Effect of Lateral Supports on the General Behavior of Reinforced Concrete Columns

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Abstract: Columns are vertical compression members which carry primarily axial Compression load; the axial load may be associated with bending moments in one or two directions. They transmit loads from the upper floors to the lower levels and then to the soil through the foundations. Since columns are compression elements, failure of one column in a critical location can cause progressive collapse of adjoining floors and might lead to total collapse of the entire structure. This study is carried out to investigate the general deformational behavior of laterally braced reinforced concrete columns at floors' levels. The columns are subjected to axial compression loads acting at the top level of column. The cross section of columns and their reinforcing steel are kept constant, while the locations of the lateral beams at floor level within the long dimension of the column cross section in addition to the unsupported length of columns and the rigidity factor of the lateral bracing beams are variables. The experimental phase of this research work comprised testing of four reinforced concrete rectangular columns of medium scale model repesenting a ground and two typical floors column. In the analytical phase of this research work, the tested columns were analyzed using a computer program (ANSYS), taking into consideration the nonlinear behavior of concrete and reinforcing steel. A comparison between the experimental and analytical results was made to verify the finite element model of the tested columns. This was a necessary step to study more related parameters by the finite element analyses such as the unsupported length of columns and the rigidity factor of the lateral bracing. This research presents a proposed equation for calculating the ultimate load of the laterally braced tied columns which takes into consideration the effect of changing the unsupported length of columns, the rigidity factor of the lateral supports and its locations within the long dimension of columns' cross section.

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Key words: lateral support; laterally braced, concrete columns, unsupported length, rigidity factor.

1. Introduction

Structural column failure is one of major significance in terms of economic as well as human loss. Thus, extreme care needs to be taken in column design, with higher conservative strength than in the case of beams and other horizontal structural elements, since compression failure provides little visual warning.

This study is undertaken to investigate the general deformational behavior of centric / eccentric laterally supported reinforced concrete columns at floors' levels. These columns are subjected to axial compression loads acting at their top level. The cross section of columns and their reinforcing steel are kept constant, while the locations of the lateral supports through the cross section of columns in addition to the unsupported length ratio of columns (l_c / b) and the rigidity factor of the lateral bracing (I_b / l_b) are variables.

Studies on high strength concrete columns under eccentric compression were conducted by **Lloyd and Rangan** [(4)]. The results of a research program on the behavior and strength of high strength concrete columns under eccentric compression were presented. Thirty-six columns were tested; the variables were column's cross-section, eccentricity of load, longitudinal reinforcement ratio, and concrete compressive strength. A theory was developed to predict the load deflection behavior and the failure load of high strength concrete columns under eccentric compression. The theory was based on a simplified stability analysis and a stress-strain relation of high strength concrete in compression.

Strength and ductility of laterally confined concrete columns was studied by **Chung *et al*.** [(1)]. The objective of this study was to determine experimentally and analytically the magnitude of the strength enhancement of concrete confined by lateral ties. Sixty-five reinforced concrete columns with a 200 mm square cross section were tested. An empirical equation was presented to determine the strength enhancement as a function of the tie stress, the effectively confined distance ratio, the volumetric ratio of ties, and the strength of concrete. The validity of the American Concrete Institute and Canadian Standards Association specifications for minimum tie spacing and the design of cross ties were examined.

Buckling behavior of slender high-strength concrete columns was investigated by **Kima and Yanga** [(3)]. The predicted behavior of the concrete columns by the numerical method proposed herein show good agreement with the test results, they also show that the ACI's moment magnifier method may be unconservative for very slender high-strength concrete column.

Slenderness and strength reliability of reinforced concrete columns was investigated by **Mirza and MacGregor** [5]. As expected, the variability of concrete strength was a major contributing factor to the slender column strength variability in a region of low eccentricity ratios, whereas the variability in the steel strength made a major contribution to the slender column strength variability when the end eccentricity ratios were high.

2. Experimental work: [6]

The experimental work of the present study consists of testing four reinforced concrete columns of medium scale model. The columns had a total height equals to 380 cm and simulates a three floors column. The cross section of the tested column was a rectangular section with dimensions (36 X 9 cm) resting on reinforced concrete footing of dimensions (115 X 90 X 25 cm). To simulate the floors beams, three beams with cross-section (9 X 25 cm) and length 75 cm in the short and long sides of the cross section, had been monolithically cast with the column.

The results of strength control tests are given in Table (1), in order to determine the characteristic strength of concrete, f_{cu}.

Table (1) Concrete control specimens

Column No.	f_{cu} kg/cm^2
C1	345.0
C2	350.0
C3	346.0
C4	348.0

2.1 Equipment and instrumentation:

All of the columns were loaded with a hydraulic jack of 120 ton capacity. The applied load was read out on the jack dial scale. The set up of loading of the tested columns is shown in figure (1) and photograph (1). Twelve displacement dial gauges of 0.01 mm accuracy were placed at different position along column height to monitor its deflected shape at different increments of loading.

The concrete strains were measured by mechanical strain gauges of 150 mm gauge length, using demic points mounted on the columns' sides at the same positions in all columns.

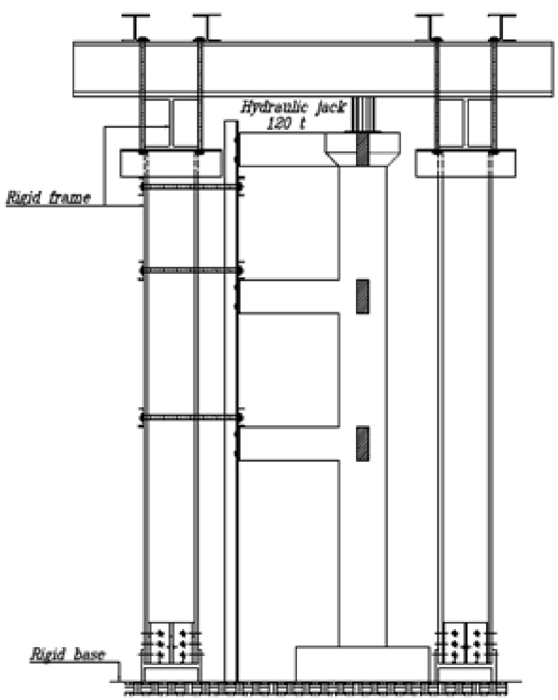


Fig.(1):General setup

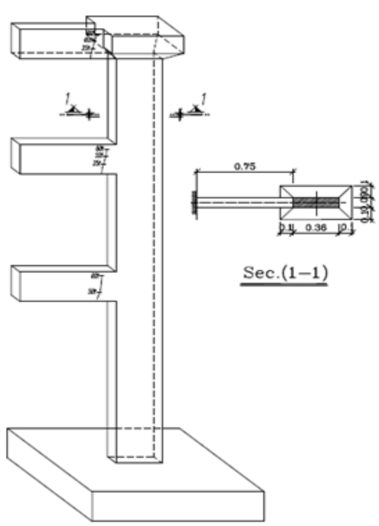
Photograph (1):General setup

2.2 Crack pattern, cracking and failure load

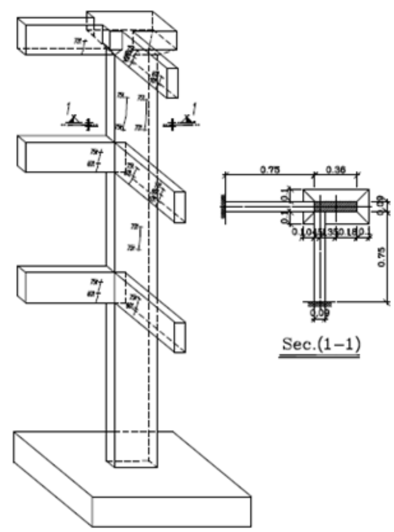
For the four tested columns, the first crack appeared at a load level about 37.5 % to 42 % of the ultimate (the failure) load, table (2) shows the value of loads at which the first crack appeared for each column. In addition, figures (2) to (5) and photographs (2) to (5) show the crack pattern and the shape of failure of the tested columns.

Table (2) Cracking and Failure Loads

Column No.	Cracking load (ton)	Failure load (ton)	Cracking load % Failure load	Mode of failure
C1	25.00	60.00	41.67	Crushing
C2	30.00	80.00	37.50	Crushing
C3	40.00	100.00	40.00	Crushing
C4	45.00	115.00	39.13	Crushing



Fig(2) Crack Pattern of Column(C1)



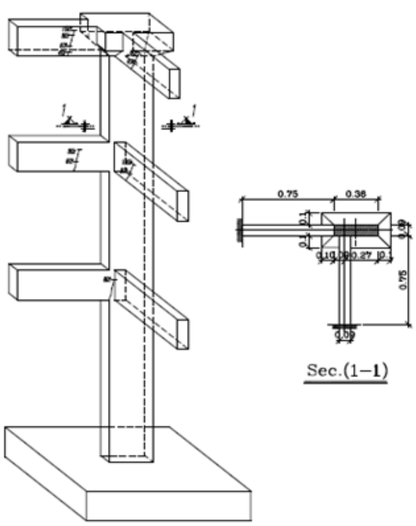
Fig(3) Crack Pattern of Column (C2)

Photograph(2) Shape of Faliure of (C1)

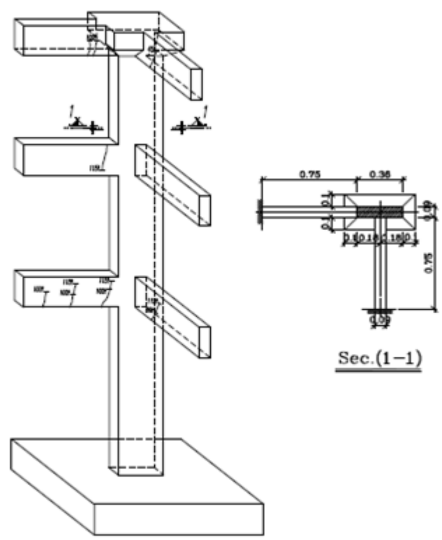
Photograph(3) Shape of Faliure of (C2)

Photograph(4) Shape of Faliure of (C3)

Photograph(5) Shape of Faliure of (C4)



Fig(4) Crack Pattern of Column (C3)



Fig(5) Crack Pattern of Column (C4)

2.3 Deflections

The lateral deflection was measured through the tested columns height to trace the deflected shape. The experimental load-deflection curves at different sections along the heights of the tested columns at different load levels up to the failure load are shown in figures (6) to (8).

From these figures, it can be noticed that the load-deflection curves for the four tested columns were nearly linear at the early stages of loading {from zero load up to cracking of concrete}. After that; increasing the load caused an excessive cracking propagation, and consequently the stiffness of the columns decreased and accordingly a great increase in the deflection values had occurred. While, approaching the failure load, the deflections continued to increase even at constant applied load.

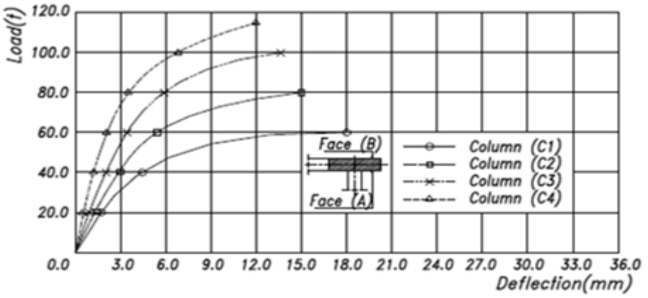


Fig.(6): Load–Deflection Curve at the Mid–Height of Ground Floor

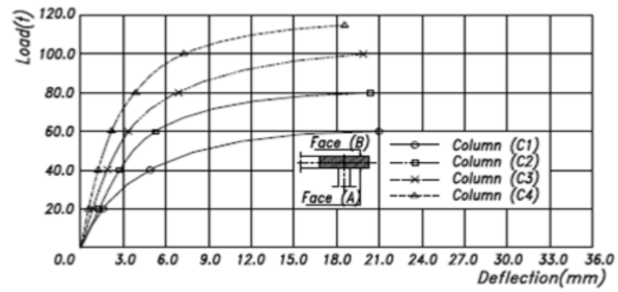


Fig.(8): Load–Deflection Curve at the Mid–Height of Second Floor Level

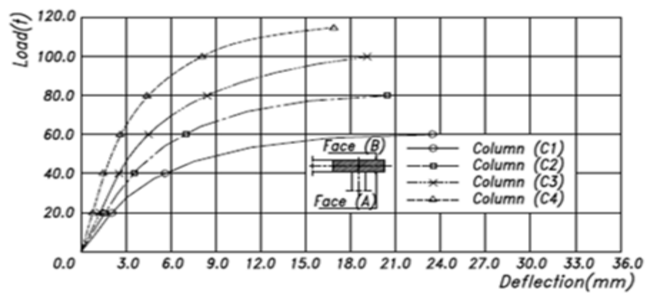


Fig.(7): Load–Deflection Curve at the Mid–Height of First Floor

3. Finite Element Analysis

3.1 Description of the analyzed columns

The four tested columns of present study were analyzed under the effect of concentric loads acting at the top level of columns. The load was increased incrementally from zero up to the failure load in the same way used in the experimental phase of this research.

The material properties of the analyzed columns are kept constant, and they are taken as an average value from the recorded experimental data. These properties are listed in table (3).

Table (3) Material properties

Property	Value
Modules of elasticity for concrete E_c	210 kg / cm^2
Poisson's ratio for concrete ν	0.16
Shear transfer coefficient for an open crack	0.6
Shear transfer coefficient for a closed crack	0.9
Uniaxial tensile cracking stress	37 kg / cm^2
Uniaxial crushing stress	350 kg / cm^2
Modules of elasticity for steel E_s	2100x10^3 kg / cm^2
Poisson's ratio for steel ν	0.3

3.2 Finite Element Model Verification

A comparison is made to verify the finite element model of tested columns. This is a necessary step to use the finite element analysis in the parametric study reported in this study. This verification is achieved through a comparison of analytical values to the corresponding experimental ones. Values of the cracking and failure loads and deflections are considered in this comparison.

The large capacity of the used computer program called ANSYS allowed the use of a finer mesh and increaed number of nodes and elements of the analyzed columns. It also gave an opportunity to increase the number of the load steps and consequently increasing the accuracy of determining the cracking and failure loads.

3.2.1 Cracking and failure loads

The relative values of the experimental and analytical cracking and failure loads are shown in table (5) and figures (9) and (10).

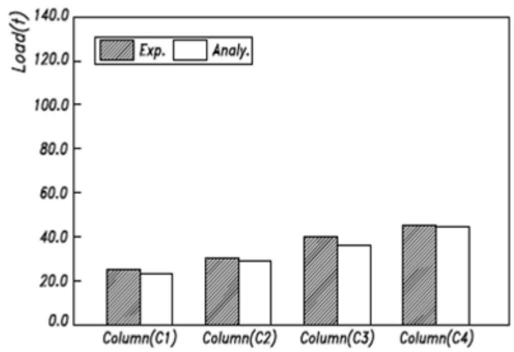


Fig.(9): Comparison between Experimental and Analytical Cracking Load

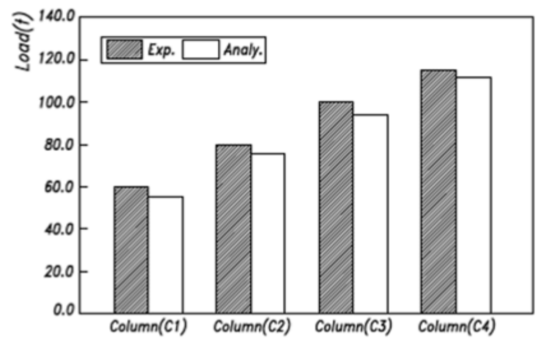


Fig.(10): Comparison between Experimental and Analytical Failure Load

Table (4) Relative values of the experimental and analytical cracking and failure loads

	Cracking load (ton)			Failure load (ton)		
Col. No.	Exp.	Analy.	Analy./Exp. %	Exp.	Analy.	Analy./Exp. %
C1	25.0	23.1	92.4	60.0	54.9	91.5
C2	30.0	29.2	97.3	80.0	75.4	94.3
C3	40.0	36.2	90.5	100.0	94.3	94.3
C4	45.0	44.6	99.1	115.0	111.5	97.0

From the table (4), it can be concluded that:

Analytical values were always smaller than experimental values. Analytical values of cracking load were smaller than the experimental ones by 0.9 % to 9.5 % ,while the analytical values of the failure load were smaller by a maximum of 8.5 % than experimental values.

3.2.2 Deflections

The comparison between experimental and analytical load-deflection curves at the mid-height of ground, first and second floor of all columns are shown in figures (11) to (13). In addition, the relative deflection values at the failure load of the tested and analyzed columns at the mid-height of ground, first and second floor are shown in figures (14) to (6-8).

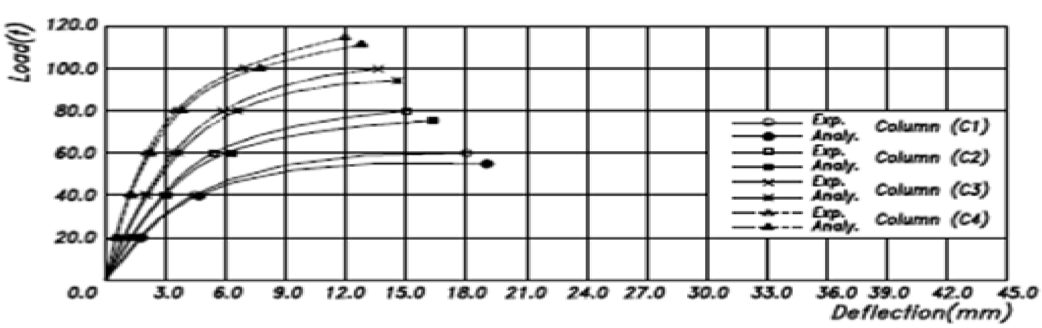


Fig.(11): Comparison between Experimental and Analytical Load–Deflection Curves at the Mid-Height of Ground Floor of Columns

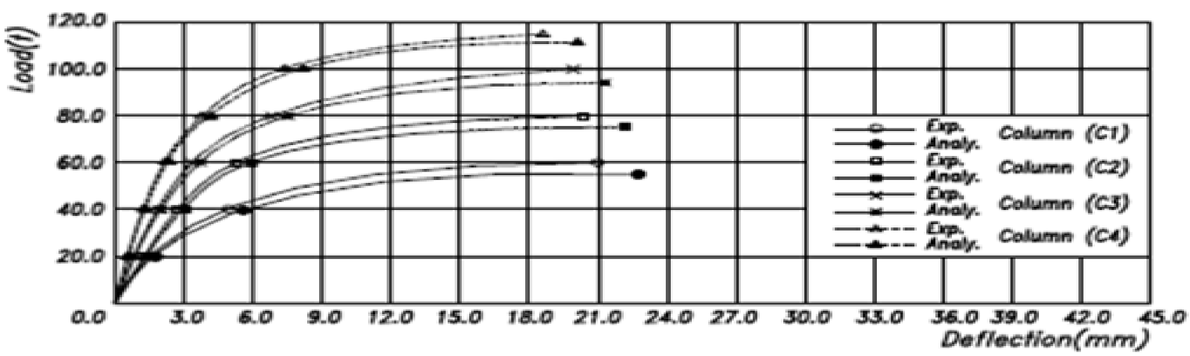


Fig.(13): Comparison between Experimental and Analytical Load-Deflection Curves at the Mid-Height of Second Floor of Columns

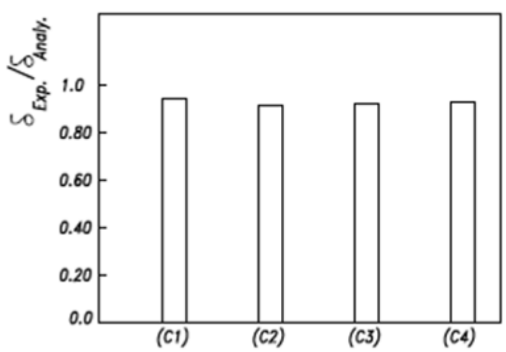


Fig.(14): Relative Deflections Values at the Failure Load of the Tested and Analytical Columns at the Mid-Height of Ground Floor

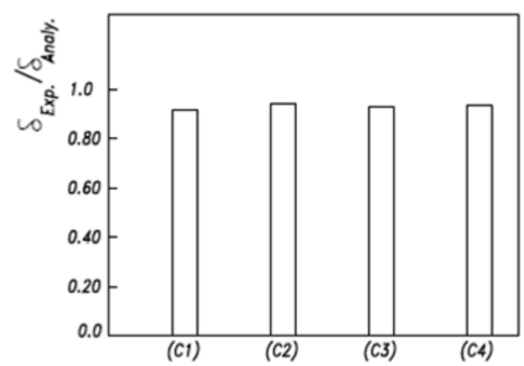


Fig.(15): Relative Deflections Values at the Failure Load of the Tested and Analytical Columns at the Mid-Height of First Floor

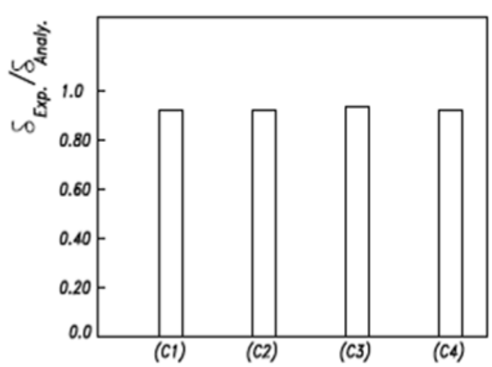


Fig.(16): Relative Deflections Values at the Failure Load of the Tested and Analytical Columns at the Mid-Height of Second Floor

3.3 Parametric Study

A comparative study was conducted by using ANSYS program to give more information about the behavior of the laterally supported columns under the following conditions:

1- Effect of changing the unsupported length of columns while keeping the inertia of the lateral bracing constant.
2- Effect of changing the inertia of the lateral bracing while keeping the unsupported length of columns constant.
3- Effect of changing both the unsupported length of columns and the inertia of the lateral bracing.

3.3.1 Description of the analyzed columns

A number of 27 reinforced concrete columns divided into 6 groups denoted as group (A , B , C , D , E and F) , each contained 4 columns, except group (A) which contained 7 columns, were analyzed.

The following properties were considered common to all columns of each group as shown in Table(3).

- Characteristic strength of concrete, f_{cu}
- Yield strength of steel, f_y
- The cross section of columns.
- Reinforcement of columns.
- The cross section of the lateral bracing in the short dimension.

Summary of the properties of different columns groups is shown in table (5).

Table (5) Physical properties of the analyzed columns

Group name	Column no.	e/t	Unsupported length ratio (l_c / b)			Rigidity factor of the lateral bracing (m^3) (I_b / l_b)
			G. F.	F.F.	S.F.	
A	AC1	0.375	15.0	9.44	9.44	$1.56x10^{-4}$
	AC2	0.25	15.0	9.44	9.44	$1.56x10^{-4}$
	AC3	0.125	15.0	9.44	9.44	$1.56x10^{-4}$
	AC4	0.0	15.0	9.44	9.44	$1.56x10^{-4}$
	AC5	-0.125	15.0	9.44	9.44	$1.56x10^{-4}$
	AC6	-0.25	15.0	9.44	9.44	$1.56x10^{-4}$
	AC7	-0.375	15.0	9.44	9.44	$1.56x10^{-4}$
B	BC1	0.375	30.0	18.88	18.88	$1.56x10^{-4}$
	BC2	0.25	30.0	18.88	18.88	$1.56x10^{-4}$
	BC3	0.125	30.0	18.88	18.88	$1.56x10^{-4}$
	BC4	0.0	30.0	18.88	18.88	$1.56x10^{-4}$
C	CC1	0.375	22.5	14.17	14.17	$1.56x10^{-4}$
	CC2	0.25	22.5	14.17	14.17	$1.56x10^{-4}$
	CC3	0.125	22.5	14.17	14.17	$1.56x10^{-4}$
	CC4	0.0	22.5	14.17	14.17	$1.56x10^{-4}$
D	DC1	0.375	15.0	9.44	9.44	$0.78x10^{-4}$
	DC2	0.25	15.0	9.44	9.44	$0.78x10^{-4}$
	DC3	0.125	15.0	9.44	9.44	$0.78x10^{-4}$
	DC4	0.0	15.0	9.44	9.44	$0.78x10^{-4}$
E	EC1	0.375	15.0	9.44	9.44	$1.04x10^{-4}$
	EC2	0.25	15.0	9.44	9.44	$1.04x10^{-4}$
	EC3	0.125	15.0	9.44	9.44	$1.04x10^{-4}$
	EC4	0.0	15.0	9.44	9.44	$1.04x10^{-4}$
F	FC1	0.375	30.0	18.88	18.88	$0.78x10^{-4}$
	FC2	0.25	30.0	18.88	18.88	$0.78x10^{-4}$
	FC3	0.125	30.0	18.88	18.88	$0.78x10^{-4}$
	FC4	0.0	30.0	18.88	18.88	$0.78x10^{-4}$

Where:

- l_c : column unsupported length.
- b : Short side of the column's cross-section.
- I_b : Moment of inertia of the lateral bracing beam.
- l_b : Length of the lateral bracing beam.
- *e/t* : { lateral eccentricity / long side of column's cross-section } ratio.
- *G. F.* : Ground floor.
- *F. F.* : First floor.
- *S. F.* : Second floor.

3.3.2 Cracking and failure loads

The cracking and failure loads, and the percentage of cracking load to failure load for all analyzed columns are shown in table (6)

Table (6) Cracking and failure loads of the analyzed columns

Group name	Column no.	Cracking load (ton)	Failure load (ton)	Cracking load % Failure load
A	AC1	29.2	75.4	38.73
	AC2	36.2	94.3	38.4
	AC3	40.3	102.1	39.5
	AC4	44.6	111.5	39.6
	AC5	42.2	107.6	39.2
	AC6	40.5	104.2	38.9
	AC7	37.4	101.4	36.9
B	BC1	25.4	58.1	43.7
	BC2	26.8	62.8	42.7
	BC3	27.9	70.7	39.5
	BC4	28.6	74.3	38.5
C	CC1	27.0	67.5	40.0
	CC2	29.9	72.5	41.2
	CC3	32.4	80.0	40.5
	CC4	34.1	85.0	40.1
D	DC1	26.6	70.1	37.9
	DC2	31.0	78.2	39.6
	DC3	34.3	86.3	39.7
	DC4	36.9	93.4	39.5
E	EC1	28.3	71.3	39.7
	EC2	33.0	84.3	39.1
	EC3	36.2	91.7	39.5
	EC4	40.5	102.8	39.4
F	FC1	18.4	46.0	40.0
	FC2	23.0	52.4	43.9
	FC3	25.5	60.1	42.4
	FC4	26.9	66.2	40.6

4. Calculating the load carrying capacity of columns using code equation with suggested modified factors.

A relation between the load carrying capacity and the lateral eccentricity / length ratio "e/t", Moment of inertia of the lateral bracing beam / Length of the lateral bracing beam "I_b / l_b" and Unsupported length of column / Short side of the column's cross- section "l_c / b" can be obtained by curve fitting using linear, exponential and power equations for columns with lateral bracing in the inner and outer half of the long dimension as shown in figures a , b and c. Apparently, the best fitting was achieved when the regression analysis factor (R^2) was maximum.

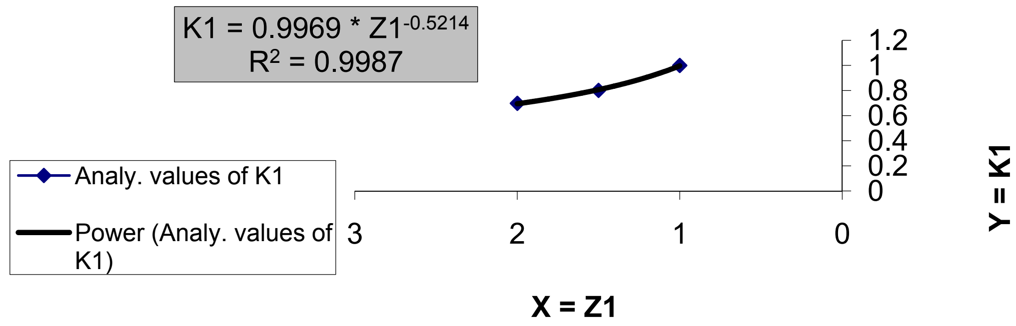


Fig.(a): Curve Fitting Equation of the Factor (K1)

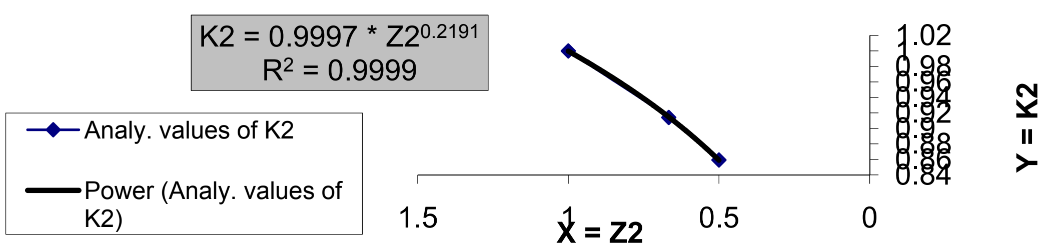


Fig.(b): Curve Fitting Equation of the Factor (K2)

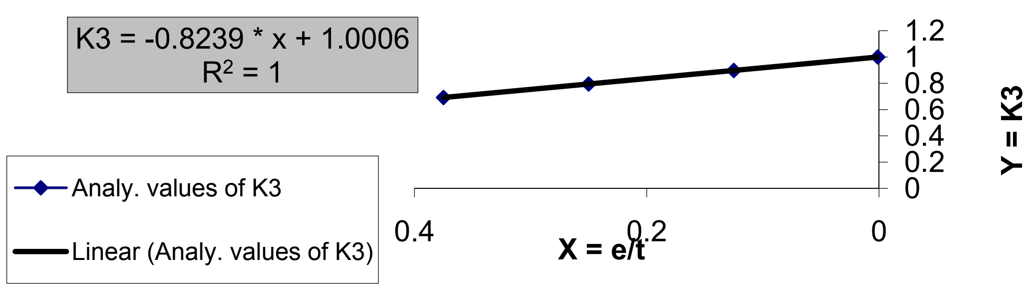


Fig.(c): Curve Fitting Equation of the Factor (K3)

Assuming that columns considered in the present study is part of a braced structure ; the ultimate load carrying capacity of the column (AC4) {short braced column with lateral supports in the short dimension and in the middle of the column's longer dimension }can be calculated by the following equation:

$$\mathbf{P_u = f_{cu} * A_c + f_y * A_{sc}} \quad \text{..........(1)}$$

$= 350.0 * 36.0 * 9.0 + 2400 * 4.15$
$= 123.36$ ton

The design load of the same column can be obtained through the Egyptian code equation ECP 203(2):

$$\mathbf{P_u = 0.35 * f_{cu} * A_c + 0.67 * f_y * A_{sc}} \quad \text{..........(2)}$$

$= 0.35 * 350.0 * 36.0 * 9.0 + 0.67 * 2400 * 4.15$
$= 46.36$ ton

It can be noticed that the difference between the design load of equation (2) and the ultimate load carrying capacity of equation (1) reflects the implied safe factor of the code design equation. The effect of the unsupported length ratio, the rigidity factor of the lateral bracing and the lateral eccentricity / length ratio can now be introduced to the code equation in order to preserve the same factor implied by the code. This is done through introducing the K_1, K_2 and K_3 factors to the code equation to obtain the design load of the tied columns in the form:

$$\mathbf{P_u = K_1 * K_2 * K_3 (0.35 f_{cu} A_c + 0.67 f_y A_{sc})} \text{ ...(4)}$$

Where:

P_u = Design load of tied columns, (ton).
f_{cu} = Characteristic strength of concrete.
A_c = Area of column's cross section.
f_y = Yield strength of steel.
A_{sc} = Area of longitudinal reinforcement.

$K_1 = 0.9969 (Z_1)^{-0.5214}$

Z_1 = Unsupported length ratio (l_c / b) of each group / unsupported length ratio (l_c / b) of group (A).

l_c / b	10	12	14	16	18	20	22	24	26	28	30
Z1	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2

$K_2 = 0.9997 (Z_2)^{0.2191}$

Z_2 = Rigidity factor of the lateral bracing (I_b / l_b) of each group / rigidity factor of the lateral bracing (I_b / l_b) of group (A).

I_b / l_b	0.8	1	1.2	1.4	1.6	1.8
Z_2	0.5	0.6	0.7	0.8	0.9	1

K_3 = A factor reflecting the effect of eccentricity of the lateral support
from mid-distance of the long dimension of column's cross section, and given by
the following equations:

$K_3 = - 0.8239 X + 1.0006$ for +ve "e/t" (X= "e/t") in the range of (0.0 – 0.375)
$K_3 = e^{0.2542 X}$ for -ve "e/t" (X= "e/t") in the range of (-0.375 – 0.0)

5. Conclusion

1- The presence of lateral supports in the shorter dimension and at the inner half of the longer dimension increased both the cracking and ultimate loads
2- Average measured values of deflection at the mid-height of ground, first and second floors levels in the case of the presence of lateral supports at the inner half of the longer dimension were smaller than that of the case of their presence in the column's shorter dimension only.
3- From the results of the parametric study, the following conclusions could be obtained:

a) Increasing the unsupported length ratio (l_c / b) of columns decreased both cracking and failure load.
b) Decreasing the rigidity factor of the lateral bracing (I_b / l_b) decreased both cracking and failure load.

4- A proposed equation for calculating the design load of the laterally braced tied columns which takes into consideration the effect of changing the unsupported length ratio, the rigidity factor of the lateral bracing and the lateral eccentricity / length ratio "e/t" was presented as follows:

$$\underline{P_u = K_1 * K_2 * K_3 (0.35 f_{cu} A_c + 0.67 f_y A_{sc})}$$

Where:
K1 unsupported length factor.
K2 lateral bracing rigidity factor.
K3 lateral support eccentricity factor.

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