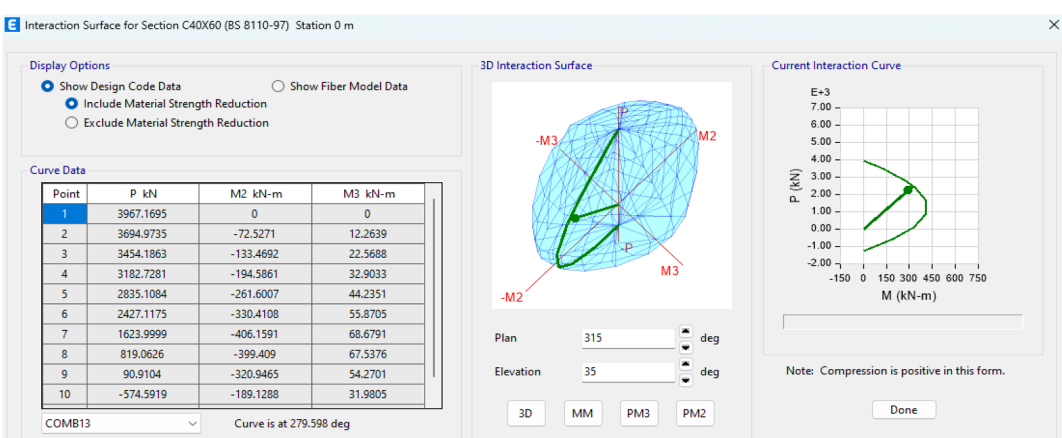

Figure 9. Interaction diagram of column 6C (COMB13)

The corresponding results extracted from Etabs are summarized in Tables 3 and 4, which present the load-bearing capacities of columns 2B and 6C at their base sections.

Table 3: Calculation results of the load capacity factor C_p for column 2B

Comb	N (kN)	M2 (kNm)	M3 (kNm)	Cp
COMB1	3688.543	98.344	13.182	0.646
COMB2	3046.016	54.353	131.218	0.598
COMB3	2989.896	99.516	150.080	0.619
COMB4	2597.086	314.399	7.250	0.628
COMB5	3432.897	156.768	11.653	0.640
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COMB9	4061.89	111.955	15.135	0.713
COMB10	3655.588	73.682	127.895	0.696
COMB11	3645.414	119.661	153.430	0.730
COMB12	3206.658	333.729	10.572	0.732
COMB13	4042.469	137.439	14.975	0.724

Table 4: Calculation results of the load capacity factor C_p for column 6C

Comb	N (kN)	M2 (kNm)	M3 (kNm)	Cp
COMB1	2538.712	50.003	23.232	0.725
COMB2	2431.523	15.220	88.144	0.794
COMB3	1984.925	72.144	53.661	0.635
COMB4	2468.474	181.494	15.607	0.840
COMB5	1951.130	265.054	18.768	0.819
COMB6	2739.743	24.442	87.075	0.873
COMB7	2337.804	75.674	40.550	0.711
COMB8	2772.999	152.601	21.791	0.886
COMB9	2307.389	249.293	24.636	0.887
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COMB11	2330.207	78.987	47.909	0.720
COMB12	2771.893	175.744	21.032	0.897
COMB13	2254.549	270.805	24.193	0.901

It is observed that both columns 2B and 6C satisfy the load-bearing capacity requirements.

Column 2B experiences its most critical load combination at COMB12, which produces the maximum capacity factor $C_p = 0.732$, while column 6C is most critical under COMB13, corresponding to its maximum capacity factor $C_p = 0.901$.

The interaction diagrams corresponding to these governing load combinations, extracted from the Etabs analysis, are illustrated in Figures 10 and 11, respectively.

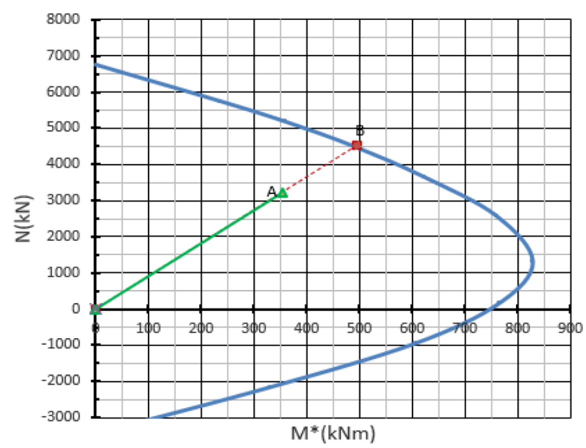

Figure 10. Interaction diagram of column 2B under load combination COMB12 (data extracted from Etabs)

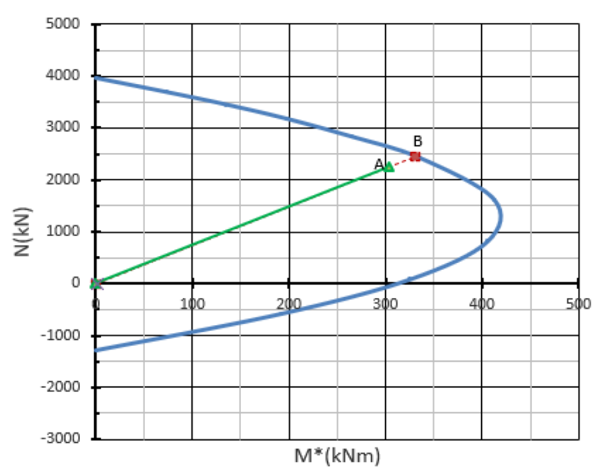

Figure 11. Interaction diagram of column 6C under load combination COMB13 (data extracted from Etabs)

3.3. Comparative Analysis of Column Load-Bearing Capacity According to TCVN 5574:2018 and BS 8110:1997

Comparison of Load-Bearing Capacity Factors and Deviations:

$$\Delta\% = \frac{C_{p1} - C_{p2}}{C_{p1}} \times 100\% \qquad (24)$$

Where C_{p1} is the load-bearing capacity factor calculated according to TCVN 5574:2018. C_{p2} is the load-bearing capacity factor calculated according to BS 8110:1997.

The results are summarized in Tables 5 and 6.

Table 5: Comparison of Load-Bearing Capacity Factors for Column 2B According to TCVN 5574:2018 and BS 8110:1997

Column	Comb	C_{p1}	C_{p2}	%Δ
2B	COMB1	0.645	0.646	-0.155
	COMB2	0.591	0.598	-1.184
	COMB3	0.610	0.619	-1.475
	COMB4	0.645	0.628	2.636
	COMB5	0.651	0.640	1.690
	COMB6	0.681	0.695	-2.056
	COMB7	0.712	0.722	-1.404
	COMB8	0.755	0.730	3.311
	COMB9	0.714	0.713	0.140
	COMB10	0.684	0.696	-1.754
	COMB11	0.722	0.730	-1.108
	COMB12	0.757	0.732	3.303
	COMB13	0.731	0.724	0.958

Table 6: Comparison of Load-Bearing Capacity Factors for Column 6C According to TCVN 5574:2018 and BS 8110:1997

Column	Comb	C_{p1}	C_{p2}	%Δ
6C	COMB1	0.657	0.725	-10.332
	COMB2	0.722	0.794	-9.953
	COMB3	0.576	0.635	-10.304
	COMB4	0.815	0.84	-3.090
	COMB5	0.811	0.819	-0.932
	COMB6	0.786	0.873	-11.083
	COMB7	0.651	0.711	-9.160
	COMB8	0.849	0.886	-4.368
	COMB9	0.870	0.887	-1.999
	COMB10	0.802	0.883	-10.125
	COMB11	0.659	0.72	-9.275
	COMB12	0.881	0.897	-1.846
	COMB13	0.891	0.901	-1.165

It is observed that the load-bearing capacity factors C_p calculated according to the two standards for columns 2B and 6C exhibit only minor discrepancies, with deviations ranging from -11% to 3.5%.

For the most critical load combinations—COMB12 for column 2B and COMB13 for column 6C—the interaction diagrams are plotted on the same coordinate axes based on the extracted data. The results are presented in Figures 12 and 13.

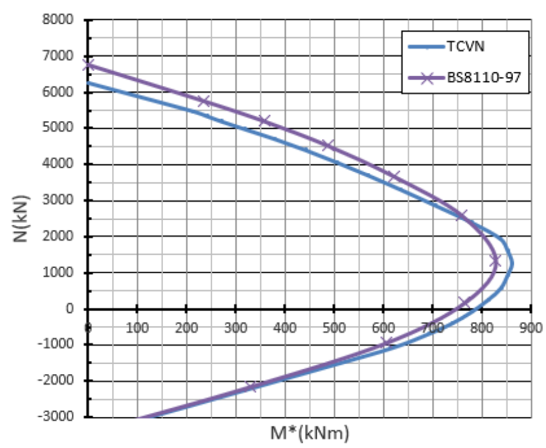

Figure 12. Interaction diagram of column 2B under load combination COMB12, based on TCVN 5574:2018 and BS 8110:1997

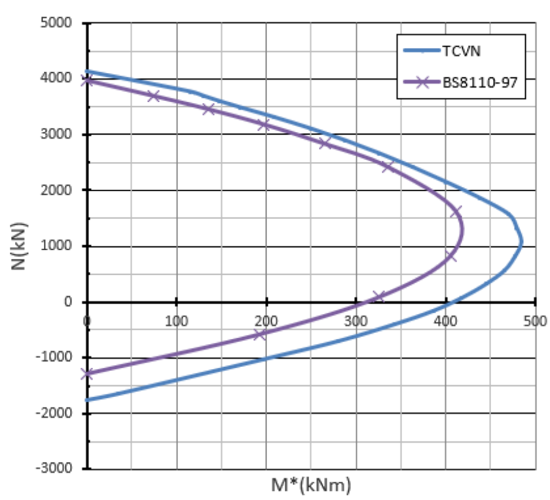

Figure 13. Interaction diagram of column 6C under load combination COMB13, based on TCVN 5574:2018 and BS 8110:1997

Remarks: The interaction diagrams of columns 2B and 6C exhibit only minor and insignificant discrepancies. The largest deviation occurs for column 6C at an axial load of approximately 1000 kN, with a deviation of about 12%.

4. CONCLUSIONS

The method of calculating eccentrically loaded reinforced concrete members by reducing them to an equivalent plane eccentricity provides results with limited accuracy. Nowadays, with the aid of computers and the interaction diagram method—which delivers the most reliable results—the author has developed a program to determine the load-bearing capacity of rectangular cross-section reinforced concrete members under eccentrically inclined compression in accordance with TCVN 5574:2018.

A comparison of the results with interaction diagrams generated in Etabs according to BS 8110:1997 (with material properties converted according to TCVN) shows, through various calculation examples, that the relative deviations are small, indicating good agreement and the reliability of the proposed method.

Although the number of examples presented in this study is limited, the results indicate that the deviations between the interaction diagrams are relatively small and acceptable. Therefore, structural designers can use Etabs following BS 8110:1997 as a reliable reference for checking the load-bearing capacity of reinforced concrete columns.

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