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EVALUATION OF AXIAL CAPACITY OF HIGH STRENGTH STEEL COLUMN UNDER ECCENTRIC LOADS

[1]Assistant Lecturer. Adnan Kadhim Jawad, [2]Assistant Lecturer. Mohammed Jalil Jihad,

[3]Assistant Lecturer. Oday Jawad Kadhim

[1]University of Al-Furat Al-Awsat Technical, Al-Najaf Engineering Technical College, Building & Construction Technical Engineering Department
adnan.jewad@atu.iq
[2]University of Al-Furat Al-Awsat Technical, Al-Najaf Engineering Technical College, Building & Construction Technical Engineering Department
mohammadjalil482@gmail.com
[3]University of Al-Furat Al-Awsat Technical, Al-Mussaib Technical College, Building & Construction Technical Engineering Department
odaydoban@gmail.com

Abstract

Most of the current studies and developments which deal with the effect of eccentric axial compression load, especially at the column, in addition to it affects Columns behaviour with closely spaced transverse reinforcement improves significantly with the use of high-strength confinement steel. The use of larger bar diameters for longitudinal reinforcement produces little beneficial effect on the ductility of column, increasing ratio of longitudinal reinforcement in high strength concrete columns leads to an increase in column capacity but decreases its ductility, tie configuration is very effective in strength and ductility of high strength columns. In general, when axial load increases, the flexural ductility of the column decreases, as eccentricity increases, columns give more ductile behaviour in under and post-peak stage. this paper experimentally investigates the elastic buckling of steel columns with three different area cross-sections of the square cross-sections, different eccentric and two different boundary conditions, i.e. fixed-free(F-F) and pinned-pinned (P-P) boundary conditions, under axial compression. It is concluded that calculations made the P-P and the lowest differences in F-F at difference buckling load, also the square cross-section of P-P has the lowest slenderness ratios and square cross-section of F-F has the highest slenderness ratios and the buckling loads of P-P column is higher than F-F column.

Introduction

Over the last decade, the use of high-strength steel bars in the construction industry has prompted extensive research in this area. High-strength steel bars have the advantage of lowering reinforcement congestion and construction costs, especially in high-rise and special

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buildings. The use of high-strength steel as longitudinal reinforcement can enhance concrete members' load capacity; moreover, its use for stirrups may decrease the transverse reinforcement amount required to ease concrete placement. In recent years, the continuous development of steel smelting technology has produced new high-strength steel (for example, Grade 100 in USA, Grade 600 in Korea, and HRB600 in China). The new developed high-strength steel has a linear pre-yield behaviour, obvious yield plateau and comparatively good ductility, while ultra-high-strength reinforcing bars have a high yield strength, but no yield plateau and poor ductility. There have been many investigations on the performance of high-strength steel (including high-strength longitudinal reinforcement and transverse reinforcement) used in concrete beams column joints, and walls. As a result, the use of high-strength steel has become widespread in concrete structural applications, the pioneer experiments on the buckling of bars under centrally compression, were performed by Musschenbroek and then Euler investigated the elastic stability of a centrally loaded structures and given a formula for buckling of columns. Euler's formula reckon as the failure of a column stems from the stresses induced by sidewise bending only. This assumption is valid for long columns only, because the failure which occurs in short and medium columns stems from the combination of direct compression and bending. Euler developments of columns had been reviewed by Bleich and Timoshenko [1]. Elastic beam-columns were examined by Timoshenko and Gere. In the monograph of Brush and Almroth, the buckling of bars, plates and shells was investigated. **Shrivastava** presented in his study of the elastic buckling of columns under varying axial force was examined when a truss or open-web steel joist is used as a frame member, parts of the bottom chord near the ends carry compressive forces and adequate bracing must be provided to prevent lateral buckling of the chord. The compression in the chord varies from panel to panel; the problem is to determine the buckling load factor for a given loading and assumed location of the bracing point. Although the problem can be solved exactly, a simple procedure which may be used in the design office for rapid, hand computation of the buckling load factor in the elastic range [2] **Simitses** were given in the monograph of the fundamental concepts and the methodology developed through the years to solve structural stability problems [3] **Karabalis and Beskos**: developed a finite-element method based on an exact flexural and axial stiffness matrix for the static, dynamic, and stability analyses of beams with constant width and variable height. Based on the method of assumed modes [4] **Smith:** used the energy method to develop an analytic solution for determination of critical buckling load of a tapered column. [5] **Domokos et al.** considered the buckling of elastic columns with lateral defections constrained by rigid, frictionless side-walls both theoretically and experimentally, i.e. problems with nonlinearities arising due to contact with boundary conditions. [6] **Magnusson et al.** abandoned the commonly adopted assumption in the investigation of the post-buckling behaviour of compressed columns, of constant loading force in post-buckling equilibrium states, and investigated instead the behaviour of extensible elastic. [7] **Mazzilli** who analytically approached the problem of extensible elastic by loading not only axial forces but also transverse forces and bending moments at the column ends, and found post-buckled configurations in a number of cases. Structural design practice requires the incorporation of other factors besides material and geometrical nonlinearities, such as load-column system imperfections (e.g. initial curvature, non-uniformity of cross-section, eccentricity of load), heat affected zones, residual stresses or the interactions of various modes of deformation. [8] **Barbero et al.:** experimentally studied the interaction of local (flange) and global (Euler) buckling modes of a column, and revealed that upon interaction of the mechanisms a tertiary combined mode of buckling develops. They showed that the critical load value of the combined mode is lower than that predicted by the classic single mode Euler buckling model and that it is highly sensitive to imperfections. Nowadays in practice, the design of column-like structural elements largely relies on two modelling design tools. [9] **Bryan, and Turneaure**: recommended the Euler formula, preceded by a modifying constant that adjusted it to conform to available test data. Their modifications were equivalent to use of an equivalent length coefficient of K = 0.785 for pinned end columns and K = 0.628 for flat ended columns[10] **C. A. Ellis, and D. M. Brown:** studied the "Euler load" is the critical load at which a slender elastic column can be held in a bent configuration under axial load alone. In Euler's time columns were made either of masonry or timber, the latter being considered by Euler as subject to bending [11] **Batterman**: developed computer programs to determine maximum loads for aluminum alloy H-section columns, with finite web areas, about both the weak and strong axis bending, in both the straight configuration and with varying degrees of initial curvature. [12] **Zhang et al.:** study their cross-section local buckling behaviour and resistances, while the flexural-torsional buckling behaviour and resistances of press-braked angle section intermediate columns made of the new high-chromium stainless steel [13] The main objective of this research is to study elastic buckling of steel columns at two different boundary conditions, fixed-free(F-F) and pinned-pinned (P-P) boundary conditions under axial compression addition Study the behaviour of steel columns under eccentric compression using experimental and analytical programs.

Materials

Ten steel columns, which is tested under eccentric loads the materials used in this study are available locally and are selected from materials currently used in construction in Iraq.

Basic Equations

Consider an elastic column of length **L** loaded by an axial compressive load **P** with the action line coinciding with the z axis of a rectangular coordinate system Ozx with (F-F) and (P-P) boundary conditions respectively as shown in Figure (1.1) In here, dashed lines denote the buckled shape of the columns. Furthermore, the cross section of the column has been shown in Figure (1.1) a

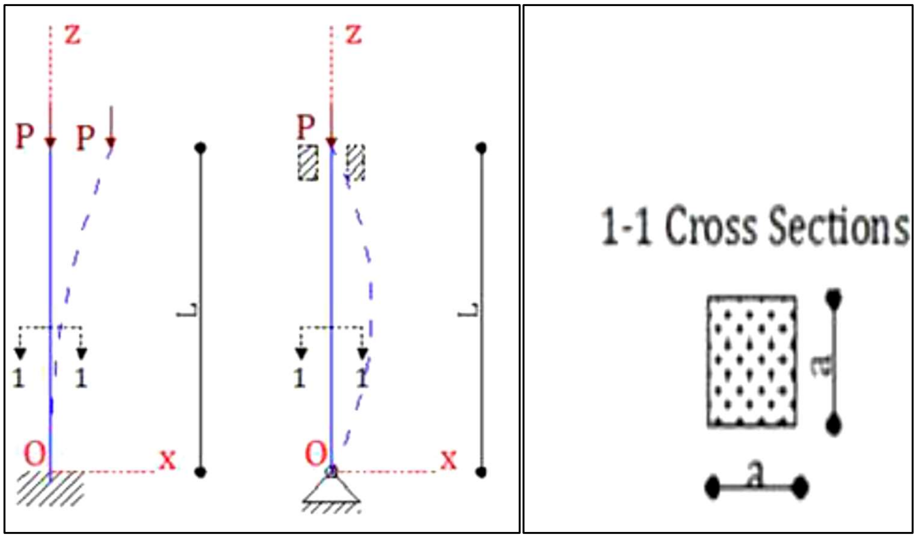


Figure (1.1) Geometry of column under axial compression

The governing equation for the buckling of such columns is:

$$EI\frac{d^4u}{dz^4}+P_{cr}\frac{d^2u}{dz^2}=0 \qquad (1)$$

where:

E = The Young's modulus of the column,

I = The area moment of inertia of the column cross section, and

u= The transverse displacement.

from Eq. (1) is modified the following equation yields:

$$\frac{d^4u}{dz^4}+k^2\frac{d^2u}{dz^2}=0 \qquad (2)$$

where the following definition applies:

$$k^2 = \frac{P}{EI} \qquad (3)$$

The general solution of Equation (2) as follows:

$$u(z) = C_1 \sin kz + C_2 \cos kz + C_3 z + C_4 \qquad (4)$$

where are C1, C2, C3 andC4 coefficients and can be identified with boundary conditions.

The boundary conditions satisfy the P-P column as follows:

$$u\big|_{z=0} = 0 \text{ and } \left.\frac{d^2u}{dz^2}\right|_{z=0} = 0 \qquad (13)$$

$$u\big|_{z=L} = 0 \quad \text{and} \quad \left.\frac{d^2u}{dz^2}\right|_{z=L} = 0 \qquad (14)$$

Substituting Eqs. (13), (14) into (4), and after some mathematical rearrangements, the determinant of coefficient matrix and its solution found as follows:

$$\begin{vmatrix} 0 & 1 & 0 & 1 \\ 0 & -k^2 & 0 & 0 \\ \sin(kL) & \cos(kL) & L & 1 \\ -k^2\sin(kL) & -k^2\cos(kL) & 0 & 0 \end{vmatrix} = -k^4 L \sin(kL) = 0 \qquad (15)$$

$$kL = n\pi \qquad (16)$$

$$k^2 = n^2\left(\frac{\pi}{L}\right)^2 \qquad (17)$$

$$\frac{P}{EI}=n^2\left(\frac{\pi}{L}\right)^2 \qquad (18)$$

$$P=n^2\left(\frac{\pi}{L}\right)^2 EI,\ \ n=1,2,... \qquad (19)$$

Consequently, the critical buckling load occurs when n = 1and we get [4]:

$$P_{cr}=\left(\frac{\pi}{L}\right)^2 EI_{min} \qquad (20)$$

The buckling load can be expressed as the following general form also:

$$P_{cr}=\frac{\pi^2 EI_{min}}{L_{eff}^2} \qquad (21)$$

Where:

Leff = the effective length of column

Leff =2L for F-F

Leff L = L for P-P columns

Design of Experiment:

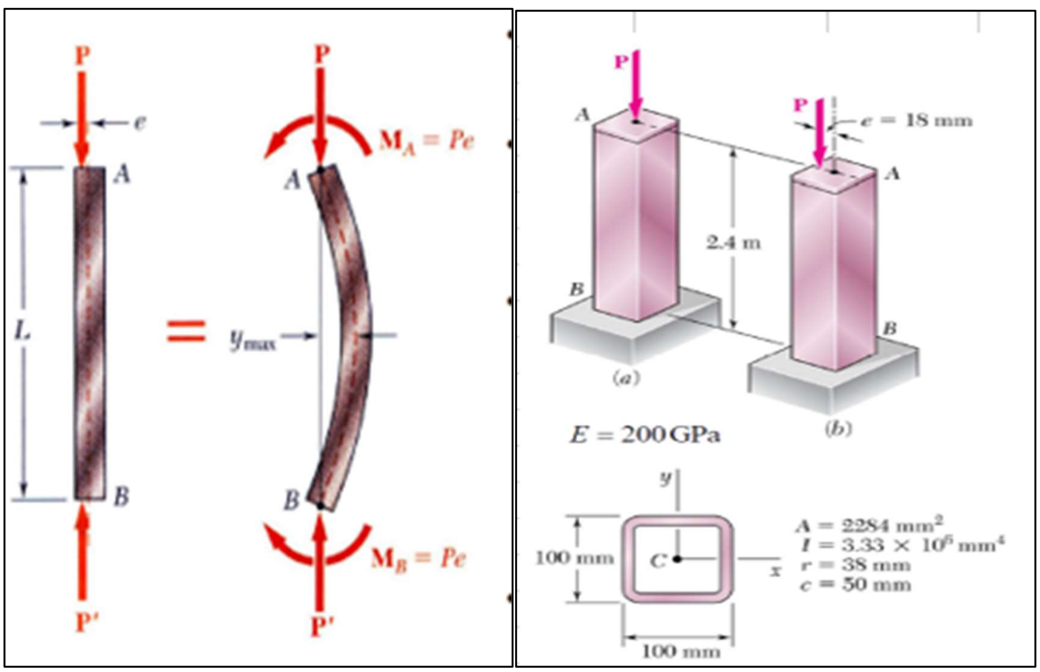
In eccentric compression tests, was evaluated on high strength steels stub column under a high performance steel bending and compression of buckling strength. The lists of the specimens were summarized in Table 1.1.

Table (1.1) Steel Stub Columns Specimens.

	BCs	L (m)	e (m)
	Fixed-Free	**4.8**	**0.018**
e	**Pin-pin**	**2.4**	**0.01**
	Pin-pin	**2.4**	**0.015**
	Pin-pin	**2.4**	**0.02**
E	**Pin-pin**	**2.4**	**0.015**

	Pin-pin	2.4	0.015
	Pin-pin	2.4	0.015
e	Pin-pin	2.4	0.015
	Pin-pin	2.4	0.015
	Pin-pin	2.4	0.015

In table 1.1, briefly explain the specimens of steel column, and boundary condition which that fixed-free, pin-pin. The modulus of elasticity was assumed at 200000Mpa. The length (L) of specimens was 2.4-4.8. Specimens were designed as axial load is 142700 and 200000 for the combined force of the various conditions to evaluated the performance by adjusted the eccentric distance. The specimens while under eccentric load department specimens were designed. Eccentric compression tests for built-up L-section and built-up as square columns specimens were set up as shown in Figure 1.2



(a) (b)

Figure 1.2 (a)Buckling of axially loaded compression members with eccentric loading (b) properties of column

Test Specimens:

Buckling of columns with pinned ends is often called the fundamental case of buckling. Addition, many other conditions such as fixed free ends are encountered in practice. The critical forces for buckling for each of these end conditions can be determined by applying the appropriate boundary conditions and solving the differential equations. These solutions lead to the concept of an "effective length, Le, appropriate for each end condition which is a multiple of the actual length, L, of the column as shown in Table 1.2 and Figure 1.3.

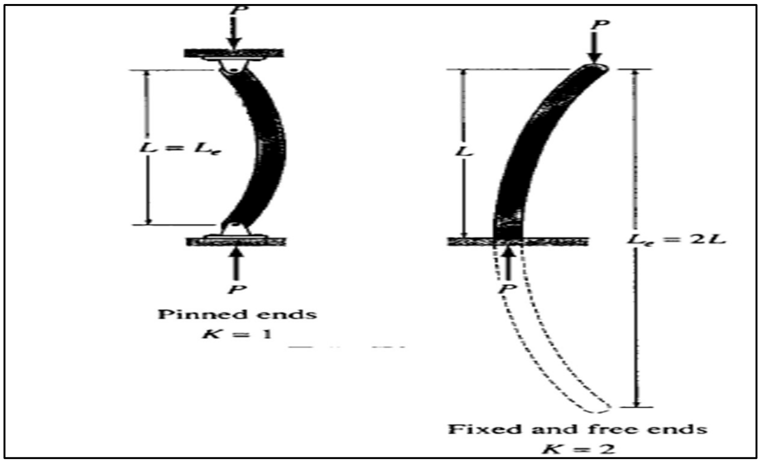


Figure 1.3 Illustration of end conditions for columns

- Measure the diameter and lengths of each specimen to 0.02 mm.
- Zero the force output (balance).
- Activate force protect (~50 N) on the test machine to prevent overloading the specimen during installation.
- Install the top end of the test specimen in the top grip of the test machine while the test machine is in displacement control.
- Install the bottom end of the test specimen in the lower grip of the test machine
- In displacement control adjust the actuator position of the test machine to achieve nearly zero force on the specimen.
- Deactivate force protect.
- Initiate the data acquisition and control program.
- Continue the test until buckling or compressive failure of the test specimen occurs
- Examine the force versus displacement trace for each test. Note the force at the onset of buckling or compressive failure (i.e., significant deviation from linearity)

Axially Loaded Steel Columns:

In Table 1.2 1illstrate the properties of column addition the variation of the boundary condition of steel column in Table 4.1 E which mean elastic modulus, area of square cross section, 100x100 mm

1.2 Properties of column addition the variation of the boundary condition of steel column

BCs	E (Pa)	A (m2)	I (m4)	r (m)	c (m)	L (m)	K	e (m)
Fixed-Free	2.00*1011	2.28*10-3	3.33*10-6	0.038	0.05	4.8	2	0.018

Pin-pin	2.00*1011	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.01
Pin-pin	2.00*1011	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015
Pin-pin	2.00*1011	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.02
Pin-pin	5.00*1010	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015
Pin-pin	1.00*1011	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015
Pin-pin	1.50*1011	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015
Pin-pin	2.00*1011	1.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015
Pin-pin	2.00*1011	2.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015
Pin-pin	2.00*1011	3.28*10-3	3.33*10-6	0.038	0.05	2.4	1	0.015

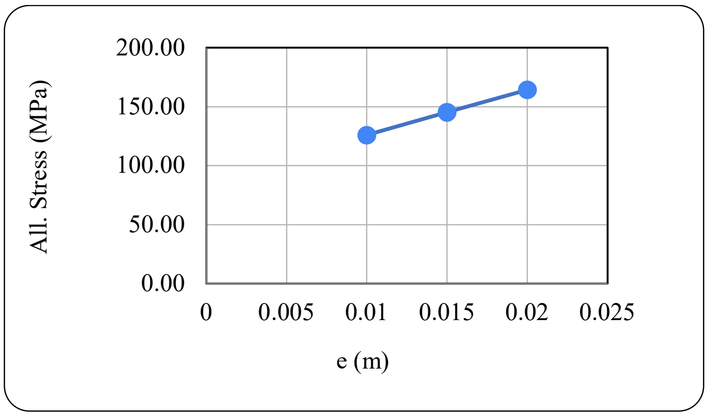


Figure1.4 Relationship Between allowable stress and eccentric.

The columns subjected to small-eccentric which designed to study the effect of load eccentricity. which contains eccentricities of 0.018, 0.01, 0.015 and 0.02 of column thickness, respectively. As the eccentricity of the applied loads increased, the depth of the compressed zone at mid-height decreased. In Table 1.3 variations of the eccentric versus length, which are obtained from the formula of Euler, for steel columns with two different F- F, P-P ends are given. In figure 1.5 illustrate the relationship between allowable stress with various e which show a decrease in e against the increase in allowable stress. As the buckling loads are compared with the results of the formula of Euler. where used the General Formulation for F-F and P-P Columns

Table 1.3 the value of load, critical load, allowable stress, critical stress and slenderness ratio

P (KN)	Pcr(KN)	All. Stress (MPa)	Slenderness ratio	Critical Buckling Stress(Mpa)
142.7	70.51626	151.18	126.32	30.9

200	1128.26	125.97	63.16	494
200	1128.26	145.17	63.16	494
200	1128.26	164.37	63.16	494
200	282.065	271.93	63.16	123
200	564.13	164.10	63.16	247
200	846.195	150.49	63.16	370
200	633.409	282.94	63.16	493
200	1128.26	145.17	63.16	494
200	1623.111	98.03	63.16	494

All the columns showed similar behaviour under the eccentric loading except the column with F-F end show different behaviour. the failure of the column specimens in all cases was characterised by a very loud failure. the lowest buckling loads are found in column with F-F, and so it is the least efficient boundary condition of the column against buckling, for the examined problem.as shown in figure 1.5

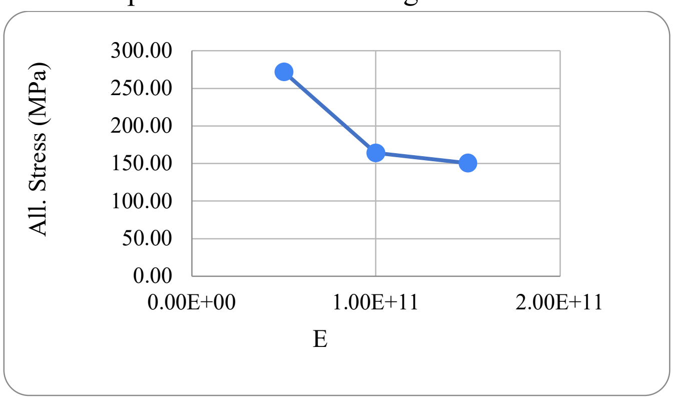


Figure1.5 Relationship Between allowable stress and Elastic Modulus.

In figure 1.6 shows the relationship Between allowable stress and Elastic modulus. The difference in the elastic modulus values shows the inverse relationship between all. Stress and the elastic modulus in terms of increasing the elastic modulus values corresponding to a decrease in the all. stress valuess.

While in Table 1.4 variations of the buckling loads of column for steel columns with versus length under F-F, P-P boundary conditions are presented. One of the dominant parameter acting on the elastic buckling of the column is the slenderness ratio

Slenderness ratio = $\frac{L}{r}$

r = The radius gyration $\Rightarrow \sqrt{\frac{I_{min}}{A}}$

As it is seen from the above results that the column with P-P the boundary condition has the lowest slenderness ratios, and the column with F-F has the highest slenderness ratios for the investigated problem and note that the slenderness ratio increase with the increase of the length of the column. In addition, it is observed that the buckling loads decrease with the increase of slenderness ratio, in all columns. Consequently, it is observed that the effects of the variation of slenderness ratios on the buckling loads increase with the increase of the length of the column and remains in a the almost same interval for all cross-sections, under F-F, P-P boundary conditions.

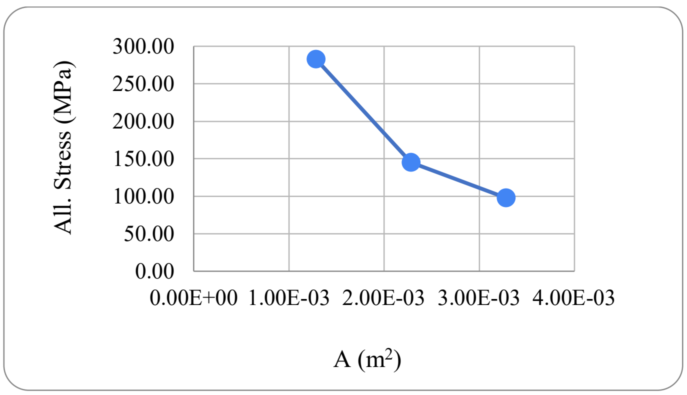


Figure1.6 Relationship Between allowable stress and Area.

Let changed the column section area values, it is noticed that this change causes an effect on the value of r and thus an effect on the values of the critical load as shown in Figure 4.3 shows the relationship between the values of the variable area with the all. stress, and this means the decrease in the all. stress values by increasing the area.

Finally, in Table 4.2 the buckling loads of F-F and P-P steel columns for L = 2400 and 4800mm, which are shown in Figs. 3-2, respectively, are compared in each other, it is seen that the buckling loads of P-P columns are higher than those for F-F columns. In addition, the boundary conditions have a constant influence on the buckling loads in set of column, approximately 93.3%,75%,87%. where the following expressions are used for the calculation of the percentages

$$\left(\frac{P_{crP-P} - P_{crF-F}}{P_{crP-P}}\right) \times 100$$

Consequently, it is observed that columns with P-P conditions have more resistant against buckling than F-F columns.

Conclusion:

Through the findings of laboratory tests and analysis. In this project, elastic buckling of steel columns with different F-F and P-P boundary conditions under axial compressive load is studied. The effects of the boundary conditions and slenderness ratios on the buckling load of the steel column have been discussed. And briefly, the following results are obtained for the investigated problem:

1. The highest differences between F-F and P-P boundary conditions. computation occurs in the P-P and the lowest differences in F-F at difference buckling load.
2. The square cross-section of P-P has the lowest slenderness ratios and square cross-section of F-F has the highest slenderness ratios.
3. The effects of the variation of slenderness ratios on the buckling loads increase with the increase of the length of the column in both of F-F and P-P boundary conditions.
4. As the convenient buckling loads of F-F and P-P columns are compared, the buckling loads of P-P column is higher than F-F column.

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