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STABILITY ANALYSIS OF REINFORCED CONCRETE SLENDER COLUMN

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STAIBILITY ANALYSIS OF REINFORCED CONCERETE SLENDER COLUMN

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Areej Taha Yaseen ALBU-ABO

Signature

DEDICATION

To

My Wonderful Father and Mother ..

My Lovely Husband ..

All My Teachers in my live ..

All those Who always encourage me to fly toward my dreams ..

AREEJ

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At first, all Gratitude is presented to Allah who enabled me to achieve this work.

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AREEJ

ABSTRACT

STAIBILTY ANALYSIS OF REINFORCED CONCRETE SLENDER COLUMN

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A study was made on 168 rectangular concrete columns (HSC, NSC) available in literature, tested under concentric and eccentric loads at short time. 149 RC slender columns have been experimentally tested and the rest are short. By using the theoretical predictions, test results are compared for the methods chosen of column capacity. Three current universal codes are being used (ACI 318M-19, BS 8110 and TS500). In this study, the flexural rigidity (EI), the stress intensity factor (α1), the ultimate compression Strain of concrete values are investigated to propose a new expression to predict the value of lateral deflection and the ultimate bearing capacity. For 12096 hypothetical rectangular RC slender columns. The proposed method will prove the effect of some properties of the column on the final bearing capacity and lateral deflection, such as (b/h, Lu/h, ρ). By the statically regression analysis of the columns, the effect of each factor will be shown. Comparisons between the proposed values and the standard value will be counted to observe the effect of the properties on the final values.

To reach the maximum accuracy of the theory a comparison was made between (P_{test} / P_{calc}) and it gave (COV=11.09)% and mean (X=1.21), also (P_{test} / P_{prop}) and it gave (COV=8.88)% and mean (X=1.19), that mean the proposed method reached the acceptable level of accuracy.

Keywords: Slender columns; Reinforced columns; Stability analysis.

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LIST OF ABBREVIATIONS

ACI	:	American Concrete institute (ACI 318M-19)
BS 8110	:	British Standard Institution.
TS 500	:	Turkish Standard Institution
COV	:	Coefficient of variation.
HSC	:	High strength concrete.
NSC	:	Normal strength concrete.
RC	:	Reinforced concrete.
SD	:	Standard deviation.

LIST OF SYMBOLS

A- Roman Letter Symbols

A_g	Gross area of section.
A_s'	Area of compression reinforcement.
A_{si}	Area of steel in the ith level.
A_{st}	Total area of longitudinal reinforcement bars.
b	Width of section.
b'	Smaller dimension of the column section.
b_i	Width of tensile flange.
C	Position of the neutral axis, (distance from extreme compression fiber to the natural axis
C_m	Correction factor to account for unequal end moments
c_n	Position of the neutral axis, distance from extreme compression fiber to natural axis , used by Rangan
d	Effective depth
E	Modulus of elasticity
e	Eccentricity of compressive loading on a column.
e/h	eccentricity ratio.
E1	moduli of elasticity of concrete, = 4700 $\sqrt{f_c'}$ in MPa;
e2	Larger end eccentricity;
e_b	Value of e at balanced failure.

E_c	Modules of elasticity of concrete
EI	Flexural rigidity of the column.
EI_{beam}	Effective flexural stiffness of beam
EI_{col}	Effective flexural rigidity of column at connection.
E_s	Modules of elasticity of reinforcing steel.
f'_c	Specified compressive strength of cylinder concrete.
f_1	Intercept factor
f_2	Stress correction factor
f_{ci}	Stress of concrete in the i^{th} position corresponding to the strain distribution
f_r	Modulus of rupture of concrete = $0.7\sqrt{f'c}$ in MPa
f_s'	Stress of compression steel
f_{si}	Stress of the reinforcing steel layer in the i^{th} position
f_y	Specified yield strength of reinforcement
f_{yh}	Yield stress of hoops in MPa
G_1, G_2	Ratios of column to beam stiffness at the two ends
G_s	Smaller of G_1 & G_2
h	Overall thickness of cross section in plane of bending
$\sum H$	Story lateral load producing Δ_1
h_f	Thickness of tensile flange
I_{cr}	Moment of inertia of cracked section
I_g	Moments of inertia of gross concrete cross section

I_{ss}	moment of inertia of structure steel
I_{se}	Moments of inertia of steel reinforcement
i_c	critical radius of gyration
k	Effective length factor for compression members
k_1	The ratio of the depth of the stress block to the depth of neutral axis in TS500 code
k_{ns}	Non-sway effective length factor
k_o	Effective length factor
k_r	An optional reduction factor
k_s	Sway effective length factor
L	Height of the story
ℓ	Height of the column
ℓ / h	Slenderness ratio
ℓc	Length of compression member in a frame
ℓe	$\beta\ \ell o$ = effective length of the column, in which ℓo is unsupported length of column
Lu	Unsupported length of the column
M_{02}	Larger end moment in a column obtained from the first-order conventional analysis
M_1	Smaller end moment on a compression member due to the load; positive when the member is bent in single curvature, and will be negative if the member bent in double curvature
M2	Larger factored end moment on compression member due to load, always positive

Ma	Max. un factored moment
M_{add}	Additional moment
M_c	Factored moment to be used for design of compression member
M_{col}	Applied end moment calculated by a conventional elastic frame analysis
M_{cr}	Cracking moment = fr Ig / yt
M_{cs}	Bending moment capacity of cross section
M_{max}	max. second-order moment along length of column
M_n	Bending nominal strength
M_{nx}	Nominal bending moments about x-axis
M_{ny}	Nominal bending moments about y-axis
M_r	Applied bending moment
M_u	Factored moment at section
n	The number of steel layers according to axis of bending, modular ratio = Ec / Es
N_{bal}	Axial load corresponding to the balanced condition
N_{uz}	Capacity of the column section under pure axial load
$\sum P$	Sum of the axial forces of all columns in a story
$\sum(P/L)$	Sum of the axial force-to-length ratio of all members in a story
P_{bal}	Axial load at balanced condition
P_c	Critical load, Euler load
P_{cal}	Nominal axial capacity for the column obtained analytically
P_i	The axial compressive force for member I

P_L	Axial load calculated only for rigid columns which provide side sway stiffness
P_n	Nominal axial compression (positive), or tension (negative).
P_{nb}	Nominal axial compression at balanced strain condition
P_o	Nominal RC cross sectional column capacity under pure axial Load
P_r	Applied axial load
P_{test}	Nominal axial capacity for the column obtained experimentally test.
P_u	Factored axial load at given eccentricity
P_{uz}	Pure axial load capacity of the column
Q	Stability index for a story
r	Radius of gyration of the column
R_{ns}	Ratio of axial compressive load to the non-sway Euler buckling load (P_u / P_c).
V_u	Factored horizontal shear in a story
X	Arithmetic mean
$\bar{y}$	Distance between the centroid of the compression block to the center of cross-section
y_t	Distance from centroidal axis of gross section
z	Distance between points of maximum and zero moment

B- Greek Letter Symbols

α	A dimensionless reduction factor (effective rigidity factor);
$\alpha 1$	The intensity of stress at the extreme fiber of stress block
$\beta 1$	The ratio of the depth of the stress block to the depth of neutral axis

βd	Creep factor
γ	Center-to-center distance between exterior layers of longitudinal reinforcement divided by the overall depth h for the column
γm	Partial safety for materials
δ	Moment magnification factor
δ_{ns}	Non-sway moment magnification factor
δ_s	sway moment magnification factor
Δ_1	First-order inter-story deflection
Δ_{cp}	Creep deflection
Δ_e	Elastic component of the total deflection due to sustained load
Δ_m	Lateral deflection at mid-height of the column
Δ_{tot}	Total deflection due to sustained load
Δy	Short term deflection at failure of the column
Ø	Strength reduction factor
Φ_{eff}	creep coefficient
η	the first order relative eccentricity
λ_m	mechanical slenderness of the column
ε	Strain
ε_{cu}	Ultimate concrete strain in compression
ε_s	Tensile strain in tension reinforcement
ε_y	Yield strain of reinforcement

λ	Parameter (= P/EI) in (1/m)
ξ	Depth of equivalent rectangular compressive stress block for the cross section by Duan
ρ	Longitudinal steel reinforcement ratio
ρ_h	Volumetric ratio of hoop (ties) reinforcement to confined concrete
ρ_t	Total ratio of reinforcement (= Ast / Ag)
σ_{cr}	Average stress over the cross sectional area of the column at the critical load
$Ø_{cc}$	Creep factor used by Rangan
$Ø_e$	Curvature at column ends
$Ø_y$	Curvature of a section at the first yield of tension steel

1.INTRODUCTION

The column is an important structural element, it is considered a compression member whose height is three or more times greater than the lateral dimension and it is mainly used to support loads in structure. Currently, there are several types and shapes of column, it will be different according to the type of reinforce such as columns reinforced with longitudinal bars in addition to closed spaced spirals. Also some types of columns contain composite compression elements that are reinforced longitudinally and with shape steel structure, and its shapes are tubes, pipes with or without additional bars. [1]

All columns are subjected to the loads that are axially, uniaxial, or biaxial. At the present time, columns are most widely used are short column, which ultimate load is controlled at a specified eccentricity by the cross section dimensions and the bearing strength of the materials. also the slender column which is affected by slenderness and ultimate load which may cause transverse deflections in the column during the loadings.

The theory that used to explain the effect of load on the structure is when the structure is subjected to a load at a certain point the structure will undergo deformation so that the bearing point moves towards the affecting general load. Sometimes the impact of deformations associated with forces with a critical axial load Euler (P_c) is large and causes high stress making the structure unserviceable. While, the nonlinear analysis of the structure and behavior of inelastic materials makes it extremely complex. As a result, Erule Buckling (P_c)[2] has an essential role in the analysis and its equation (1-1) is explained below:

$$P_c = \pi^2 EI / (KL_u)^2 \qquad \text{....... (1-1)}$$

Where

P_c =axial load on the structure,

EI = flexural rigidity of the column,

K = the effective length factor,

L_u =unsupported length of column,

Can be written the eq (1-1) is another format :

$\sigma_{cr} = \pi^2 E \; / \; (KL_u \; / \; r^2 \;)^2$ ……. (1-2)

Where

σ_{cr} = the average stress over the cross-section at the critical load,

E =modulus of elasticity of material,

r =radius of gyration.

Column slenderness is greatly affected by the lateral deflections that occur as a result of bending. Also, the effects of deflections depend on the type of loads and the conditions of the ends of the column. Slenderness ratio has been determined by different approaches (KL_u / r) ,(L_u / r) , (L / r).

In slender column a secondary moment (P Δ) is happening as a result from the deflection caused by the effect of the axial forces. The behavior of the column under the gravity effect alone or under load and gravity combined cannot be known precisely through first-order analysis[3].

In the short column type, it has a low slenderness ratio where the ultimate strength is not affected by the height and there will be no secondary moments due to no deflections or it can be completely neglected. However, in slender column, the ultimate strength is affected by deflection. The type of load applied on the column determines the resulting deflection, it can be a single or double curvature. The column reaches the maximum value of the moments (first order analysis plus additional secondary moment) at the ends of the columns or within the heights, that means the maximum moments occur within the height between its ends for singular curve states (non-sway columns). Columns with non-sway frames that under effects double curvature, the secondary moments do not amplify the primary moments (first-order analysis) and occur at the ends. columns with sway frames that are not subjected to a high axial load, the magnification moments with high values may occur at the ends. Deformations occur in the columns while the structure is subjected to loads, which leads to a decrease in the rigidity of the column and beams

Through the studies that have been made for the short column in non-sway frame, its shows that, when the decrease in the moments occurs due to the decrease in the stiffness is greater than the increase in the moments due to the deflection, then the load capacity of columns will increase while the maximum moments decrease.[4]

For the slender column in non-sway frame, when the moments due to deflection increase more rapidly than the restraint moments, then the load capacity of columns will decrease while the maximum moments increase.[3][5]

Fig (1-1) shows research made by Furlong and Ferguson [3] to explain the behavior of slender non-sway column.

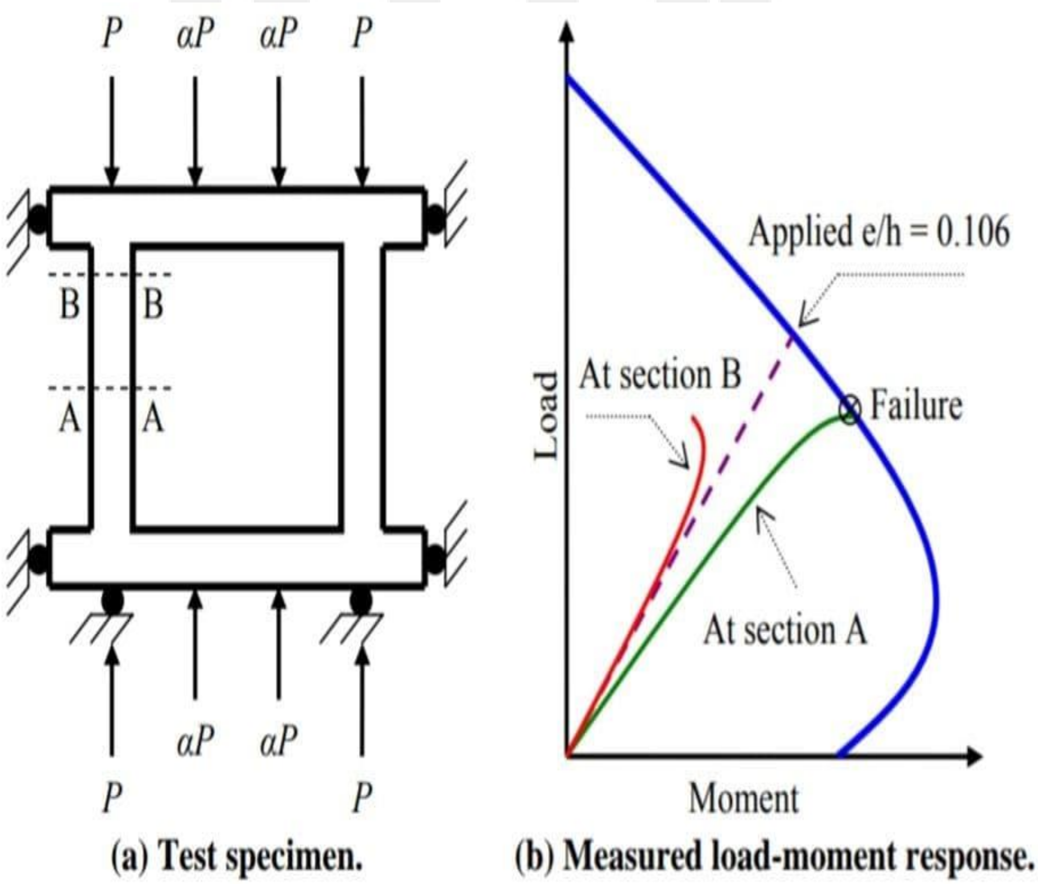


Fig (1-1) Behavior of column in frame tested by Furlong and Ferguson [3]

In order to find the values of forces and moments for columns in frames, an accurate analysis is required. since the second-order analysis is complicated, so alternative approach shall be taken magnified moment methods, this approach includes a first-order structural analysis and the values of lateral deflections (P Δ) are included.

At the present time, there are many methods of analysis and design for columns and frames. To calculate the forces and moments. which will be further explained and discussed in Chapter Three.

1.1 OBJECTIVES

The main objective of this work is to predict the bearing capacity of a reinforced concrete column. This research includes the Universal methods (codes) used for analysis and design (ACI 318M-19, BS 8110 and TS500), and the method that has been proposed is provided with some modifications to the current methods.

168 concrete columns available in the literature. Various models of rectangular concrete column are used in this work, includes short and slender, HSC and NSC columns. The samples were tested under concentric, eccentric and short time loading.

1.2 CONTENT OF THE RESEARCH

This research is divided into five chapters, the first chapter the general definition and description of the types of columns and their behavior. The second chapter conducts extensive review of the research and a survey of the literature and its summarization. The third chapter analyze and explain the theories that were used to find these methods, whereas each method is described separately. Chapter Four, clarify the columns that are available in the laboratory based on the literature and the methods used to analyze 168 columns, and discuss its finding. Finally, chapter number five will describe and detail out the conclusions, suggest recommendations, and to be facilitated for further future research activities.

1.3 PURPOSE OF THIS RESEARCH

As a result of the continuous urban development and the proliferation of tall buildings, modern technology methods have been found to produce high strength concrete (HSC), it is widely used in multi-story buildings (high-rise buildings). This type of concrete provides high structural efficiency as better bearing strength and improved stiffness.

As the high-strength type of concrete is widely used in buildings and structures, therefore slender columns have evolved in their composition in practice. Several developments have been made in the formation of slender columns through some methods and expressions such as bending stiffness (EI), strain values ... etc, all these factors will directly effects on the column, whereby a new proposed formula will be formed to predict the values of bearing capacity and the values of deflections as a result of the imposed loads[6] . This is achieving by comparisons between the deflections and bearing capacity that obtained at the laboratory of 168 columns and the results obtained from the application of the new proposed formulas.

This study includes a discussion of the quality of each method by analyzing different types of columns, slender, short, sway and non-sway frames and the columns that made of high strength concrete HSC and normal strength concrete NSC.

2. LITERATURE REVIEW

2.1 GENERAL

During the past years, scientists have adopted several different methods to analyze and design building columns, along with understanding the behavior of these columns when load is applied. The suitable behavior of the slender columns depends directly on the additional length of the column; this factor will lead to the reduction in the column strength. According to the difference in the effective length, section width, slenderness ratio and end conditions of the column, all these factors determine the rigidity of the column.

In order to accurately analyze the column, the forces and moments must be determined at each point in the column and adopt second-order analysis method to correspond the cross-section of the column resisting the loads.

In this chapter, some present researches that explains and analyzes the behavior of slender concrete columns

2.2 HISTORICAL THEORY

For decades, tests have been conducted in several universities around the world to analyze and interpret the behavior of the concrete column cross-section under the influence of loads. The use of design and analysis theory depended on the ultimate strength of cross-sections is in effect come back to the original theory of design, the early design formulas were empirical, being depended on the failure due to loads of typical elements.

One of the first theories to be used was called the theory of ultimate load,[7] Koenen’s theorem in 1886 in north America which specified the distribution of concrete stress as a straight line and neutral axis at med-depths, and that shown in fig(2-1) below

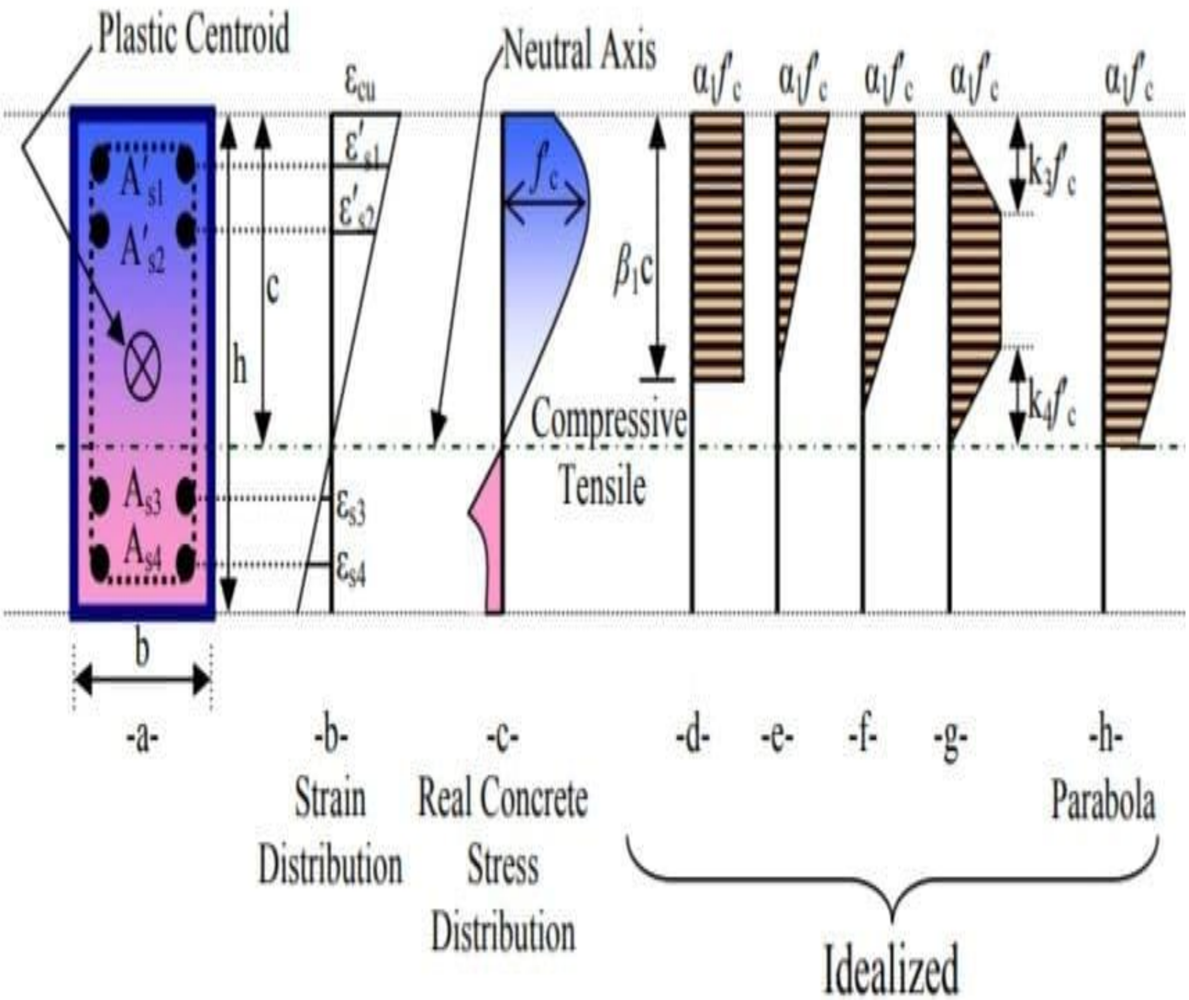


Fig. (2-1) Concrete stress and strain distribution of column cross section.

Figs. (2-1) ignore the tensile stress of concrete.

One of the first theories to be used was called the theory of ultimate load,[7] Koenen's theorem in 1886 in north America which specified the distribution of concrete stress as a straight line and neutral axis at med-depths, and that shown in fig(2-1) below

Many scientists have conducted research on the stress distributions of columns. These experiments and analysis show that using the principle of {rectangular stress block} type in design and analytical calculations will show more accurate and effective results.[8] In 1904, for the first time, Von Emperger proposed the use of a stress block with a rectangular shape. Many engineers at that time followed this formula, the most famous was C. S. Whitney, where his papers were from the literature of the design of ultimate strength design.

In the 1930s an extensive investigation of the ACI code was conducted on con-centrally loaded columns where a law was concluded stipulating that the final strength of the column is equal to 85 percent of the force applied on the cylinder multiplied by the area of steel. Where the approach used was a rectangular stress block with a maximum stress 0.85 fc` , where $\alpha 1 = 0.85$.

Jensen[9], presented a study showing the stress distribution in the shape of a trapezoid and derived the properties of this shape as a function of the cylinder strength. Then the maximum value of the stress distribution was found by Hognestad[10] with a value equal to 0.85 use of the ascending parabola and a straight descending line after the maximum stress.

Conclusions were presented through experiments conducted at that time in several universities, including Imperial College, University of London and University of Illinois in 1956, [7]Mattock created appropriate values for design and analysis through the parameters, where β1 mean ratio of the depth of the stress block and α1= 0.85 mean the intensity of stress, at the extreme fiber of stress block shown in the eq (2-1) below: -

$\beta 1= 1.05-0.05\ (f_c'/6.9)\leq 0.85$ for f_c' in MPA …..(2-1)

Which also proved that the compressive strength of cylindrical concrete, whether central or not, ranges between (8.2-57.4) MPa within a rectangular stress block. In an attempt to obtain the best relationship between stress and strain of concrete during the loading process and reaching the failure stage. The maximum value of the compressive strain of 0.003 was obtained at the University of Illinois through laboratory experiments on concrete and reinforced concrete.

In 1976[11], the ACI318-71 Code was adopted to find the minimum value of $\beta 1 = 0.65$ for cylinder concrete strength exceeding the 55 MPC, according to Nederman's opinion.

$\beta 1 = 0.85 - 0.008\,(f_c' - 30)$ $\qquad 0.65 \leq \beta 1 \leq 0.85 \qquad$ for f_c' in MPa …..(2-2)

Continuous development over the years and as a result of the development of techniques for producing different types and shapes of materials according to advanced specifications, as high-strength columns(HSC) were produced with high compression strength more than normal-strength columns(NSC)[12], where in 1950 the compressive strength was adopted with a value from 30 MPa and More for high strength columns.

Depending on the characteristics and features of (HSC), the researchers tried to obtain suitable values for the $\alpha 1$ and $\beta 1$ of the rectangular stress block, where the $\alpha 1$ value should not exceed 0.85 and not less than 0.75 with a value of f_c' equal to 80 according to the opinion of Park and Setty et al[12] , it’s given in eq (2-3):

$\alpha 1 = 0.85 - 0.004(f_c' - 55)$ $\qquad 0.75 \leq \alpha 1 \leq 0.85 \qquad$ for f_c' in MPa …..(2-3)

S. A. mirza[13], in ACI code approach used 9500 columns for analysis process was based on the stress-strain curve, where this approach is affected by the flexural rigidity El of the column and considered the $\alpha 1$ and $\beta 1$ values as shown in the Eq (2-4)&(2-5) below

$\alpha 1 = 0.85 - (f_c'/800) \geq 0.75$ $\qquad$ for f_c' in MPa …..(2-4)

$\beta 1 = 0.95 - (f_c'/400) \geq 0.70$ $\qquad$ for f_c' in MPa …..(2-5)

Lawrence F. Khan and Karl F. Meyer[14], conducted a study of 19 high-strength columns according to ACI318M code that used at that time, used both equations (2-2) and (2-3) to find $\alpha 1$ and $\beta 1$,Where fc′ between (66.4-102.2) MPa, ultimate strain of concrete = 0.003, the modulus of elasticity Ec= $4700\sqrt{fc'}$. Where used rectangular stress block on triangular and circular compression zones, and results showed conservative values, some of the beams were a semicircular cross-section to make the compression zone in a circular column under a center load and bending, the remaining beam were triangular cross-section to make rectangular column under the influence of biaxial bending. The reinforcing distribution was according to the size of the compression area and the location of the neutral axis, where it was divided to measure the strain of both concrete and steel.

Ford et al [15], Presented a study on a set of columns to study the ability of deformation and predict the instantaneous bending of large axial loads. The strains ranged from 0.003 to a maximum safety value of 0.004, where the ultimate compressive strain of RC as equation shown below:

$$\epsilon_{cu} = 0.003 + 0.02\ b/z + (\rho\ f_y/100)^2 \quad \text{…. (2-6)}$$

where

z = distance between points of maximum and zero moment

ρ = volumetric ratio of ties reinforcement

f_y = yield stress of ties in MPa

Migue et al[16], introduced a new iterative algorithm for finding the ultimate strain limits used in bending theory. The study provided high efficiency for any shaped cross section by comparing with experimental results.

Through all experiments, it was found that the accuracy of the results increased in the rectangular compression block compared to the semicircular and triangular shapes due to the shape of the compression block. They recommended their results using the rectangular stress block in the ACI code, and the results were satisfactory for the values of the moment capacities.

In this time all codes ACI318M-95, ACI318M-02, ACI318M-05, ACI318M-11, ACI318M-14, ACI318M-17, ACI318M-19 all are still use constant value of ϵ_{cu} = 0.003, α1=0.85 and β1 use formulas below:

- For ACI318M-95, ACI318M-02

β1= 0.85- (fc'-30)/140 ….. (2-7a)

- For ACI318M-05, ACI318M-11, ACI318M-14, ACI318M-17, ACI318M-19

β1= 0.85- (fc'-28)/140 ….. (2-7b)

In all standard the limit of β1 not greater 0.85 and not less than 0.65.

Ref [17][18], the British code BS (8110), the ultimate compressive strain of RC in this code equal to 0.0035. the values of this code are based on a rectangular stress block to design and analyze the concrete sections according to the fig(2-2) below, α1=0.67 (for f_{cu} in MPa) and β1=0.9 , Note that f_{cu} is the compressive strength value of cubic concrete, and γ_m is a partial safety factor for the strength of the material and equals 1.5 (at reciprocal is 0.67) , that show in fig(2-3) below

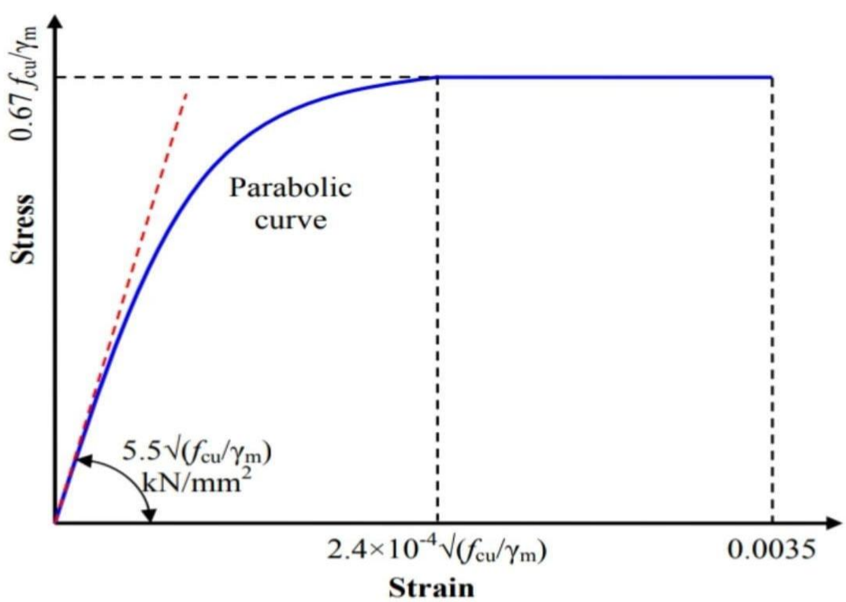


Fig (2.2) short term design stress-strain curve for normal-weight concrete in compression

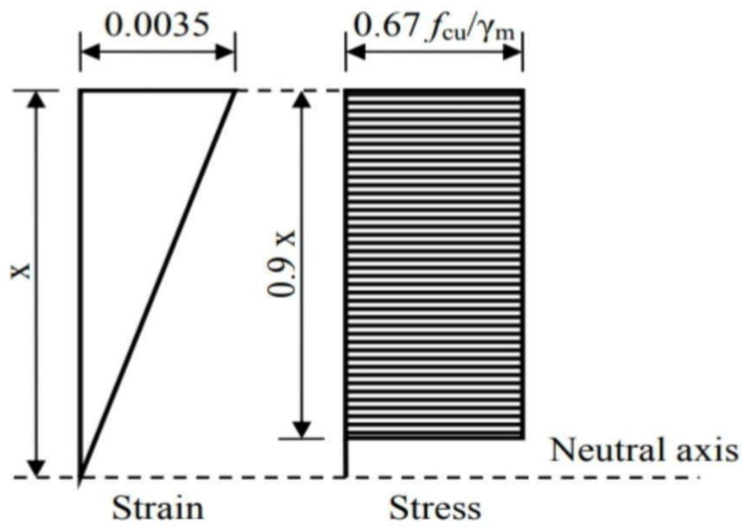


Fig (2.3) simplified stress block for concrete at ultimate state

Ref[19], Turkish code (TS-500), the ultimate compressive strain of RC in this code equal to 0.003. The concrete distribution stress is rectangular block define by $\alpha 1$ that equal 0.85 and K1 as Eq (2-8) below :

K1= 0.85- 0.006(f_{cd}-25) …… (2-8)

the amount of K1 ranging between 0.85 as a maximum value and the minimum value is 0.7 fig (2-4) shown the idealization of stress and strain distribution in a column section.

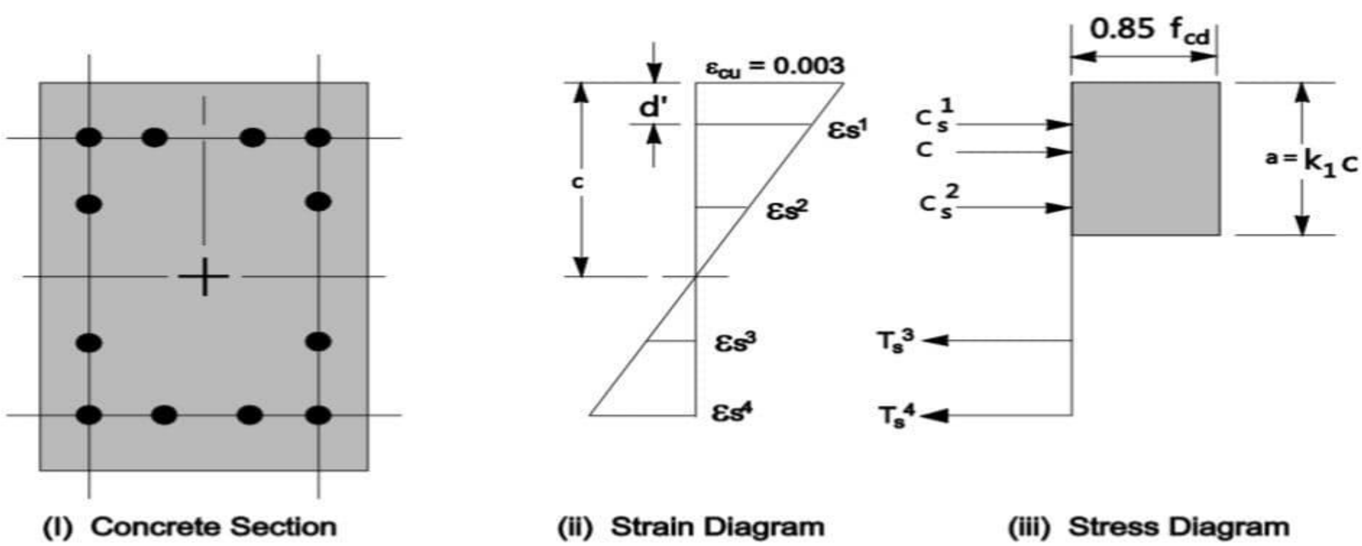
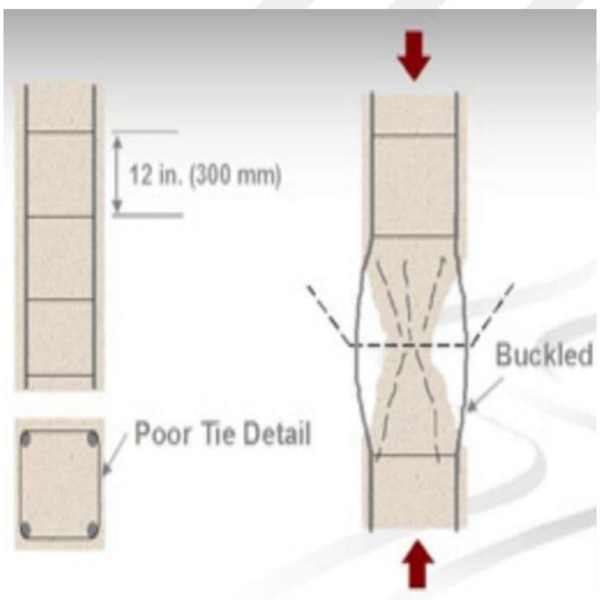


Fig (2-4) Idealization of stress and strain distribution in a column section

2.3 THE LITERATURE SURVEY OF THE SHORT COLUMN

To know the behavior of reinforced concrete, firstly should study the properties of its constituent materials, such as concrete, steel, explain the behavior of concrete such as plasticity and rigidities, also the stress and strain behavior mainly depends on the thickness of the covering and the spacing between the transverse reinforcement. Scientists have proven that transverse reinforcement has a significant effect on improving concrete strength and elasticity, as well as stress and strain relationship has major effects in determining concrete behavior. The longitudinal and transverse bars of each column subjected to compression have an impact on the final bearing capacity[20], in the models (2-5)&(2-6) below a failure that occurred as a result of buckling after the model reached the ultimate strength of the column. According to the current design equations, the ties should be close enough and in sufficient number to avoid premature buckling.



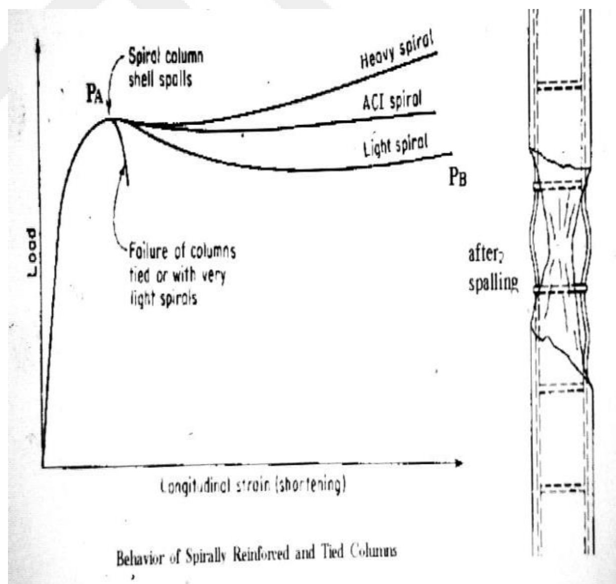


Fig (2-5) & (2-6) failure of tied short column

During previous studies, in order to know the behavior of the concrete confined by transverse reinforcement, there are several factors that must be know it:

a. type and strength of concrete;
b. amount and distributions of longitudinal reinforcement

c. amount, spacing, and configurations of transverse reinforcement;
d. size and shape of confined concrete;
e. ratio of confined area to gross area;
f. strain rate;
g. strain gradient;
h. supplementary crossties;
i. cyclic loading;
j. characteristics of lateral steel;
k. level of axial load in the case of flexural behavior.

In NSC cases, all references have proven that the increased ductility and durability depends on increasing the amount of transverse reinforcement, as columns with single hoops and hooks 90 degrees have insufficient ductility especially in cases of high axial loads and cyclic flexure, that mean the reinforcement use in this case needs for little ductility or limited and low cyclic flexure.

Young et al[21], carried out experiments showing the effects of ductility in HSC subject to compressive axial load. A group of HSC was chosen, confined to a straight line with lateral ties and longitudinal rebar, an improvement in the strain, stress behavior of the ties, the concrete cover and the increase in flexibility have been observed compared with NSC. This proves the eight longitudinal bars, are more flexible than the four-bar, also the failure of all models that were exposed to single shear.

P_o is the axial load capacity under concentric compression given in Eq (2-9)

$$P_o = \alpha 1\ f_c' (A_g - A_{st}) + f_y A_{st} \quad \text{..... (2-9)}$$

Where

A_g = gross area of section

A_{st}= total area of reinforcement bars

f_y= yield strength of reinforcement

Collins et al[22], have suggested the factor α1 in Eq(2-10) below

$$\alpha 1 = 0.6+(10/ f_c') \leq 0.85 \quad \text{..... (2-10)}$$

However, it should be noted that the Eq(2-10) ignored the effect of transverse reinforcement on axial capacity, as opposed to columns that contain little transverse steel, where the results were conservative.

Cheng-Tzu HSU[23], made a suggestion through his experiments to analyze and design twelve short and tied RC models under the influence of biaxial and uniaxial bending together, this equation of failure was written as shown below.

$$\left(\frac{Pn-Pnb}{Po-P_{nb}}\right)+\left(\frac{M_{nx}}{M_{nbx}}\right)^{1.5}+\left(\frac{M_{n}y}{M_{n}by}\right)^{1\cdot 5}=1 \qquad \text{....(2-11)}$$

P_n =nominal axial compression (positive), or tension (negative)

M_{ny}, M_{nx} = nominal bending moment about –y and –x axis

P_o = maximum nominal axial compression (positive), or tension (negative)

P_{nb} = nominal axial compression at balance strain condition

M_{nby}, M_{nbx} = nominal bending moment about –y and –x axis , respectively at balance strain condition

Hsu's method finding the ultimate strengths of RC is simpler compared to other methods, such as reciprocal and contour. Design and calculate the strength of columns affected by axial load and uniaxial or biaxial bending, where the manual calculations are tiring for uniaxial, and biaxial bending, it cannot be manually designed according to basic principles. Tables and charts are used as tools to facilitate, calculations and make them more accurate. Recently, computer programs have been used for the purpose of analysis and design of concrete columns.

J. Y. Richard Yen[24], used the computer to find an efficient and direct method for designing short RC exposed to concentric or eccentric loading. To find the section strength, two variables were introduced, namely the steel ratio (ρ) and the neutral axis direction c. The solutions were found in the shortest time and in reliable ways on any device and on any computer size, as large numbers of RC were designed in seconds. Multivalued repetition techniques are used for the variables and the minimum value of the longitudinal steel. The conclusion was proving that the loads should be less or equal to the resistance of the section and depended on steel ratio and the direction of natural axial c, its shown in equation below:

$P_r = P_n (c , \rho)$ …….. (2-12)

$M_r = M_n (c , \rho)$ …….. (2-13)

M_r , P_r = applied axial, bending moment, respectively

M_n , P_n = axial, bending nominal strength, respectively

ρ = longitudinal steel reinforcement ratio

A_g= gross area of section

$P_n (c , \rho) - P_r = 0$ …….. (2-14)

$M_n (c , \rho) - M_r = 0$ …….. (2-15)

Yen used Newton's law to solve the equations in an efficient manner for the variables c and ρ, with values of 0.8 and 0.01 respectively, h is the thickness of the section in the plane of bending.

2.4 THE LITERATURE SURVEY OF THE SLENDER COLUMN

In any structure, columns are the most important part of the building, so their behavior especially slender greatly affects the efficiency of the structure. Many scientists have developed methods and techniques to increase durability and produce high-efficiency varieties such as HSC, which are considered to be higher strength and improved ductility.

One of the economic benefits of HSC is to produce a smaller design for the same load that the NSC carries, as reducing the size of the column with efficiency leads to space savings (this greatly helps the architect in design and decoration)[25].

During the design and analysis of the frame of slender columns, several problems appeared, including nonlinearity relation resulting from inflexible properties of materials and cracking that occurs as a result of changes in the section, where the equilibrium of the structure is affected due to displacement.

S. A. Mirza[13], presented a statistical evaluation of the constants that affect the flexural stiffness of slender columns, as shown in the equations below:

$EI = (0.27\ E_c\ I_g + E_s\ I_{se}) / 1.7$ …………(2-16 a)

$EI = (0.21\ E_c\ I_g + E_s\ I_{se}) / 1.6$ …………(2-16 b)

$EI = (0.1\ E_c\ I_g + E_s\ \ I_{se}\ ) / 1.5$ ....(2-16 c)

Eq (2-16 a) for columns shall be on lower floors that have more than three floors and a roof, β_d=0.7 . Eq (2-16 b) for columns shall be on intermediate floors that have more than three floors and a roof, β_d=0.6 . Eq (2-16 c) for columns shall be on top floors supporting roof, β_d=0.5 .

$EI = (\alpha\ E_c\ I_g + E_s\ \ I_{se}\ ) / (1+ \beta_d)$ ....(2-17 a)

$\alpha_{\ mirza}= \{0.27 + 0.003\ (L/h) – 0.3\ (e/h)\} \geq 0$ ....(2-17 b)

$\alpha_{\ mirza} = \{0.3 -\ 0.3\ (e/h)\} \geq 0$ ....(2-17 c)

Eq (2-17) more accurate than Eq(2-16).Mirza derived the effective stiffness factor α as :

$EI = (\alpha\ E_c\ I_g + E_s\ \ I_{se}\ )$ ....(2-18)

$\alpha = (\ EI - E_s\ \ I_{se}\ )/(\ E_c\ I_g)$ ....(2-19)

where

EI = effective flexural stiffness of column

E = module of elasticity for (concrete + steel)

I_g = moment of inertia for the gross (concrete + steel)

I_{se}= moment of inertia of steel

α = effective stiffness factor

β_d = creep factor

L_u = unsupported height of column

h = thickness of cross section

e = larger end eccentricity

For development of the theoretical flexural stiffness, eq (2-20) below shown that:

$M_{max} = M\ \sec\ (\ \pi/2\ \sqrt{P_u/P_c}\ )$ ....(2-20)

M_{max} = Maximum bending moment

M = applied end moment

P_u = axial load acting on the column

P_c= Euler bucking strength

Where P_c is effected by EI, its written:

$$EI = \frac{L^2}{4(secsec -1\,(M_{max}/M))-2} \quad \text{...........(2-21)}$$

Duan et al.[26], Proposed an expression to represent the flexural rigidity by finding the moment curvature relationship, used 108 RC columns available in literature. The results prove that the method give a realistic representation of actual column behavior, the modifications is written below:

$$EI = (1+0.3\,\gamma_1)\,(P_n\,(d-e))\;/\;(\propto_D \O_y) \quad \text{...........(2-22)}$$

$$\propto_D = 2 + 0.2\,(e/h) \quad \text{...........(2-23)}$$

$$\O_y = 1/d\,[\,(0.7 + 2.8\,\xi)\,*\,10^{-3} + f_y / E_s\,] \quad \text{...........(2-24)}$$

$$\xi = \beta_1\,C / d = (f_y\,A_s - f_s'\,A_s')\,/\,(0.85\,f_s'\,b\,d) \le \xi_b \quad \text{...........(2-25)}$$

$$\xi_b = (0.003\,\beta_1)\,/\,(0.003 + \epsilon_y) \quad \text{...........(2-26)}$$

$$\gamma_1 = (b_1 - b)\,h_f / (b\,d) \quad \text{...........(2-27)}$$

Where

$\O_y$ = The curvature that occurs in the steel section
$\propto$ = stiffness reduction factor

MacGregor[27], from the results used for the design and analysis of slender columns, adopted the principle of moment magnifier as a second order analysis. One year later, Goyal and Jackson[28], conducted a study showing the effect of sustained load and the amount of force, as the sustained load capacity is equivalent to 50% of the load capacity of short duration, which means a decrease in the value of the force in this case.

As theories continue to evolve new research and data are gather, specifically by, Mirza and MacGregor[29], whom have studied the differences of the short duration ultimate strength of slender tied RC .Where the slender RC is exposed during their study to various variables and characteristics such as the ratio of slenderness, the ratio of deflection, the concrete strength and type, also the eccentricity, the result was the creation of an instantaneous interaction curve of loads and bending moment to find the strength of the section.

A relationship called Hognestad's stress- strain was found in compression, at the first part of curve the highest value of force is described as a second-degree parabola, the lower part of the curve when the force is after the maximum value is will be a straight line. The behavior of steel reinforcement in concrete has been described as in stress-strain plastic elastic is excellently in both cases of compression and tension.

As a result of development and urbanization, slender buildings and slender columns have become popular in use, which makes attention to all matters of stability and solving problems of deflections. the difference in concrete strength values is a major and important factor for slender columns, where the strength of the column is high derived from the strength of the steel in the case of increased eccentricities, but when the value of the eccentricities is small the strength from the steel will be secondary strength.

The slenderness effects on the column causes several problems, including displacement, as due to this displacement the stability the column is disturbed and this is called "member stability effect". Slenderness also causes an erosion called the "lateral drift effect" or "the PA effect" due to the lateral displacement of the slender column relative to the columns below it.

The stability analysis of the RC frames was developed and it is considered a complex process due to the variable reinforcement ratios and due to the non-linear relationships of the deflection, and for these reasons the frames were considered elastic and the effect of the column stability was approximated by the moment magnifier method or use the effective length method.

Therefore, MacGregor[30], proposed to use the Cranston's equations which are based on the effective length and as shown below :

$$K_{ns} = 0.7 + 0.5\,(G_1 + G_2) \leq 1 \qquad \text{......(2-28)}$$

$$K_{ns} = 0.85 + 0.05\,G_s \leq 1 \qquad \text{......(2-29)}$$

Where

K_{ns} = non sway effective length factor

G_1 , G_2 = the ratio of column to beam stiffness

G_s = the smaller of G_1 & G_2

They use this factor to find the maximum moment as below:

$M_{max} = \delta_{ns}\, M_{02} \geq M_{02}$ ……(2-30)

M_{max} = maximum second order moment

M_{02} = large end moment from first order analyses

δ_{ns} = non sway moment magnifier factor

$\delta_{ns} = C_m \frac{1+0.25R_{\cap s}}{1-R_{\cap s}}$ ……(2-31)

C_m = correction factor for unequal moment

R_{ns} = ratio of compression load to the non-sway Euler buckling

Then the (P-Δ) technique was developed, this process will not converge to columns with flexible frames whose axial loads are high. Lateral displacement (Δ_O) is calculated on each story by applying lateral loads and vertical loads to the building, they computed the additional story shears (AV) depending on vertical load (∑P) as shown below:

$AV = (\sum P\, \Delta_O) / L$ ……(2-32)

L = height of the story

According to the reference[30], a value called the stability index (Q) has been proposed and its value is extracted as below:

$Q = (\sum P\, \Delta_O) / (L\, V)$ ……(2-33)

When $Q \leq 0.0475$ during the second-order analysis, those effects are neglected and the structure is designed as a non-sway frame.

for Q between (0.0475 & 0.2), in this case the second order analysis will be recommended.

Also if $Q > 0.2$, the second order analysis in this case should be recommended

Diaz and Roesset [31], carried out long studies and research on slender columns to find out the effect of stability on frame through second-order nonlinear analysis and proved that this analysis depends to a large extent on the theory of large deformation. The numerical results extracted via computer programs were performed and compared with practical results such as the moment magnified ACI code, and the stability index methods.

Through experiments and values obtained from analysis of laboratory models, the Eq (2-34 a & 2-34 b) than Eq (2-35a & 2-35b) below give the best value for Q

$EI_{col} = 0.2\ E_c\ I_g + E_s\ I_{se}$ ………..(2-34 a)

$EI_{beam} = 0.4\ E_c\ I_g$ ………..(2-34b)

$EI_{col} = E_c\ I_g\ (0.2 + 1.2\ \rho_t\ E_s / E_c)$ ………..(2-35a)

$EI_{beam} = 0.5\ E_c\ I_g$ ………..(2-35b)

Where

EI_{col} , EI_{beam} = effective flexural stiffness for (column & beam)

E_c , E_s = modulus of elasticity for (concrete & steel reinforcement)

I_g = moment of inertia of gross section

I_{se} = moment of inertia of reinforcement

ρ_t = total ratio of reinforcement (A_s/A_g)

Rangan[32], introduced a new idea and simpler method for analyzing slender columns by estimating lateral deflection affected by sustained load. A factor directly affecting the effective modulus of elasticity of the column was found to calculate the creep effect and take the moment of inertia I as α_{rangan} I , where α_{rangan} can obtained in equation below :

$EI = \alpha_{rangan}\ E_c\ I_g\ (1 + 0.8\ \emptyset_{cc})$ ………..(2-36)

$\alpha_{rangan} = 0.6\ (e_b + \delta_e) \leq 1$ ………..(2-37)

$\Delta = \frac{P\ l_e^2\ C_M\ e_2}{\pi^2\ EI - P\ le^2}$ ………..(2-38)

Where

$\emptyset_{cc}$= creep factor, the value equal to mean amount (1.8 &3.8) = 2.5

e = eccentricity

e_b= eccentricity at balance failure

Δ = lateral deflection

le = effective length

e_2 = larger eccentricity

C_m = correction factor

P = axial thrust due to sustained load

The resulting values were compared through the laboratory results and the values from the equations the, the ratios showed good correlation, where the mean (calculated values / values through the laboratory) was equal to 1.04 and COV equal to 24%.

Through the above equations for the analysis of slender columns, Rangan[33], provided a method to find the strength of the non-sway framed column, as it relied on the stability analysis method for the pin-ended column. When the column reaches failure due to instability (P_{max}), at given eccentricity, where:

$P_{max} \geq \emptyset P_b$

Where $\emptyset P_b$ is axial load strength at balance failure , also $P_{max} = \emptyset P_n$ & $M_n = M_{max}$.

C_n represent depth of compression zone, this value helped to find k_y factor as below :

$k_y = 0.003/ C_n$ ....(2-39)

$\Delta_y = K_y l^2_e / \pi^2$ ....(2-40)

Δ_y = short term deflection at failure

While $P_{max} < \emptyset P_b$, It is be $P_{max} = \emptyset P_y$, The axial thrust that directly effects on the section when the steel reaches the yield stage.

$M_{max} = \emptyset M_y$, (coexisting bending moment). $\emptyset M_y$ & $\emptyset$, These values are obtained by carefully analyzing the cross-section of the slender column, and assuming equivalent distribution to the resulting concrete stress in the compression zone.

$k_y = \epsilon_y / (d - C_n)$ ....(2-41)

ϵ_y = yield strain of steel

d = effective depth

then

$$\Delta_{cp} = \Delta_{tot} - \Delta_e \qquad \text{………..(2-42)}$$

Δ_{cp} = creep deflection

Δ_{tot} = total deflection due to sustained load

Δ_e = elastic component of the total deflection

$$\Delta_e = \frac{P_o\, l^2_e\, C_m\, e_2}{\lambda\, \pi^2 E_c\, I_g - P_o\, l^2_e} \qquad \text{………..(2-43)}$$

$$\Delta_{tot} = \frac{P_o\, l^2_e\, C_m\, e_2}{\pi^2 EI - P_o\, l^2_e} \qquad \text{………..(2-44)}$$

$$P_{max} = M_{max} / (e + \Delta_{cp} + \Delta_y) \qquad \text{………..(2-45)}$$

Bonet and Romero[34] , A study was presented on 613 columns (HSC, NSC) experimental test available in the literature, based on effective flexural stiffness to obtain the best value of bearing capacity of slender columns. Equation is good for rectangular sections subject to short time and continuous loads, the results were compared to backward codes such as (ACI-318-2008 and Euro Code–2-2004). The flexural rigidity and the factor of stiffness can obtain in equation below:

$$EI = \alpha \cdot (E_{cd} \cdot I_{ce}) / (1 + \phi_{eff}) + (E_s \cdot I_{se}) / (1 + \xi\phi) \qquad \text{………..(2-46)}$$

Where

Φ_{eff} = creep coefficient

$$\alpha = (1.95 - 0.035 \cdot \lambda_m - 0.25 \cdot \phi_{eff}) \cdot (\eta - 0.2) + (f_{cd}/225 + 0.11) \qquad \text{…………..(2-47)}$$

$$\eta = e_0 / 4 \cdot i_c \qquad \text{………..(2-48)}$$

$$\eta = M_d / N_d \cdot 4 \cdot i_c \qquad \text{for } \eta < 0.2\text{:)} \qquad \text{………..(2-49)}$$

Where

η = the first order relative eccentricity

N_d = design axial load

M_d = the vector modulus of the first order bending moment

i_c = critical radius of gyration

The reduction factor of the stiffness of the reinforcement $\xi\phi$ is given by equation below:

$$\xi\phi = 1.9 \cdot \phi_{eff} \cdot \exp(-\lambda_m/25) \qquad \text{...........(2-50)}$$

$$\lambda_m = = l_p/i_c\ ) \qquad \text{...........(2-51)}$$

Where

α = the effective stiffness factor

λ_m = mechanical slenderness of the column

l_p = effective buckling length

Tikka K. and Mirza[35], Adopt the moment magnifier approach to designing RC slender columns. Where it is directly affected by effective flexural stiffness (EI). 27,000 columns with variable properties were laboratory tested. Develop two new equations to calculate flexural rigidity factor and effective rigidity factor as equations below:

$$EI = (\alpha\ E_c\ (I_g - I_{ss} - I_{rs}\ ) / (1+ \beta_d) + 0.85\ E_s\ (I_{ss}+ I_{rs}) \qquad \text{...........(2-52)}$$

$$\alpha = \{0.48 - 3.5\ e/h\ [1/1+\beta(e/h)] + 0.002\ (L/h)\ \} \geq 0 \qquad \text{...........(2-53)}$$

Where

β = 7 for RC columns when ρ<2% , β = 8 for RC columns when ρ>2%, β =9.5 for composite columns.

I_{ss} = moment of inertia of structure steel

I_{rs} = moment of inertia of longitudinal reinforce steel

I_g = moment of inertia of gross section

To reach the acceptable level of the resistance or load value and to reduce the possibility of the section being exposed to an overload or understrength, the ACI code specifies the resistance factors of the column in certain ways, including:

a) The resistance factor applied to cross-section strength.
b) The resistance factor applied to the buckling strength that occurs as a result of slenderness that used in moment magnification.
c) In order to arrive at the design of the resistance factors reliably, a constant value of the stability and safety factor was found, which ranges between 0.7 and 0.75, and was used in the calculations of the moment magnification of the slender RC. A value of 0.75 is the safest amount used for all column cases.

3. THEORETICAL BACKGROUND

3.1 GENERAL OVERVIEW

The column is a structural element that is subjected to axial force. Usually, the height of the column is greater than the cross-sectional dimensions. There are many types of columns such as short columns and slender columns, which are classified according to the factors affecting the body, types of reinforcement that are used in it, and slenderness.

In designing the structural members, the compressive strength of the columns is the criterion of quality, so the performance of the reinforced concrete body under imposed loads depends on the stress-strain curve (the relations between concrete and steel), also the type of loads applied to the column. The nonlinear relationship that happens in the reinforced columns on the stress-strain curve that changed due to the loads, so that the stress block distribution will be rectangular, trapezoidal or triangular, etc.

In this research, a study was made to identify the stability of reinforced concrete slender column using a non-linear stress-strain relation for concrete with all effects of cracking, creep, and shrinkage, through computer software programs whereas detailed analysis procedures were calculated with high precision.

Columns will be subjected to deflection along the length of the column, and that deflections appear on the column as additional secondary bending moments. When the materials of column failure reached this point under the combined action of loads and bending moments, the building will fail. Most countries codes are based on the moment magnifying method to analyze slender columns.

Each country has special codes that are applied in civil works, yet some civil engineers in certain countries don't use the specific design codes in their respective countries. They select comparable consulting codes from other countries to obtain desired results. The need for comparison analysis and persisted follow-up on scientific developments can convey excellent results.

3.2 SHORT COLUMN

A concrete structure with a thin cross-section. The body resists the force imposed on a given eccentricity {e} through the resistance of the materials it is formed from. In analyzing this type of column the ultimate bearing capacity is not affected by length.

The column exposed to axial force {P} and bending moment {M} applied at eccentricity {e = M/P} shown in fig (3-1) below

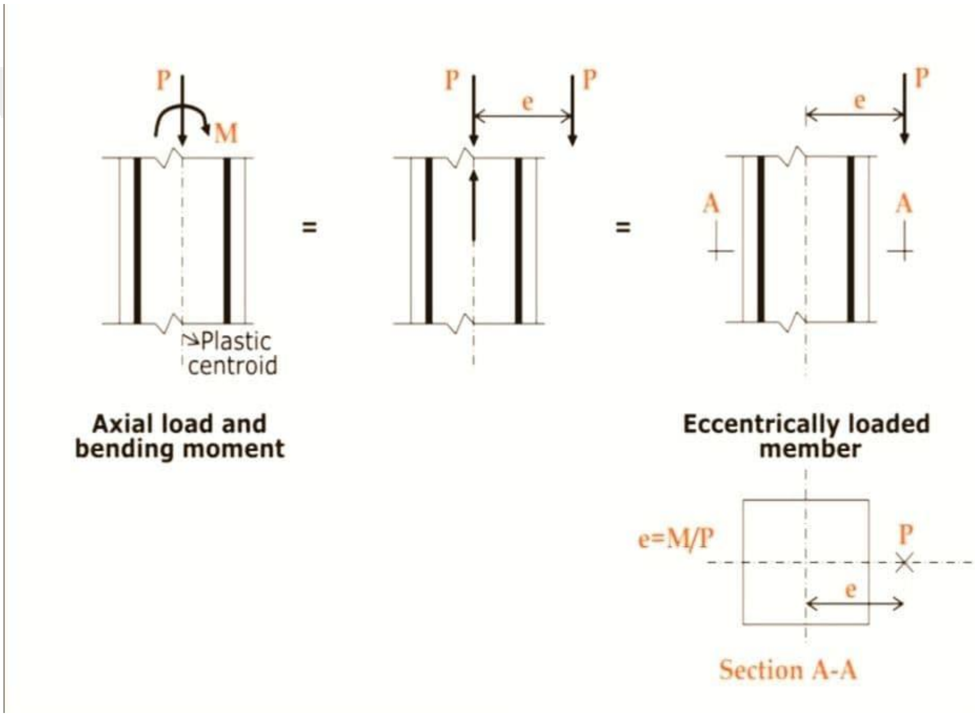
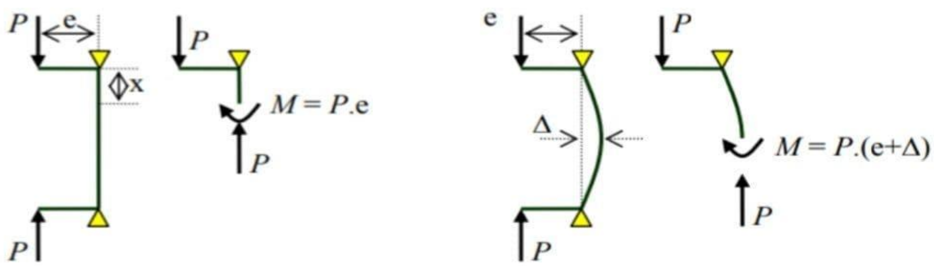


Fig (3-1) Equivalent eccentric column loading

Depending on the relations of strain and stress of concrete, the internal analytical forces of the column can be calculated by finding the neutral axis and determining the point C which represents the maximum compression distance depending on the amount of eccentricity obtained the compressive strength of concrete, tensile strength of steel, and the durability of concrete through mathematical equations and the theory of right and wrong.

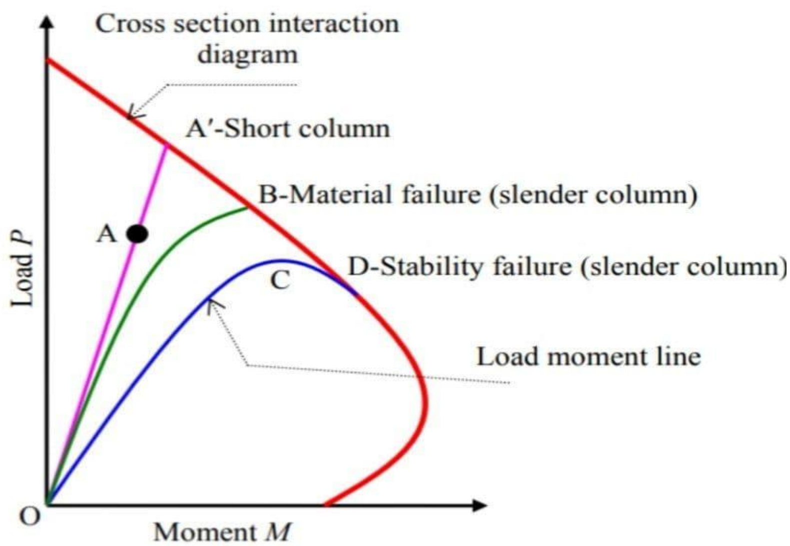
The short column is the column in which the final loads are not reduced through bending deformations because the additional eccentricity is little or far from the critical location. In any column for every amount value of eccentricity of loading, there is a particular amount of (cross-section resistance) P & M, these values are affected by the main axis specified for the value of C which is changed to acquire the necessary eccentricity value.

In this case, infinite values are found for the column strengths (P & M) by changing the eccentricity values from zero to infinity, and that is by linking all the values on a graphic design which includes two axes horizontal and vertical, where the values of M on the horizontal axis and the values of P are on the vertical axis, a closed curve is formed called (the interaction diagram) as shown in the fig (3-2c) below. This closed curve is used to design and analyze columns. Line O-A explains the relation between loads and moments as in (3-2a). At point A (Column develops to material failure) represented by increased axial loads up to the point on the intersection of the interaction diagram.



a-Short column under load

b-Slender column under load



c-Load and moments at mid-height for short and slender columns

Fig (3-2) The behavior of column under loading

3.3 SLENDER COLUMN

The slender column is one of the columns whose final bearing capacity is reduced due to the slenderness which leads to an increase in bending. The value of the deflections that occurred due to the bending depends on the end conditions and the type of impressed load. Analysis of slender column is more complex than short column, due to the additional bending that occurs.

A straight column is subjected to a load of P from both ends, resulting in a bending deformation whose total value is equal to (e + Δ), e represents the eccentricity, and Δ represents the additional eccentricity that occurs as a result of the slenderness of the column. The (P Δ) is called the secondary moment value, it is the reason for the increase in the bending moment amount Fig (3-2b). This expression indicates that the moment is of secondary importance, although in other situations it may be more significantly important.

At present, the classic stability theory is adopted for slender concrete column designs. almost all codes of practice require design for final loads but it is recommended that a flexible method of frame analysis to use. Upon failure, two important states of the slender column are formed; the first state is the curve (O- B) fig (3-2c). This failure occurs when the curve intersects the interaction diagram, leading to a material failure condition, and because of the effect of slenderness, the axial load decreases from A to B. The second state of failure is when the columns are very slender, reaching the stage of instability and the failure reaches its maximum value, point B reach to point C fig (3-2c) and (dM/dP) = zero.

For example, in the case of the short column in a braced frame due to low stiffness, the moments at the ends of the column decrease more than the increase in the moments due to the deflection, meaning the load capacity increases due to the decrease in the maximum moment. Whereas, in the case of the slender column in a braced frame the bearing capacity decreases due to the increase in moment because it is greater than the resistance moment.

Many factors affecting on the strength of slender column, which are as follows:

a. The ratio between eccentricity and height (e/h), the ratio between unsupported height and section depth (L_u/h), these percentages are directly affected.
b. The rate of rotation of the end restriction, to increase column strength, the connected beam system must be stiffer.
c. The rate of lateral restraint, the braced column is more stable than the un-braced column against end displacement.
d. The amount of reinforcement and the quality of the materials used in the column affect the strength and rigidity of the column.
e. Sustained loading on the column leads to an increased column deflection; this reduces the rigidity of the column.

3.4 PRINCIPLES OF STATICS FOR THE CROSS-SECTION

In this work, there are two distribution shapes of stress block of reinforced concrete, rectangular and triangular shape. Below will explain the deferent between the two shape.

3.4.1 RECTANGULAR STRESS BLOCK

It's defined by two factors (α1,β1) in the stress block analysis. $\alpha 1$ means the factor of compression stress block intensity, $\beta 1$ means the ratio between the depth of compression stress block to the depth of natural axis C. These values mainly depend on (fc′) compressive strength of concrete. In some codes these values are constant or formula, see fig (3-3c)

Where:

P_c = plastic centroid of cross-section column

f_c'= specified compressive strength of concrete

A_s = area of tension enforcement

A'_s = area of compression enforcement

d′ = distance from compression fiber to centroid of compression reinforcement

d = effective depth of section,

h = overall thickness of the cross-section

b = width of compression of the cross-section

ε_s = strain in reinforcement corresponding& compression respectively,

ε_{cu} = ultimate compressive strength

When the compressive strength of concrete is rectangular it's equal to:

$$C_c = b \cdot \alpha 1 \cdot f_c' \cdot \beta 1 \cdot c \qquad \text{...... (3-1)}$$

But the amount of Eq (3-1) shouldn't be greater than the amount of Eq (3-2)

$$Cc = b \cdot \alpha 1 \cdot f_c' \cdot h \qquad \text{...... (3-2)}$$

3.4.2 TRIANGULAR STRESS BLOCK

This type of stress block shows the compressive strength of concrete as triangular block fig (3-3d) shown that, and it is equal to:

$$Cc = \tfrac{1}{2} \cdot b \cdot \alpha 1 \cdot f_c' \cdot \beta 1 \cdot c \qquad \text{...... (3-3)}$$

But also the amount of Eq (3-3) shouldn't be greater than the amount of Eq (3-4) :

$$Cc = \tfrac{1}{2} \cdot b \cdot h \cdot \alpha 1 \cdot f_c'(2\ \beta 1\ c - h)/\beta 1 \cdot c \qquad \text{...... (3-4)}$$

This shape was created due to the increasing use of HSC in construction[36], where it is considered cost-effective, bearing the loads as well as the small size of the column.

Several studies were conducted about that shape where concerns were raised that safety factors and design methods for this type could not be extrapolated[36].

Fig (3-3) shows the relation between stress-strain for HSC, reaching 90 percent of the maximum stress[37].

In general, the force and moment (P & M) values of the cross-section should be calculated as follows:

$P= C_c + \Sigma^n A_{st} (f_s - f_c)$ …… (3-5a)

$M = Cc$. lever arm to plastic centroid + $\sum A_{st}(f_s-f_c)$* lever arm to plastic centroid …… (3-5b)

Where

N= the number of steel layers

A_{si}= area of steel in the i^{th} level

f_{si}= stress of the reinforcing steel i^{th} layer corresponding to the strain distribution

f_s= specified strength of reinforcing steel

f_{ci}= stress of concrete in i^{th} position corresponding to the strain distribution

i^{th}= reinforcement steel layer

a. if f_{si} is (negative value) the force of reinforcing steel $=A_{si}*f_{si}$
b. if ($f_{si}-f_{ci}$) is (negative value), it shall be neglected and the reinforcing steel force in that position.

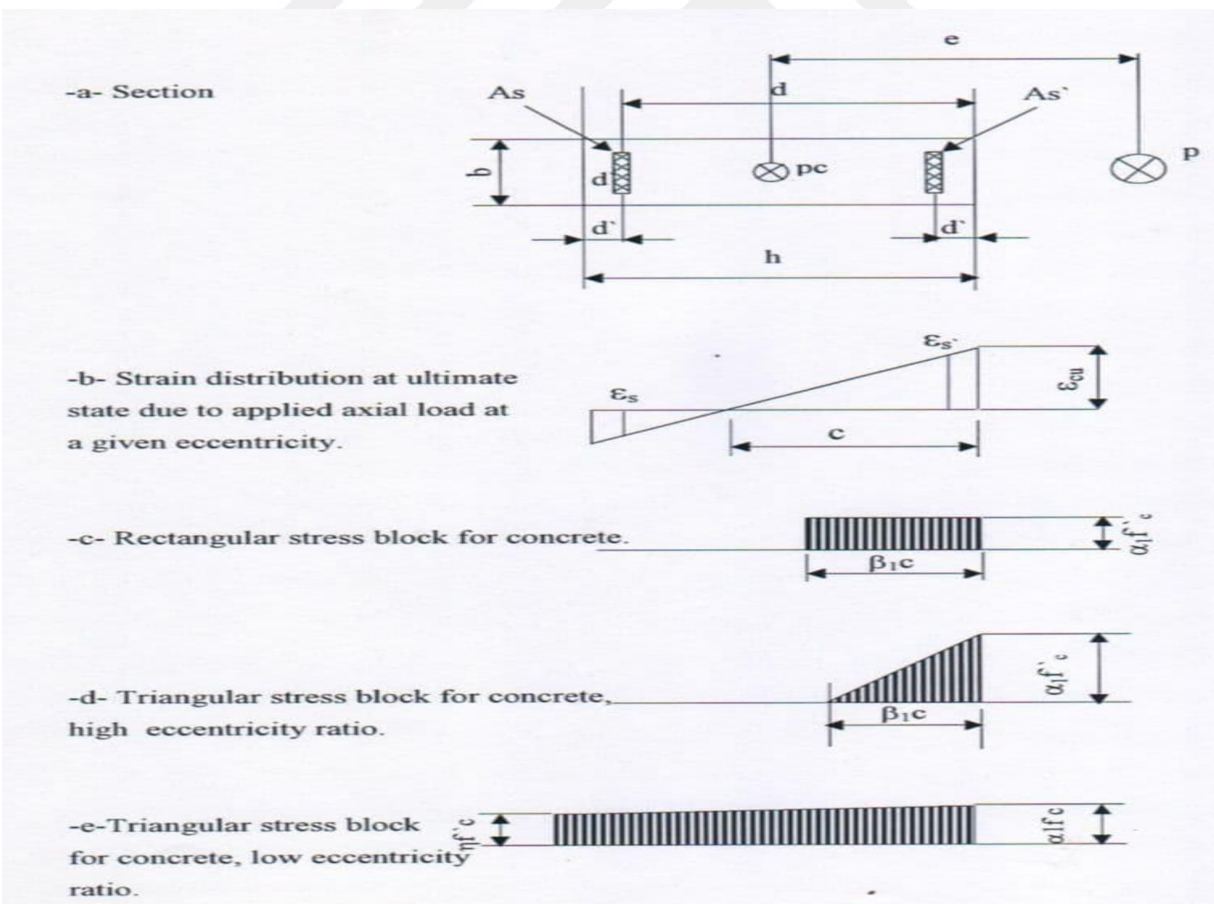


Fig (3-3) Eccentrically loaded column section at the ultimate load

3.5 DESIGN AND ANALYSIS METHOD

At present time, in the design procedure of RC columns, several modern codes have been used to idealized case of elastic, hinged ends bent in symmetrical single curvature. As a result of the magnitude of second-order moments that occurs due to the deflection of the column or the lateral drift of the structure as a whole.

The type of column will be determined as short or slender, different types of codes with different symbols have been used to deal with both types of columns. The relation of stress-strain is the most important influence that determines column analysis and design, as well as the shape of the stress block.

The current design and the proposed method (describe in chapter four) are based on these assumptions:

a. Ignore the tensile strength of concrete.
b. Ignore the contribution of confinement on strength.
c. The stress in reinforced steel (f_s) shall be equal to or greater than the specified yield strength of reinforcement (f_y). The stress-strain relation for reinforcing steel is elastic perfectly plastic.
d. It is assumed, the perfect bending exists between steel and concrete at the surface, so no slip occurs between the two materials.
e. The strain will be zero in the reinforcing and concrete where directly proportional to the distance from the natural axis. the stress equals the specified strain multiplied by the modulus of elasticity of steel.

In this chapter, three types of codes will explain how to analyze the RC slender column.

3.5.1 ACI318M- 19 CODE

This refers to the method used by reference [38]

The values of this code deal precisely with the nominal strength of the cross-section and with the force reduction factors. The concrete distribution stress in this code is rectangular define by $\alpha 1$ that equal 0.85 and the value of $\beta 1$ equal 0.85 when the concrete strength value reaches 30MPa, and for concrete strength above 30MPa it should be continuously decreased in the range of 0.05 per 7 MPa from the strength, in this situation the $\beta 1$ should not take a value less than 0.65 and its appearance in Eq (3-6) below:

$\beta 1=0.85-(fc' - 28)/140))$ $0.65 \leq \beta 1 \leq 0.85$ …… (3-6)

The strain at extreme concrete compression εcu equal to 0.003, the strength reduction factor

(Ø) = 0.7, 0.75 for (tied & spiral column)[39].

This code uses an approximate design method that relies mainly on the moment magnifier principle.

3.5.2.1 MAGNIFIED MOMENT NON-SWAY FRAMES

$M_c=\delta ns * M2$ …… (3-7)

$\delta ns= C_m / \{1- (P_u/0.75)/P_c\}$ …… (3-8)

$C_m = 0.6+0.4(M1/M2) \geq 0.4$ …… (3-9)

$P_c = (\pi^2 EI)/ (K_{ns} . L_u)$ …… (3-10)

The stability factor in this case is a constant value, it is equal to 0.75.

3.5.2.2 MAGNIFIED MOMENT SWAY frames

$M_1=M_{1s}+ \delta s * M_{1s}$ …… (3-11)

$M_2=M_{2s}+ \delta s * M_{2s}$ …… (3-12)

$\delta_s = 1/\{ 1 - (\sum P_u /\emptyset)/\sum P_c\}$ …… (3-13)

$M_c = \delta s * M_2$ …… (3-14)

$M_{max} = \delta ns * Mns + \delta s * Ms$ …… (3-15)

Where

M_{max}= maximum second-order moment

M_1= smaller end moment on compression member

M_2=larger end moment on compression member

C_m=equivalent moment factor

P_c=elastic Euler critical load

Also, the effect of slenderness has a great importance in the analysis and design of slender column, it can be shown as below:

$KL_u/r > 34-12*M1/M2$ for non-sway case …… (3-16)

$KL_u/r > 22$ for sway case …… (3-17)

Where

K = effective length factor

L_u= unsupported length

r = radius of gyration

To estimate the value of L_u should use Jakson and Moreland alignment chart fig (3-5). The flexural stiffness (EI) is estimated by using second-order frame analysis which the modulus of elasticity $(E_c)= 4700\sqrt{fc'}$ in MPa also the moment of inertia of it is as below :

beam = $0.35I_g$,

column = $0.7I_g$

flat plate = $0.25I_g$

$EI= (\ . E_c \ . I_g + E_s \ . I_s)/ (1+\beta_d)$ …… (3-18)

Where

β_d = creep factor (ratio of max. dead load to max. total load)

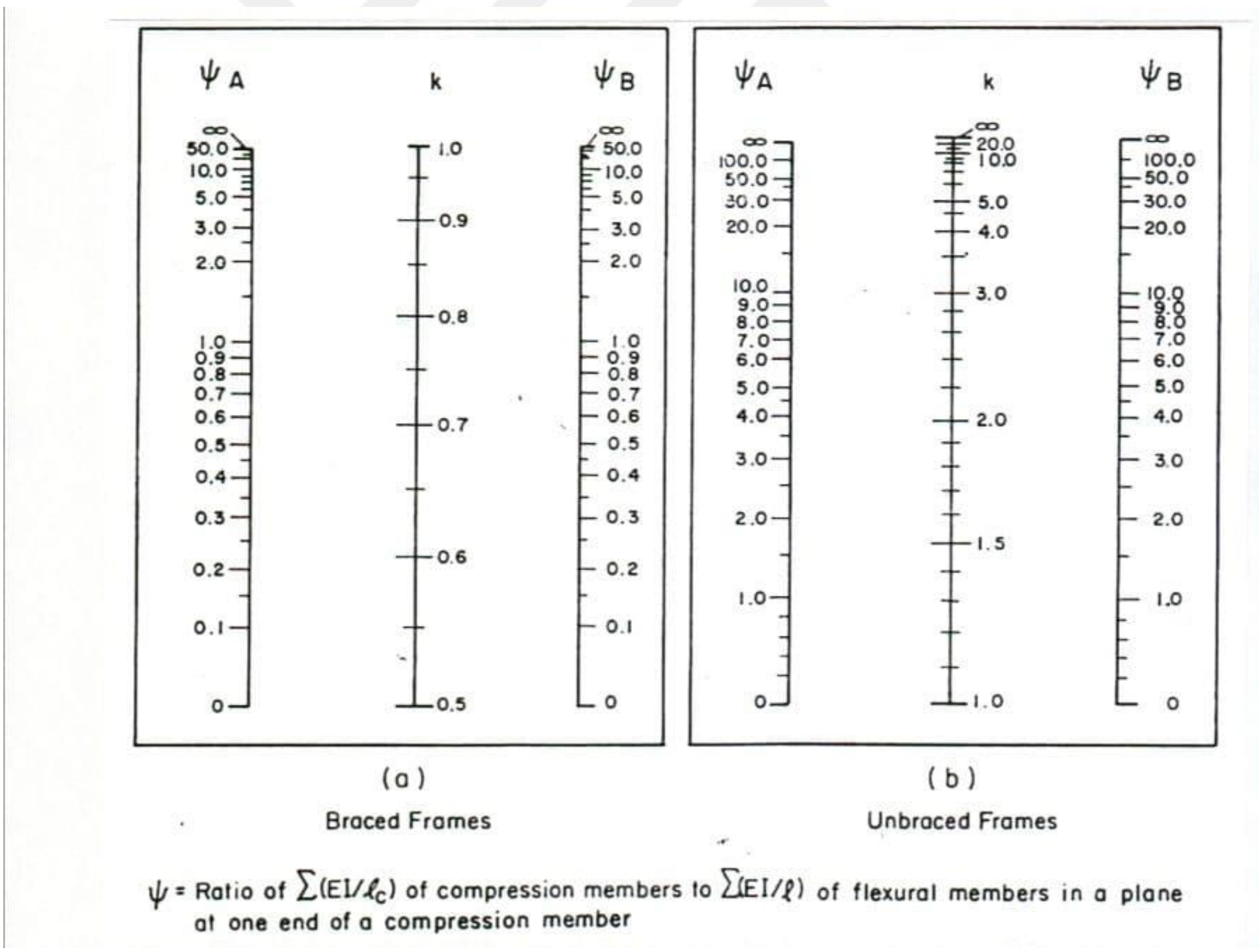
I_g = moment of inertia of gross concrete section

I_s = moment of inertia of reinforcement

E_s= modulus of elasticity of reinforcement

α = the flexural stiffness factor (= 0.2)

This status is a bit conservative since the summation of the two maximum moments must be greater or equal to the actual maximum moment, also it may be over-conservative when the two maximum moments occur in different parts and the two amounts are comparable in magnitude.



Jakson and Moreland alignment chart (3-1)

3.5.2 BS8110 CODE

This refers to the method applied by reference [40]

The concrete distribution stress in this code is rectangular define by $\alpha 1$that equal 0.67 and β1 equal 0.9 respectively.

In this code, the specified cube strength of concrete f_{cu}{cylinder strength between (70-90)% of the cube strength, so if f_c' =0.788 f_{cu} the compression block stress $\alpha 1$ equal 0.85 fc' }.

The strain concrete compression equal to 0.0035, the reduction factor of concrete {$1/\gamma_m$=1/1.5} and for reinforcement {$1/\gamma_m$=1/1.15}

This code is based on the additional moment approach; the approach is affected the curvature of the column because the columns in structures may be will elastic or homogeneous under loads.

To determine the effective length factors into case braced column and sway columns the tables (3-1) below:

Table (3-1a) Values of effective length factor for braced columns

	end condition at bottom		
end condition at top	Fixed	Partially restrained	Pinned
Fixed	0.75	0.8	0.9
Partially restrained	0.8	0.85	0.95
Pinned	0.9	0.95	1

Table (3-1b) Values of effective length factor for sway columns

	end condition at bottom		
end condition at top	Fixed	Partially restrained	Pinned
Fixed	1.2	1.3	1.6
Partially restrained	1.3	1.5	1.8
Pinned	1.6	1.8	-
Free	2.2	-	-

Table (3-1) values of effective length factor

$\frac{le}{h}$(of either axis) > 15 for non-sway frame …… (3-19)

$\frac{le}{h}$(of either axis) >10 for sway frame …… (3-20)

Where

$L_e = \beta\ L_o$

L_e = effective length of column

L_o = unsupported length of column

h = overall thickness of column

The columns in this code should be designed for its ultimate axial load (N) {axial load at given eccentricity), together with the total amount of moment (M_t), which is given by the following equation:

M_t=M_i+ M_{add} …… (3-21)

M_t=M_2 …… (3-22)

M_t=M_l+ 0.5 M_{add} …… (3-23)

M_t=Ne_{min} …… (3-24)

M_i=0.4 M_l+0.6 $M_2 \geq 0.4\ M_2$ …… (3-25)

Where,
M_i = initial moment
M_1 = smaller end moment on compression member
M_2 = larger end moment on compression member
M_{add} = additional moment

$M_{add} = N\beta_a\ k\ h$ …… (3-26)

β_a= 1/2000 ($\frac{le}{b'}$)^2 …… (3-27)

b' = smaller dimension of the column
k = an optional reduction factor

k = $\frac{Nuz-N}{Nuz-Nbal}$ ≤1 …… (3-28)

Where

N_{uz}= capacity of the column under axial load

N_{bal} = axial load corresponding to the balance condition

e_{min} = design minimum eccentricity that taken 0.5h or 20mm whichever is smaller

3.5.3 TS 500 CODE

This refers to the method applied by reference [19]

The concrete distribution stress in this code is rectangular define by $\alpha 1$that equal 0.85 and K1 (β1) as Eq (3-29) below:

k1= 0.85- 0.006(f_{cd}-25) $0.70 \leq k1 \leq 0.85$ …… (3-29)

Where
k1 = The ratio of the depth of the stress block to the depth of neutral axis
f_{cd} = Design compression strength of concrete

The analysis of the slender columns depends on the dimensioning of the members under the axial compression and bending, that were extracted from the second-order analysis, taking into account the nonlinear material and the behavior of the concrete as a result of the influences such as cracking, creep and shrinkage.

The method used for the analysis of the column is the moment magnification method (approximate method), when the section dimensions and axial load differ from their height. The magnification factor is used to know the state of the effect, if the structure or wall has stiffness against the horizontal forces, the status of non-sway can be assumed, and if the end moments of the column resulting from the second-order analysis differ by 5 percent from the moments at the end of the column calculated from the first-order analysis, non-sway can be assumed also.

$$\emptyset = 1.5\ \Delta_I \frac{\sum \frac{Nd1}{li}}{v_{F \cdot 1}} \leq 0.05 \qquad \text{...... (3-30)}$$

Where

$\emptyset$ = magnification factor

N_d =axial design load

V_f =shear force for i^{th} story

L_i = i^{th} story column length

Δ_I =drift at i^{th} story

The values that use in eq (3-30) should be calculated using un-cracked sections, can be computations by the equations below:

$$F_d = 1.0\ G + 1.0\ Q + 1.0\ E \qquad \text{...... (3-31)}$$

And

$$F_d = 1.0\ G + 1.3\ Q + 1.3\ W \qquad \text{...... (3-32)}$$

Where

F_d = design load effect

G = permanent load effect

Q = live load effect

W = wind effect

E = seismic load effect

The calculation of the effective length is done in cases, if a drop panel or capital exists is found, the effective length is calculated from the face of the drop panel or capital exists. In the case of accurate analysis, the effective length is calculated by multiplying the clear length by the factor K that is calculated by the Eq(3-33) below.

a. For column non-sway frames:

$$K = 0.7 + 0.05\,(\alpha 1 + \alpha 2) \leq (0.85 + 0.05\,\alpha 1) \leq 1.0 \qquad \text{...... (3-33)}$$

b. For column sway frames:

If $\alpha_m < 2$ $\quad K = \frac{20-\alpha_m}{20}\sqrt{1+\alpha_m}$ ...... (3-34)

If $\alpha_m \geq 2$ $\quad K = 0.9\sqrt{1+\alpha_m}$ ...... (3-35)

When sway frame K= 2+0.3α with one end hinged columns, α should calculated at the end (not hinged)

Where

αm = average of $\alpha 1$and $\alpha 2$ ratio

$\alpha 1$& $\alpha 2$ = column end restraint coefficient

$R_m = \frac{\sum Vgd}{\sum V_d}$ (for whole story) ...... (3-36)

Where

R_m = creep coefficient

V_{gd} = permanent load contribution to axial design force

V_d = design shear force

The moment magnification coefficient (β) shall be calculated by two cases shown below:

a. if the column is non-sway frame:

$$\beta = \frac{C_m}{1-1.3\frac{Nd}{N_K}} \geq 1 \qquad \text{...... (3-37)}$$

When the column is bent in single curvature, the ratio of M_1/M_2 is positive, and if the column is bent in double curvature, the ratio of M_1/M_2 is negative.

$C_m = 0.6 + 0.4\left(\frac{M_1}{M_2}\right) \geq 0.4$; $M_1 \leq M_2$ …… (3-38)

Where

C_m = moment coefficient in case of buckling

M_1 = smaller end moment on compression member

M_2 = larger end moment on compression member

If any horizontal load affected between the column ends ($C_m = 1$)

The design moment (M_d) is given in Eq(3-39) below:

$M_d = \beta\ M_2$ …… (3-39)

b. if the column is sway frame:

$\beta s = \frac{1}{1-\sum\frac{Nd}{N_K}} \geq 1$ …… (3-40)

Where

N_d = axial design load

N_k = column buckling load

$\sum N_d \leq 0.45 \sum N_k$ …… (3-41)

In the columns in sway frames, the value of β should be calculated, in calculating β (Cm should equal to 1.0). for β and βs, the larger values of (βM_2 and $\beta_s M_2$) should be used in calculating the design moment. So the slenderness ratio is:

$\frac{lk}{i} > \frac{35}{\sqrt{\frac{Nd}{f_{ck}Ac}}}$ ……(3-42)

$Md = \beta\ \ \beta s\ \ M_2$ ……(3-43)

3.6 SUMMARY

The expression "short column" is used for a column whose strength is equal to or greater than the cross-section strength using the force and moment obtained from the analysis procedure and assumptions for bending and axial load. The expression "slender column" represents a column whose strength is reduced by second-order deformations that occur.

Although the designer has determined the steps used to analyze the column, except that it does not appears until it starts the mathematical calculations.

Generally, at present time, the design procedures for hinged end concrete columns are based on elastic analysis applicable to single curvature.

Approximation methods have been introduced in the analysis of concrete columns and column frames, these approximations include selecting the EI for the concrete section, as well as treatment of creep, the use of elastic effective length factor to approximate the frame behavior.

4. ANALYSIS DATA AND RESULTS

4.1 GENERAL

In order to analyze concrete columns in an acceptable and logical manner, it is necessary to study the behavior of the columns in different conditions and examine the influencing factors. In this work, the laboratory results will be studied and compared with the analytical results.

Columns that are studied at different cases, some of them are under the influence of uniaxial bending, short or slender columns sway or non-sway frame, as well as NSC & HSC.

The characteristics of the experimental results used in the work varied, such as the cross-section dimensions, the slenderness ratio of HSC, the longitudinal reinforcement ratio, the eccentricity ratio, the compressive strength of concrete and the yield strength of the longitudinal reinforced concrete.

As a result of the development in the scientific computer programs used, several programs have been used to analyze the models in different data and determine the details of each column.

4.2 SPECIFICATIONS OF THE EXPERIMENTAL RESULTS

For the available 168 RC rectangular columns experimentally tested under short-time loads [23][15][41] [42][43] and [44]. 19 of these columns are short and the remainders are slender columns. Table (4-1) shows the detail and test result of all columns.

During the column analysis process, the lateral deflection of each column was obtained, where an extensive regression analysis process is performed to find out the main factors affecting on the bearing capacity of slender columns.

In this work, 12,096 case studies have special specifications listed in Table (4-2). The principle of moment magnifier approach was used to analyze the columns, where some amendment or improvement to the methods of analysis and design of columns were made. In this curriculum, the effective length factor K, the equivalent uniform moment diagram factor C_m and the sustained load factorβ_d will be constant values for all columns. Fig (4-1) shows during the test that these values do not affect the derived expressions later. As the studied hypothetical model shown in the figure is exposed to the effect of loads for a short period, as the symmetry in geometric materials is about the middle of the height of RC column.

Table (4 - 1) Details of experimental columns

No	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As total (mm^2)	d1 (mm)	P test (kN)	eo applied (mm)	Lu/h	b/h	Type	ρ	Ref
1	35.16	300.62	254	254	254	3096.768	38.1	391.44	317.5	1	1	SH	0.048	23
2	25.51	368.88	203.2	152.4	152.4	798.966	44.45	106.757	142.5	1	1.33333	SH	0.025799995	
3	26.89	368.88	203.2	152.4	152.4	798.966	44.45	266.892	76.2	1	1.33333	SH	0.025799995	
4	25.51	368.88	152.4	203.2	203.2	798.966	44.45	311.374	101.6	1	0.75	SH	0.025799995	
5	31.72	368.88	152.4	203.2	203.2	798.966	44.45	142.342	203.2	1	0.75	SH	0.025799995	
6	23.62	306.83	101.6	101.6	101.6	283.87	23.876	28.67	127	1	1	SH	0.027499961	
7	23.62	306.83	101.6	101.6	101.6	283.87	23.876	52.98	76.4	1	1	SH	0.027499961	
8	20.69	275.8	152.4	152.4	152.4	1029.4370	25.4	320.27	50.8	1	1	SH	0.044323074	
9	23.44	275.8	152.4	152.4	152.4	1029.4370	25.4	355.86	50.8	1	1	SH	0.044323074	
10	28.96	275.8	152.4	152.4	152.4	1608.4954	25.4	444.82	50.88	1	1	SH	0.069254803	
11	23.44	275.8	152.4	152.4	152.4	1608.4954	25.4	471.51	50.8	1	1	SH	0.069254803	
12	31.94	493.68	127	127	127	516.128	25.4	346.96	26.28	1	1	SH	0.032	
13	31.94	493.68	127	127	127	516.128	25.4	362.53	26.924	1	1	SH	0.032	
14	37.07	493.68	127	127	127	516.128	25.4	188.16	71.25	1	1	SH	0.032	

no	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	appliedLu/h	b/h	type	ρ	
15	37.07	493.68	127	127	127	516.128	25.4	204.62	69.215	1	1	SH	0.032	
16	37.07	493.68	127	127	127	516.128	25.4	104.98	133.35	1	1	SH	0.032	
17	37.07	493.68	127	127	127	516.128	25.4	105.42	133.096	1	1	SH	0.032	
18	25.28	493.68	127	127	127	516.128	25.4	189.49	69.342	1	1	SH	0.032	23
19	25.28	493.68	127	127	127	516.128	25.4	87.63	133.985	1	1	SH	0.032	
20	16.27	390.95	152.4	101.6	508	283.87	19.05	213.51	21.69	5	1.5	C.S.C	0.01833330	
21	18.89	399.91	152.4	101.6	508	283.87	19.05	235.75	21.7	5	1.5	C.S.C	0.01833330	
22	25.99	384.74	152.4	101.6	508	283.87	19.05	298.03	21.61	5	1.5	C.S.C	0.01833330	
23	38.54	401.29	152.4	101.6	508	283.87	19.05	385.21	27	5	1.5	C.S.C	0.01833330	
24	27.03	399.22	152.4	101.6	508	283.87	19.05	346.96	20.2	5	1.5	C.S.C	0.01833330	
25	36.34	466.79	152.4	101.6	508	283.87	19.05	394.56	22.43	5	1.5	C.S.C	0.01833330	15
26	41.3	537.81	152.4	80.962	508	169.65	17.27	326.94	16.99	6.27451	1.882	C.S.C	0.01374943	
27	36.44	505.4	152.4	127	508	283.87	19.05	491.53	27.22	4	1.2	C.S.C	0.01466664	
28	58	430	175	175	1500	660	25	1476	15	8.571429	1	S.S	0.02155102	
29	58	430	175	175	1500	660	25	830	50	8.571429	1	S.S	0.02155102	41
30	58	430	175	175	1500	660	25	660	65	8.571429	1	S.S	0.02155102	

No	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	Lu/h	b/h	type	ρ	Ref
31	58	430	300	100	1500	660	25	1192	10	15	3	S.S	0.022	
32	58	430	300	100	1500	660	25	436	30	15	3	S.S	0.022	
33	58	430	300	100	1500	660	25	342	40	15	3	S.S	0.022	
34	58	430	175	175	1500	440	25	1140	15	8.571429	1	S.S	0.014367347	
35	58	430	175	175	1500	440	25	723	50	8.571429	1	S.S	0.014367347	
36	58	430	175	175	1500	440	25	511	65	8.571429	1	S.S	0.014367347	
37	58	430	300	100	1500	440	25	915	10	15	3	S.S	0.014666667	
38	58	430	300	100	1500	440	25	425	30	15	3	S.S	0.014666667	
39	58	430	300	100	1500	440	25	262	40	15	3	S.S	0.014666667	41
40	92	430	175	175	1500	660	25	1704	15	8.571429	1	S.S	0.02155102	
41	92	430	175	175	1500	660	25	1018	50	8.571429	1	S.S	0.02155102	
42	92	430	175	175	1500	660	25	795	65	8.571429	1	S.S	0.02155102	
43	92	430	300	100	1500	660	25	1189	10	15	3	S.S	0.022	
44	92	430	300	100	1500	660	25	471	30	15	3	S.S	0.022	
45	92	430	300	100	1500	660	25	422	40	15	3	S.S	0.022	
46	92	430	175	175	1500	440	25	1745	15	8.571429	1	S.S	0.014367347	
47	92	430	175	175	1500	440	25	908	50	8.571429	1	S.S	0.014367347	
48	92	430	175	175	1500	440	25	663	65	8.571429	1	S.S	0.014367347	
49	92	430	300	100	1500	440	25	1043	10	15	3	S.S	0.014666667	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	appliedLu/h	b/h	type	ρ	Ref
50	92	430	300	100	1500	440	25	369	30	15	3	S.S	0.01466666	41
51	92	430	300	100	1500	440	25	312	40	15	3	S.S	0.01466666	
52	97	430	175	175	1500	440	25	1975	15	8.571429	1	S.S	0.01436734	
53	97	430	175	175	1500	440	25	1002	50	8.571429	1	S.S	0.01436734	
54	97	430	175	175	1500	440	25	746	65	8.571429	1	S.S	0.01436734	
55	97	430	300	100	1500	440	25	1610	10	15	3	S.S	0.01466666	
56	97	430	300	100	1500	440	25	436	30	15	3	S.S	0.01466666	
57	97	430	300	100	1500	440	25	333	40	15	3	S.S	0.01466666	
58	97	430	175	175	1500	440	25	1932	15	8.571429	1	S.S	0.01436734	
59	97	430	175	175	1500	440	25	970	50	8.571429	1	S.S	0.01436734	
60	97	430	175	175	1500	440	25	747	65	8.571429	1	S.S	0.01436734	
61	97	430	300	100	1500	440	25	1650	10	15	3	S.S	0.01466666	
62	97	430	300	100	1500	440	25	509	30	15	3	S.S	0.01466666	
63	97	430	300	100	1500	440	25	314	40	15	3	S.S	0.01466666	
64	79.1	454	300	100	1386	660	29	1387	10	13.86	3	S.S.S	0.022	
65	79.1	454	300	100	1386	440	29	1200	10	13.86	3	S.S.S	0.01466666	42
66	79.1	454	300	100	936	440	29	1375	10	9.36	3	S.S.S	0.01466666	
67	79.1	454	300	100	936	660	29	1464	10	9.36	3	S.S.S	0.022	
68	79.1	454	300	100	936	440	29	616	30	9.36	3	S.S.S	0.01466666	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As total (mm^2)	d1 (mm)	P test (kN)	eo applied (mm)	Lu/h	b/h	type	ρ	Ref
69	79.1	454	300	100	936	660	29	650	30	9.36	3	S.S.S	0.022	42
70	79.1	454	300	100	1386	660	29	551	30	13.86	3	S.S.S	0.022	
71	79.1	454	300	100	1386	440	29	425	30	13.86	3	S.S.S	0.014666667	
72	79.1	454	300	100	1386	440	29	349	40	13.86	3	S.S.S	0.014666667	
73	79.1	454	300	100	1386	660	29	417	40	13.86	3	S.S.S	0.022	
74	79.1	454	300	100	936	660	29	510	40	9.36	3	S.S.S	0.022	
75	79.1	454	300	100	936	440	29	420	40	9.36	3	S.S.S	0.01466666	
76	98	446	350	350	3600	3619.1147	52	3601.5	0.35	10.28571	1	C.L.L	0.02954379	43
77	98	446	350	350	3600	3619.1147	54.8	3601.5	0.35	10.28571	1	C.L.L	0.02954379	
78	93	446	350	350	3600	3619.1147	52	6835.5	0.35	10.28571	1	C.L.L	0.02954379	
79	93	446	350	350	3600	3619.1147	54.8	6835.5	0.35	10.28571	1	C.L.L	0.02954379	
80	93	446	350	350	3600	3619.1147	52	6835.5	0.35	10.28571	1	C.L.L	0.02954379	
81	23.33957	331.649	155.57	103.18	2641.6	285.02295	16.954	168.1427	7.5184	25.6	1.507	S.S.S	0.01775468	42
82	34.95765	331.649	155.57	103.18	2641.6	285.02295	16.954	68.94741	40.132	25.6	1.507	S.S.S	0.01775468	
83	28.89005	331.649	155.57	103.18	2641.6	285.02295	16.954	189.4942	6.2738	25.6	1.507	S.S.S	0.01775468	
84	30.0622	331.649	155.57	103.18	2641.6	285.02295	16.954	72.50599	39.395	25.6	1.507	S.S.S	0.01775468	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	appliedLu/h	b/h	type		Ref
85	32.75125	331.649	155.57	103.18	2641.6	285.02295	16.954	122.7709	20.1	25.6	1.507	S.S.S	0.01775468	42
86	33.57865	396.462	155.57	103.18	2641.6	285.02295	16.954	197.501	6.6548	25.6	1.507	S.S.S	0.01775468	
87	23.09825	263.389	127	90.487	1470.4	126.41103	14.287	216.1835	0.090587	16.25	1.403	F.F	0.011	44
88	23.09825	263.389	127	90.487	1470.4	126.41103	14.287	271.7862	0.090587	16.25	1.403	F.F	0.011	
89	23.09825	263.389	127	90.487	1470.4	126.41103	14.287	236.2005	0.090587	16.25	1.403	F.F	0.011	
90	19.5818	247.530	127	90.487	1470.4	287.29781	15.875	276.6793	0.090587	16.25	1.403	F.F	0.025	
91	19.5818	247.530	127	90.487	1470.4	287.29781	15.875	233.5316	0.090587	16.25	1.403	F.F	0.025	
92	19.5818	247.530	127	90.487	1470.4	287.29781	15.875	250.8796	0.090587	16.25	1.403	F.F	0.025	
93	31.4412	263.389	127	90.487	1470.4	126.41103	14.287	333.6165	0.090587	16.25	1.403	F.F	0.011	
94	31.4412	263.389	127	90.487	1470.4	126.41103	14.287	235.7557	0.090587	16.25	1.403	F.F	0.011	
95	34.81975	247.530	127	90.487	1470.4	287.29781	15.875	395.0019	0.090587	16.25	1.403	F.F	0.025	
96	34.81975	247.530	127	90.487	1470.4	287.29781	15.875	374.9849	0.090587	16.25	1.403	F.F	0.025	
97	34.81975	247.530	127	90.487	1470.4	287.29781	15.875	343.4026	0.090587	16.25	1.403	F.F	0.025	
98	24.5462	263.389	127	90.487	1764.5	126.41103	14.287	214.849	0.090587	19.5	1.403	F.F	0.011	
99	24.5462	263.389	127	90.487	1764.5	126.41103	14.287	215.7387	0.090587	19.5	1.403	F.F	0.011	
100	24.5462	263.389	127	90.487	1764.5	126.41103	14.287	191.7183	0.090587	19.5	1.403	F.F	0.011	
101	20.8229	247.530	127	90.487	1764.5	287.29781	15.875	211.2905	0.090587	19.5	1.403	F.F	0.025	
102	20.8229	247.530	127	90.487	1764.5	287.29781	15.875	198.3906	0.090587	19.5	1.403	F.F	0.025	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	appliedLu/h	b/h	type	ρ	Ref
103	20.8229	247.530	127	90.487	1764.5	287.29781	15.875	215.2938	0.090587	19.5	1.403	F.F	0.025	44
104	33.99235	263.389	127	90.487	1764.5	126.41103	14.287	297.1411	0.090587	19.5	1.403	F.F	0.011	
105	33.99235	263.389	127	90.487	1764.5	126.41103	14.287	342.9578	0.090587	19.5	1.403	F.F	0.011	
106	31.92385	247.530	127	90.487	1764.5	287.29781	15.875	364.754	0.090587	19.5	1.403	F.F	0.025	
107	31.92385	247.530	127	90.487	1764.5	287.29781	15.875	370.0919	0.090587	19.5	1.403	F.F	0.025	
108	21.99505	263.389	127	90.487	2058.5	126.41103	14.287	177.484	0.090587	22.75	1.403	F.F	0.011	
109	21.99505	263.389	127	90.487	2058.5	126.41103	14.287	158.8015	0.090587	22.75	1.403	F.F	0.011	
110	21.99505	263.389	127	90.487	2058.5	126.41103	14.287	189.4942	0.090587	22.75	1.403	F.F	0.011	
111	21.02975	247.530	127	90.487	2058.5	287.29781	15.875	171.2565	0.090587	22.75	1.403	F.F	0.025	
112	21.02975	247.530	127	90.487	2058.5	287.29781	15.875	194.832	0.090587	22.75	1.403	F.F	0.025	
113	21.02975	247.530	127	90.487	2058.5	287.29781	15.875	192.1631	0.090587	22.75	1.403	F.F	0.025	
114	36.9572	263.389	127	90.487	2058.5	126.41103	14.287	247.7659	0.090587	22.75	1.403	F.F	0.011	
115	36.9572	263.389	127	90.487	2058.5	126.41103	14.287	246.4314	0.090587	22.75	1.403	F.F	0.011	
116	36.9572	263.389	127	90.487	2058.5	126.41103	14.287	250.4348	0.090587	22.75	1.403	F.F	0.011	
117	28.6832	247.530	127	90.487	2058.5	287.29781	15.875	229.973	0.090587	22.75	1.403	F.F	0.025	
118	28.6832	247.530	127	90.487	2058.5	287.29781	15.875	229.0833	0.090587	22.75	1.403	F.F	0.025	
119	28.6832	247.530	127	90.487	2058.5	287.29781	15.875	242.428	0.090587	22.75	1.403	F.F	0.025	
120	19.1681	263.389	127	90.487	2352.6	126.41103	14.287	148.5705	0.090587	26	1.403	F.F	0.011	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As total (mm^2)	d1 (mm)	P test (kN)	eo applied (mm)	Lu/h	b/h	type	ρ	Ref
121	19.1681	263.389	127	90.487	2352.6	126.41103	14.287	151.2395	0.090587	26	1.403	F.F	0.011	
122	19.1681	263.389	127	90.487	2352.6	126.41103	14.287	141.0086	0.090587	26	1.403	F.F	0.011	
123	20.06445	247.530	127	90.487	2352.6	287.29781	15.875	190.3838	0.090587	26	1.403	F.F	0.025	
124	20.06445	247.530	127	90.487	2352.6	287.29781	15.875	161.0256	0.090587	26	1.403	F.F	0.025	44
125	20.06445	247.530	127	90.487	2352.6	287.29781	15.875	167.6979	0.090587	26	1.403	F.F	0.025	
126	38.54305	263.389	127	90.487	2352.6	126.41103	14.287	238.4246	0.090587	26	1.403	F.F	0.011	
127	38.54305	263.389	127	90.487	2352.6	126.41103	14.287	243.7625	0.090587	26	1.403	F.F	0.011	
128	29.30375	247.530	127	90.487	2352.6	287.29781	15.875	226.8592	0.090587	26	1.403	F.F	0.025	
129	29.30375	247.530	127	90.487	2352.6	287.29781	15.875	242.8728	0.090587	26	1.403	F.F	0.025	
130	29.30375	247.530	127	90.487	2352.6	287.29781	15.875	223.3006	0.090587	26	1.403	F.F	0.025	
131	18.2028	247.530	127	90.487	2529.1	287.29781	15.875	169.0324	0.090587	27.95	1.403	F.F	0.025	
132	18.2028	247.530	127	90.487	2529.1	287.29781	15.875	151.6843	0.090587	27.95	1.403	F.F	0.025	
133	18.2028	247.530	127	90.487	2529.1	287.29781	15.875	137.45	0.090587	27.95	1.403	F.F	0.025	
134	30.2001	247.530	127	90.487	2529.1	287.29781	15.875	196.1665	0.090587	27.95	1.403	F.F	0.025	
135	30.2001	247.530	127	90.487	2529.1	287.29781	15.875	225.0799	0.090587	27.95	1.403	F.F	0.025	
136	30.2001	247.530	127	90.487	2529.1	287.29781	15.875	225.5248	0.090587	27.95	1.403	F.F	0.025	
137	33.99235	263.389	127	90.487	1270.4	126.41103	14.287	395.8916	0.090587	14.04	1.403	F.F	0.011	
138	31.92385	247.530	127	90.487	1270.4	287.29781	15.875	381.2125	0.090587	14.04	1.403	F.F	0.025	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	appliedLu/h	b/h	type	ρ	Ref
139	43	684	120	120	2400	448	21	320	20	20	1	S.S.S	0.03111111	42
140	43	684	120	120	2400	448	21	280	20	20	1	S.S.S	0.03111111	
141	86	684	120	120	2400	448	21	370	20	20	1	S.S.S	0.03111111	
142	86	684	120	120	2400	448	21	330	20	20	1	S.S.S	0.0311111	
143	33	636	200	200	3000	796	23	990	20	15	1	S.S.S	0.0199	
144	33	636	200	200	3000	796	23	990	20	15	1	S.S.S	0.0199	
145	91	636	200	200	3000	796	23	2310	20	15	1	S.S.S	0.0199	
146	92	636	200	200	3000	796	23	2350	20	15	1	S.S.S	0.0199	
147	37	636	200	200	4000	796	23	900	20	20	1	S.S.S	0.0199	
148	37	636	200	200	4000	796	23	920	20	20	1	S.S.S	0.0199	
149	93	636	200	200	4000	796	23	1530	20	20	1	S.S.S	0.0199	
150	93	636	200	200	4000	796	23	1560	20	20	1	S.S.S	0.0199	
151	30.40695	217.192	76.2	37.363	1828.8	76.871459	15.1	25.79968	5.08	48.9463	2.039	S.S.C	0.027	
152	29.0969	217.192	76.2	37.363	1828.8	76.871459	15.2	24.28728	5.08	48.9463	2.039	S.S.C	0.027	
153	25.9252	217.192	76.2	37.363	1828.8	76.871459	15.3	25.39934	5.08	48.9463	2.039	S.S.C	0.027	
154	28.89005	217.192	76.2	37.363	1524	76.871459	15.4	33.89544	4.233333	40.78858	2.039	S.S.C	0.027	
155	31.09645	217.192	76.2	37.363	1524	76.871459	15.5	32.87235	4.233333	40.78858	2.039	S.S.C	0.027	
156	30.40695	217.192	76.2	37.363	1524	76.871459	15.6	34.96301	4.239	40.78858	2.039	S.S.C	0.027	

NO	Cylinder strength f'c (Mpa)	fy (Mpa)	b (mm)	h (mm)	Lu (mm)	As (mm^2)	totald1 (mm)	P test (kN)	eo (mm)	appliedLu/h	b/h	type	ρ	Ref
157	29.7864	217.192	76.2	37.363	1219.2	76.871459	15.7	43.68152	3.389	32.63086	2.039	S.S.C	0.027	42
158	30.68275	217.192	76.2	37.363	1219.2	76.871459	15.8	44.25979	3.389	32.63086	2.039	S.S.C	0.027	
159	27.78685	217.192	76.2	37.363	1219.2	76.871459	15.9	40.4788	3.389	32.63086	2.039	S.S.C	0.027	
160	24.822	217.192	76.2	37.363	914.4	76.871459	16	49.82006	2.54	24.47315	2.039	S.S.C	0.027	
161	27.58	217.192	76.2	37.363	914.4	76.871459	16.1	55.60275	2.55	24.47315	2.039	S.S.C	0.027	
162	24.2704	217.192	76.2	37.363	914.4	76.871459	16.2	49.82006	2.55	24.47315	2.039	S.S.C	0.027	
163	33.5097	217.192	127	63.5	1524	217.7415	16.3	221.9662	4.239	24	2	S.S.C	0.027	
164	33.5097	217.192	127	63.5	1016	217.7415	16.4	270.8966	2.829	16	2	S.S.C	0.027	
165	33.5097	217.192	127	63.5	1016	217.7415	16.5	262.0002	2.829	16	2	S.S.C	0.027	
166	33.5097	217.192	127	63.5	1016	217.7415	16.6	263.3346	2.829	16	2	S.S.C	0.027	
167	32.06175	217.192	127	63.5	508	217.7415	16.7	314.0443	1.411111	8	2	S.S.C	0.027	
168	32.06175	217.192	127	63.5	508	217.7415	16.8	312.7099	1.411111	8	2	S.S.C	0.027	

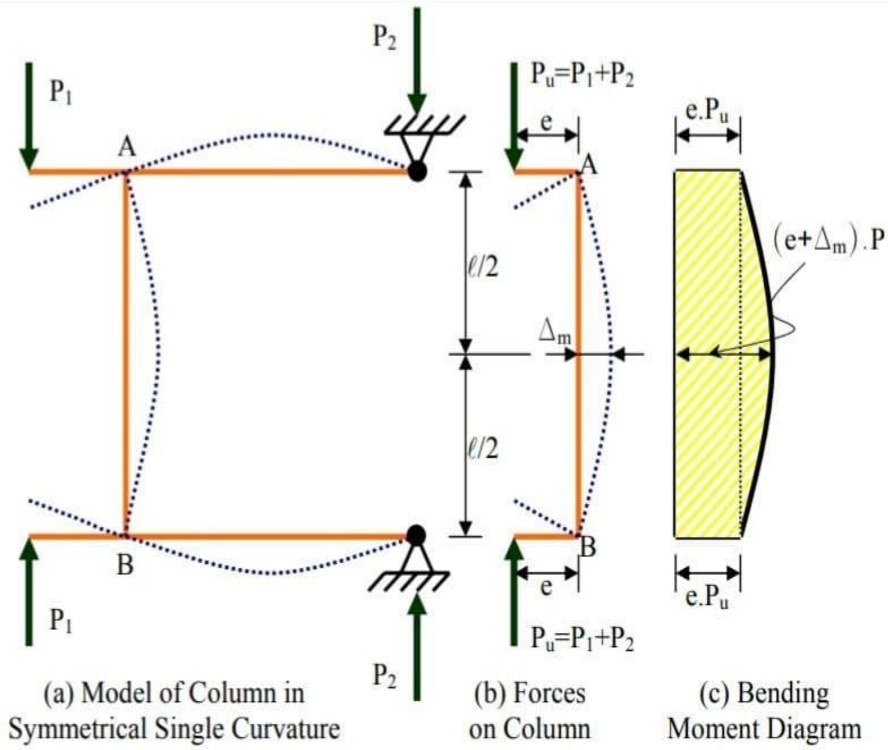


Fig. (4-1) Type of hypothetical columns studied

properties	Specified values	No. of specified values
F_c' (MPa)	20;30;40;50;60;70;80	7
F_y (MPa)	100;200;300;400	4
L_u/h	1;10;20;30;40;50	6
ρ_t	Table (4-3) for combinations of steel ratios and bars arrangements	12
b/h	0.5;1;1.5;2;2.5;3	6

Total number = 7* 4*6*12*6 = 12,096 columns

Table (4-2) specified properties of columns studies

In which

L_u/h = slenderness ratio
e/h = eccentricity ratio

In table (4-2) are the ranges of values used (f_y, f_c', L_u/h, b/h, ρ_t) employed by the construction industry.

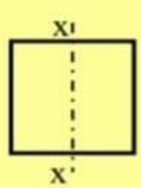
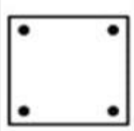
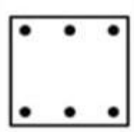
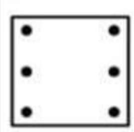
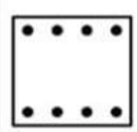
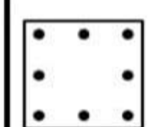
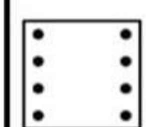
Steel Ratio ρ_t %, b = h = 300 mm.						
Bar Size (mm)	**No. of Bars**					
	4	6	6	8	8	8
16	0.89	1.34	1.34	---	---	---
20	---	---	---	2.79	2.79	2.79
22	---	---	---	3.38	3.38	3.38
25	---	---	---	4.36	4.36	4.36
 x-x is the axis of bending	Arrangement of Longitudinal Reinforcement					
						

Table (4-3) longitudinal reinforcement details of selected hypothetical columns

4.3 COMPUTER PROGRAMS AND COMPARISON FORMULAS

The two programs Matlab 2019a and Microsoft office excel 2020 were presented in this study to conduct an inclusive study for analysis of the concrete columns available in the literature, by different analysis methods that were developed in this work.

Matlab 2019a is one of the most powerful programs used at the present time, as characterized by accuracy and speed in carrying out the required tasks and for a large number of values in a short period of time, in this study, this program is utilized to analyze columns and all specifications of each column.

The Microsoft office excel 2020 program is used for modeling, scheduling and comparing all results attained from the analysis of each method.

In order to count the results and data in a useful way, the most commonly scale was used to compare the results, the arithmetic mean (X), and for the interpretation of the results the coefficient of variation (COV) is used.

4.4 THE ANALYTICAL PROPOSED METHODS

In this work a proposed method will made on the standard analysis and design procedure, for analysis 168 reinforced concrete columns using moment magnifier approach for (ACI318M-19 and TS500) codes and additional moment approach for (BS8110) code by statically regression analysis. Where existence some of constants in the standard equations leads to less accurate in the results. Therefore, this constant will changed to equations will be proposed that include some of details of columns like the amount of steel ratio, the dimension of the section and slenderness ratio.

By the statically regression analysis for 12096 hypothetical rectangular RC slender columns, the effect of each factor will be shown. Comparison is made between the results of proposed method and the results of standard methods, where the effect of the properties will be appearing on the final values.

During the column analysis process by matlab program, the maximum lateral deflection of each column was obtained, where an extensive regression analysis is performed to find out the main factors affecting on the deflection and ultimate bearing capacity like flexural rigidity EI, buckling load Pc, and c; depth of neutral axis, all these factors gives the maximum deflection and ultimate bearing capacity of columns.

Two formulas and one value are suggested in the analysis procedure to obtain the most accurate value of bearing capacity and maximum lateral deflection of the column that occurs as a result of the imposed loads.

All suggested formulas will be explained below:

4.4.1 Effective Rigidity Factor (α)

- In (ACI318M-19 and TS500) codes, the analysis of reinforced concrete slender column subjected to load is effected by the flexural rigidity (EI), and it's given in the equation below:

$$El = (0.2\ E_c\ I_g + E_s\ I_s) / (1 + \beta_d) \qquad \text{….. } (4-1)$$

- In (BS8110) code, the analysis of reinforced concrete slender column is based on the additional moment approach and it shown in the equation below:

$$M_{add} = N\beta_a\ k\ h \qquad \text{….. } (4-2)$$

$$\beta_a = 1/2000\ \left(\frac{le}{b'}\right)^2 \qquad \text{….. } (4-3)$$

$$k = \frac{Nuz-N}{Nuz-Nbal} \leq 1 \qquad \text{….. } (4-4)$$

Through the equation (4-1), there is less of an inaccuracy that occurs due to the constant that used (0.2) which is not affected by the factors that affect the stiffness. Therefore, in this work a modified version will be made to the equation below:

$$El = (\alpha E_c\ I_g + E_s\ I_s\ ) / (1+\beta_d\ ) \qquad \text{….. } (4-5)$$

Where

α = Dimension less reduction factor which is influenced by factors affecting the stiffness process

β_d = ratio of max factored dead (sustained) load to max total factored load (it's always positive), for short time load β_d equal to zero.

Based on the observation of the results that obtained by the regression analysis of the slender columns that examined experimentally performed, it appears evidently that the factors (L_u/h, steel ratio) have an effect on the values of the flexural rigidity (EI), and the other factors (concrete cover γ, e/h, b/h) that are independent variables with the effective stiffness factor (α) as a dependent variable.

With these assumptions, an equation below found that explains the effect of each factor, Eq (4-6a) shows the expression:

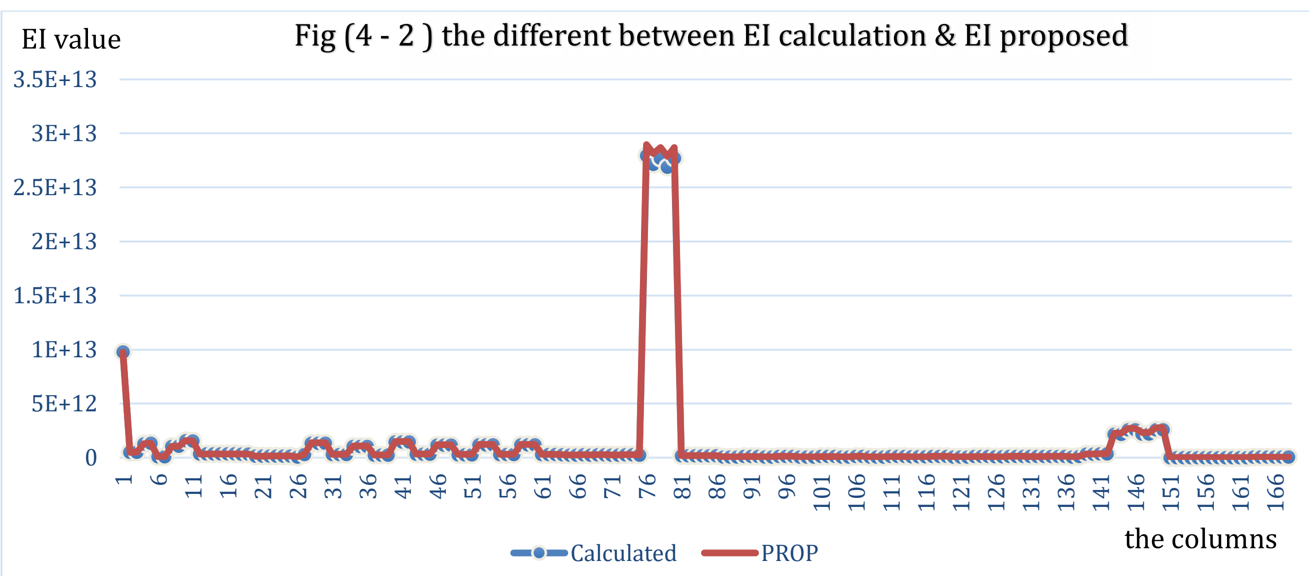
$\alpha_{prop} = 0.2+0.002\ L_u/h+0.01\ \rho$ .....(4 – 6a)

Which

$EI_{\ effective} = (\alpha_{prop} E_c\ I_g + E_s\ I_s)/\ (1+\beta_d)$ .... (4 – 6b)

Accordingly, the change in the value of effective flexural rigidity factor (α) was observed as a result of the regression analysis.

Fig (4-2) shows the deferent between the two results (EI calculation & EI proposed). Noting that the values in laboratory tested with f_c' values are within (16.27 - 98) MPa, L_u/h ranging between (0- 48.94), steel ratio ρ ranging shown in table (4-2).



A ratio was made between (αProp/ α exact), where α exact = 0.2 to find the value of coefficient of variation COV between the two expression which gives (COV=8.67)% and mean (X=1.16). Fig (4-3) below show that.

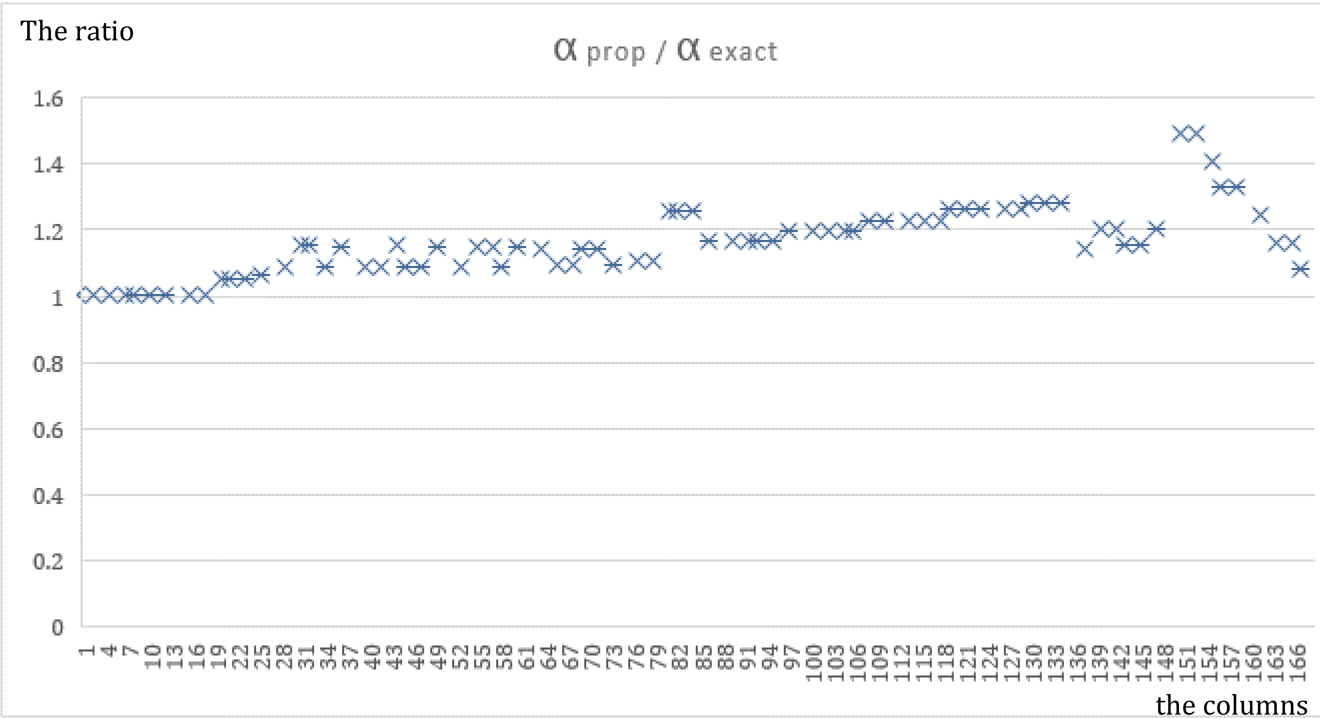


Fig (4 - 3) comparison between αProp &α exact

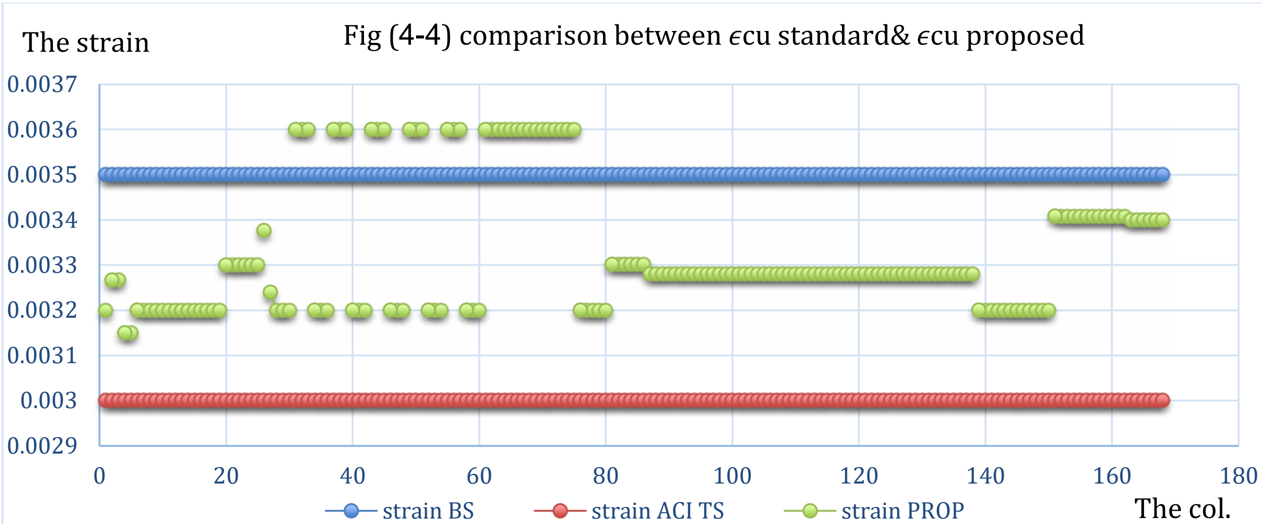
4.4.2 ULTIMATE COMPRESSION STRAIN OF CONCRETE

The ultimate strain of concrete is a constant value for ACI 318M-19 and TS500 codes, which is equivalent to 0.003, and for BS-8110 code the strain of concrete is equivalent to 0.0035. This work will show the effect of dimensions on the amount of strain value by calculating the ratio between width to height of column (b/h) in experimental results in the literature, which is ranging between (0.75- 3).

By this ratio the following expression is obtained:

$\epsilon_{cu} = 0.003 + 0.0002\ b/h$ …. (4 -7)

Where the regression analysis was made for the experimental results, the change of values ϵ_{cu} standard and ϵ_{cu} proposed becomes clear, and be observed in fig (4-4).



4.4.3 THE INTENSITY OF STRESS AT THE EXTREME FIBER OF STRESS BLOCK

The distribution of the rectangular stress block has great influence in knowing the bearing capacity and lateral deflection of the column. This block is directly affected by many factors and especially by ($\alpha1$). In ACI 318M-19, BS-8110 and TS500 codes the factor $\alpha1$ is equal to 0.85.

In this work, the value of $\alpha1$ will be proposed as 0.8. This change will effect directly on the value of compressive strength of concrete (f'_c), where in the experimental results the values of (f''_c) ranges between (13.83 and 83.3), while during the regression analysis the values of (f''_c) will ranging between (13.01 and 78.4), therefore the bearing capacity and lateral deflection values of the columns will change.

4.5 THE COMPARISON BETWEEN EXPERIMENTAL RESULTS AND PROPOSAL RESULTS

Statically regression analysis is performed for 168 columns for the three codes and the proposed method. The coefficient of variation and mean are the most popular measures they have been used for compare results and data in useful way. By Matlab the ultimate bearing capacity for each column will extract and for the four methods. Table (4-4) shows a comparison between the results by the mean and the coefficient of variation.

No.	The code	Mean X	COV
1	ACI 318M –19	754.089	277.2248
2	BS8110	768.810	279.0306
3	TS500	755.926	276.7155
4	Proposed	742.049	269.4325

Table (4 – 4) statistical evaluation of load bearing capacity for 168 tested Rc slender column

Fig (4-5)& (4-6), show the graphics for the different values of (COV & X) between the codes and proposed method.

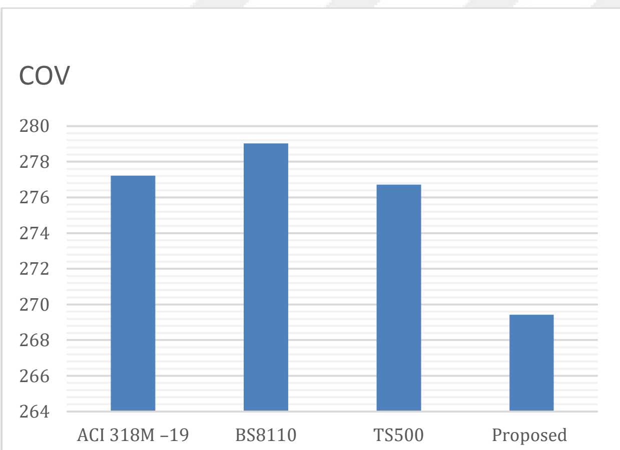


Fig (4 – 5) COV % for the results of load bearing capacity calculated by the three codes and the proposed method for 168 RC columns.

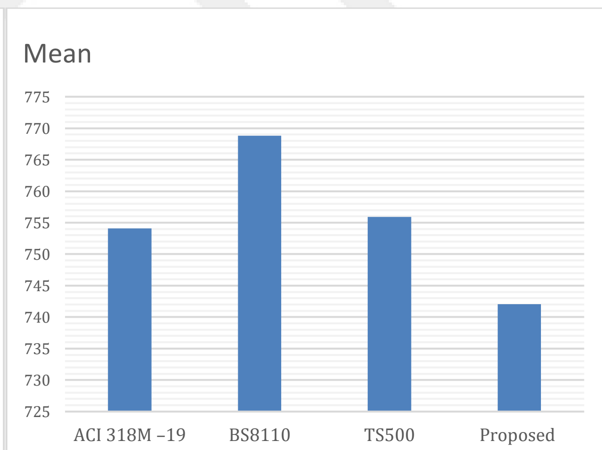


Fig (4 – 6) Mean value for the results of load bearing capacity calculated by the three codes and the proposed method for 168 RC columns.

In order to achieve the maximum accuracy of the theoretical, used the nominal axial capacity for the column obtained experimentally test (Ptest)was used by ratios with the results of 168 columns. where a comparison was made between the ratio of (Ptest) to calculated bearing capacity (Pcalc) with the ratio of (Ptest) to the proposed bearing capacity (Pprop) and that shown in table (4-5). Also the maximum and minimum value of this ratio will show.

Fig (4- 7) shown the different between amounts of the two ratios for 168 concrete column.

No.	The ratios	X	COV%	MAX	MIN
1	P test /P calc	1.21	11.09	1.62	1.02
2	P test / P prop	1.19	8.88	1.66	1.04

Table (4 – 5) statistical evaluation of the ratios for 168 tested Rc slender column

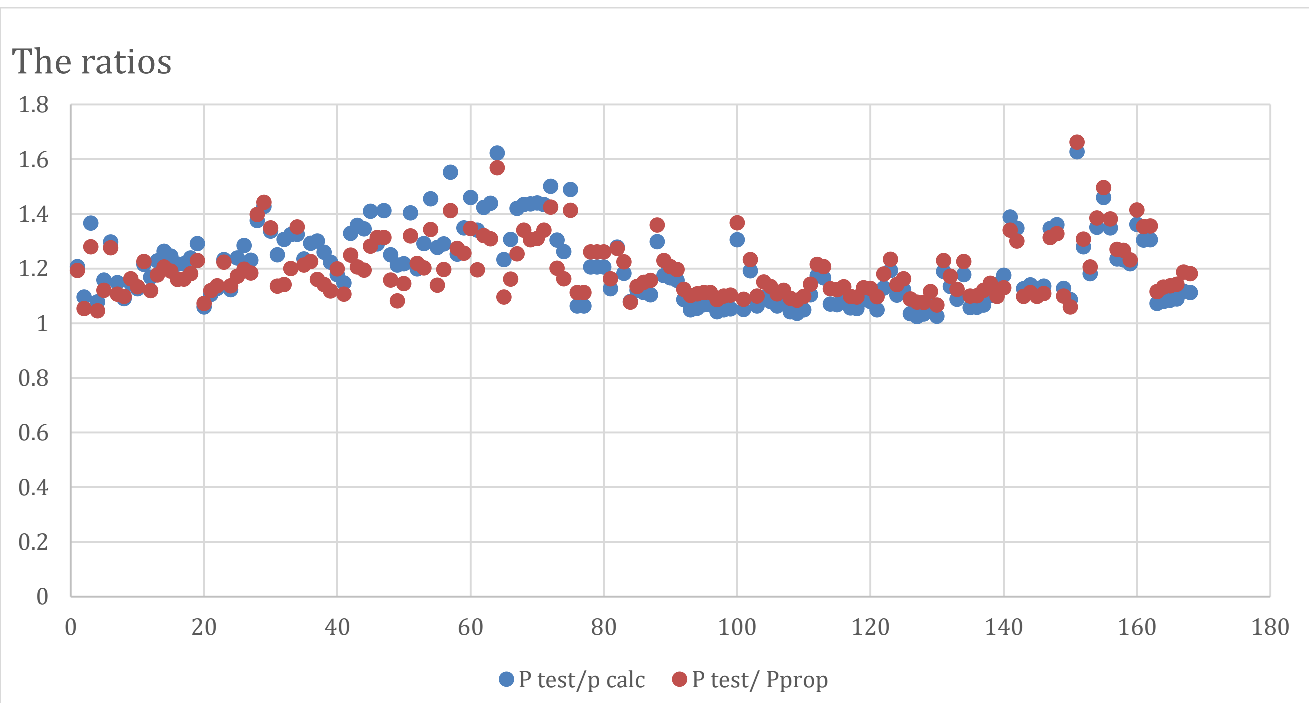


Fig (4-7) the different between the amounts of the two ratios for 168 concrete column.

5. CONCLUSION AND RECOMMENDATION

5.1 INTRODUCTION

In this work, a study was made to investigate codes method to analyze and design RC slender columns. 168 experimentally rectangular columns HSC and NSC tested in the literature, it will be exposed to short time concentric and eccentric load.

Two types of columns are available short and slender, buckle in sway or non-sway mode. A case study of 12,096 rectangular RC columns was used with different combination of specific values of variables that affected on load bearing capacity, column behavior as well as stiffness.

Four methods were used in this study to find the ultimate of bearing capacity of slender columns. Three methods are universal codes (ACI 318M-19, BS 8110 and TS500) in addition to one method was proposed in Chapter Four, where they were derived statistically.

The values of bearing capacity of the slender columns calculated from the four methods were compared with the results of 168 slender columns that were experimentally tested. The values that were laboratory tested with f_c' values are within (16.27 - 98) MPa, Lu/h ranging between (0- 48.94), steel ratio ρ ranging, shown in table (4-2).

A new proposed expression to analyze RC slender column was made by introducing some changes on flexural rigidity (EI) values, strain values and the factor ($\alpha 1$), all of these factors were statistically derived.

Using Matlab 2019a program helped greatly in simplifying the analysis mechanism and adopting accuracy in results. In order to modeling, scheduling and comparing the values, use Microsoft Excel 2020.

5.2 THE CONCLUSION

From the studies presented in this work, several things have been concluded, such as:

a. Recommended to use for analysis 168 reinforced concrete columns using moment magnifier approach for (ACI318M-19 and TS500) codes and additional moment approach for (BS8110) code by statically regression analysis, where ACI 318M-19 code gave (COV = 277.2248) percent and mean (X = 754.089), BS 8110 code gave (COV = 279.0306) percent and mean (X = 768.810), TS 500 code gave (COV = 276.7155) % and mean (X = 755.926), the method proposed in this work gave the lower value of (COV= 269.4325) % and mean (X = 742.049), which means the proposed method gave an acceptable value more than other codes.
b. The ratio between Ptest / Pcalc gave(COV=11.09) % and mean (X=1.21), and Ptest / Pprop gave lowest value of (COV=8.88) % and mean (X=1.19), that mean the proposed results was in more agreement with the test results and the closest to unity.
c. Improvement in the results accuracy of column strength was achieved using the new flexural stiffness factor in the equation of EI. The effect of the factors (Lu/h , ρ) on the amount of (α), where the flexural rigidity (EI) in (ACI 318M-19, TS500) codes extract in the Eq (4-5), where (α =0.2). In this study Eq (4-6a) is proposed to obtain the flexural rigidity factor (α), this suggestion proved that constant values cause some of an inaccuracy in the results.
d. The strain of concrete is a constant value for ACI 318M-19 and TS500 codes, which is equal to 0.003, and for BS-8110 code the strain of concrete is equal to 0.0035. In this work it will show the effect of the ratio (b/h) on the strain value, the Eq (4-7) is proposed to find the strain value and it will be ringing between (0.003-0.0036).

5.3 RECOMMENDATIONS

To better understand the behavior of RC slender column and to calculate the maximum lateral deflection and the nominal capacity of the RC rectangular column, this work recommends using the proposed method and depends on moment magnifier approach.

5.4 SUGGESTIONS FOR FUTURE RESEARCH

Based on the conclusions achieved from this work, and in order to gain added benefit from the recommendations in the areas of research that are concerned with the behavior, bends of concrete columns and strength, several vital points have been identified for future implementation:

Expand the research range to include:

a. Other types of experimental columns, including circular columns, columns with unequal end moments, column with ultra HSC, columns under biaxial bending, and centrally loaded columns, etc.
b. Innovative techniques for additional uploading to identify load deflection response through overall loading history.
c. The procedures used to calculate details of columns analysis, such as deflections and bearing strength for RC frame and RC beams.
d. The effect of factors (P/P_o , f_y, e/h) on column specifications.
e. The effect of continuous loads on failure of concrete columns.
f. The effect of loading rate on failure of concrete columns.
g. The effect transverse reinforcement and concrete cover on the failure that happen of concrete columns.
h. The effect of effective length factor for columns in the sway or non-sway frames.

6. REEFRENCESS

[1] D. Darwin, C. W. Dolan, and A. H. Nilson, *Design of concrete structures*. McGraw-Hill Education New York, NY, USA:, 2016.

[2] E. P. Popov, “Introduction to mechanics of solids,” 1968.

[3] R. Park and T. Paulay, *Reinforced concrete structures*. John Wiley & Sons, 1975.

[4] P. M. Ferguson, “Reiforced Concrete Fundamentals, SI Version.” terjemahan Budianto S., dkk.), Erlangga, Jakarta, 1984.

[5] R. Narayanan, *Concrete framed structures: stability and strength*. Elsevier applied science publishers, 1986.

[6] Z. Chen and W. Yi, “Numerical Simulation of the Moment—Curvature Relationship for HSCCRST,” in *Computing in Civil Engineering (2005)*, 2005, pp. 1–8.

[7] A. H. Mattock, L. B. Kriz, and E. Hognestad, “Rectangular concrete stress distribution in ultimate strength design,” in *Journal Proceedings*, 1961, vol. 57, no. 2, pp. 875–928.

[8] A. H. MATTOCK and L. B. KRIZ, “Rectangular Concrete Stress.”

[9] R. H. Evans, K. HAJNAL KONYI, V. P. JENSEN, C. S. WHITNEY, and L. J. MENSCH, “THE PLASTIC THEORIES FOR THE ULTIMATE STRENGTH OF REINFORCED-CONCRETE BEAMS.,” *J. Inst. Civ. Eng.*, vol. 21, no. 2, pp. 98–121, 1943.

[10] E. Hognestad, N. W. Hanson, and D. McHenry, “Iconcrete STRESSDISTRIBUTION~.”

[11] A. C. I. C. 441, “High-Strength Concrete Columns: State of the Art.” ACI 441R-96, American Concrete Institute Committee 363, Farmington Hills, MI, 1996.

[12] R. H. Basappa Setty and B. V Rangan, “Failure load of high strength concrete (HSC) columns under eccentric compression,” *Trans. Inst. Eng. Aust. Civ. Eng.*, vol. 38, no. 1, pp. 19–30, 1996.

[13] S. Mirza, “Flexural stiffness of rectangular reinforced concrete columns.,” *ACI Struct. J.*, vol. 87, no. 4, pp. 425–435, 1990.

[14] L. F. Khan and K. F. Meyer, “Rectangular stress block for nonrectangular compression zone,” *Struct. J.*, vol. 92, no. 3, pp. 304–310, 1995.

[15] J. S. Ford, D. C. Chang, and J. E. Breen, “Behavior of concrete columns under controlled lateral deformation,” in *Long Reinforced Concrete Columns*, 2010, pp. 188–205.

[16] L. Pallarés, P. F. Miguel, and M. A. Fernández-Prada, “A numerical method to design reinforced concrete sections subjected to axial forces and biaxial bending based on ultimate strain limits,” *Eng. Struct.*, vol. 31, no. 12, pp. 3065–3071, 2009.

[17] B. Standard, “BS 8110-1: 1985, Structural Use of Concrete: Part 1: Code of Practice for Design and Construction,” *Br. Stand. Inst.*, 1985.

[18] B. S. British Standard, “8110 (1997) Structural use of concrete-part 1. Code of practie for design and construction,” *Br. Stand. Institution, London*.

[19] T. Standard, “TS500: Restrictions for design and construction of R/C structures,” *Ankara, Minist. Public Work. Settl.*, 2000.

[20] P. Soroushian, J. Sim, and J.-W. Hsu, “Axial/flexural behavior of reinforced concrete sections: Effects of the design variable,” *Struct. J.*, vol. 88, no. 1, pp. 17–21, 1991.

[21] Y.-K. Yong, M. G. Nour, and E. G. Nawy, “Behavior of laterally confined high-strength concrete under axial loads,” *J. Struct. Eng.*, vol. 114, no. 2, pp. 332–351, 1988.

[22] B. V Rangan, *Large concrete buildings/editors, BV Rangan and RF Warner.* Longman, 1996.

[23] C.-T. T. Hsu, “Analysis and design of square and rectangular columns by equation of failure surface,” *Struct. J.*, vol. 85, no. 2, pp. 167–179, 1988.

[24] J. Y. R. Yen, “Optimized direct design of reinforced concrete columns with uniaxial loads,” *Struct. J.*, vol. 87, no. 3, pp. 247–251, 1990.

[25] G. J. Smith and F. N. Rad, “Economic advantages of high-strength concretes in columns,” *Concr. Int.*, vol. 11, no. 4, pp. 37–43, 1989.

[26] J.-M. Zeng, L. Duan, F.-M. Wang, and W.-F. Chen, “Flexural rigidity of reinforced concrete columns,” *Struct. J.*, vol. 89, no. 2, pp. 150–158, 2000.

[27] J. G. MacGregor and J. E. Breen, “Design of slender concrete columns,” in *Journal Proceedings*, 1970, vol. 67, no. 1, pp. 6–28.

[28] B. B. Goyal and N. Jackson, “Slender concrete columns under sustained load,” *J. Struct. Div.*, vol. 97, no. 11, pp. 2729–2750, 1971.

[29] S. A. Mirza and J. G. MacGregor, “Slenderness and strength reliability of reinforced concrete columns,” *Struct. J.*, vol. 86, no. 4, pp. 428–438, 1989.

[30] J. G. MacGregor and S. E. Hage, “Stability analysis and design of concrete frames,” *J. Struct. Div.*, vol. 103, no. 10, pp. 1953–1970, 1977.

[31] M. A. Diaz and J. M. Roesset, “Evaluation of Approximate Slenderness Procedures for Nonlinear Analysis of Concrete Frames,” *Struct. J.*, vol. 84, no. 2, pp. 139–148, 1987.

[32] B. V. Rangan, “Lateral deflection of slender reinforced concrete columns under sustained load,” *Struct. J.*, vol. 86, no. 6, pp. 660–663, 1989.

[33] B. V. Rangan, “Strength of reinforced concrete slender columns,” *Struct. J.*, vol. 87, no. 1, pp. 32–38, 1990.

[34] J. L. Bonet, M. L. Romero, and P. F. Miguel, "Effective flexural stiffness of slender reinforced concrete columns under axial forces and biaxial bending," *Eng. Struct.*, vol. 33, no. 3, pp. 881–893, 2011.

[35] T. K. Tikka and S. A. Mirza, "Nonlinear El equation for slender reinforced concrete columns," *ACI Struct. J.*, vol. 102, no. 6, p. 839, 2005.

[36] J. Webb, "High-strength concrete: economics, design and ductility," *Concr. Int.*, vol. 15, no. 1, pp. 27–32, 1993.

[37] P. K. Mehta, "Concrete. Structure, properties and materials," 1986.

[38] A. C. I. C. 318, "Building Code Requirements for Reinforced Concrete and Commentary (ACI 318M-19 and ACI 318RM-19)," 2019.

[39] J. K. Wight, "Changes to Expect in the 2008 Edition of the ACI Building Code (ACI 318-08)," *Concr. Int.*, vol. 29, no. 7, pp. 49–53, 2007.

[40] B. Standard, "BS 8110 Part 1, Structural use of concrete part 1. Code of practice for design and construction," *Br. Stand. Inst.*, 1997.

[41] N. A. Lloyd and B. V. Rangan, "Studies on high-strength concrete columns under eccentric compression," *Struct. J.*, vol. 93, no. 6, pp. 631–638, 1996.

[42] W. F. Chang and P. M. Ferguson, "Long-hinged reinforced concrete columns," in *Journal Proceedings*, 1963, vol. 60, no. 1, pp. 1–26.

[43] L. P. Saenz and I. Martin, "Test of reinforced concrete columns with high slenderness ratios," in *Journal Proceedings*, 1963, vol. 60, no. 5, pp. 589–616.

[44] C. Claeson and K. Gylltoft, "Slender high-strength concrete columns subjected to eccentric loading," *J. Struct. Eng.*, vol. 124, no. 3, pp. 233–240, 1998.

CIV

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