Ministry of Higher Education
and Scientific Research
University of Diyala
College of Engineering

ELASTIC POST-BUCKLING BEHAVIOR OF SEMI-RIGID CONNECTIONS PLANE STEEL FRAMES USING FINITE ELEMENT METHOD

A Thesis Submitted to Council of College of Engineering, University of Diyala in Partial Fulfillment of the Requirement for the Degree of Master of Science in Civil Engineering

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January 2019 **Rabi Al-Thani, 1440**

DEDICATION

Every work we done, needs support and guidance especially from people closer to our hearts

To the one whom I missed and was not beside me when facing difficulties

….. My father (Allah have mercy on him)

Who taught me self-independence and suffered the difficulties in reaching what I became

.... My mother

For those who supported me and encouraged me

... .My life partner

…. My husband

ACKNOWLEDGEMENTS

"In The Name of Allah, Most Gracious, Most Merciful"

Praise be to Allah, for helping me to accomplish this work.

It is a great pleasure to acknowledge my thanks and gratitude to my supervisor Asst. Prof. Dr Murtada Ameer Ismael for his valuable guidance and his advices throughout this work.

I would like to thank all members at Department of Structure Engineering, especially Prof. Dr Amer M. Ibrahim for giving me some advices that were of great benefit.

All thanks to my family especially my husband, who shared the difficulties of this work with me.

Finally, I would like to thanks to everyone who helped me in one way or another in this work

ABSTRACT

Estimating post-buckling behavior for structures have slender elements is very important task, since post-buckling state means loss the structures stability related with large displacement and that lead to destruction the structures.

On the other hand, in the design and analysis of steel frame, the beam-columns connection is assumed perfect pin or fully rigid, this assumption leads to incorrect estimation of the structural behavior. Practically, beam-column connection is between these two assumptions and this type of connection is called semi-rigid.

This study presents a numerical analysis using finite element method by ANSYS program to study the influence of the semi-rigid connections on post-buckling behavior of plane frames subjected to static loads. The influence of the semi-rigid beam-columns connections was studied on different frame cases such as: supporting type, number of frame stories, state of bracing (braced and unbraced), presence of lateral loading, length of frame span, depth of the frame beam and modulus of elasticity of the frame elements, also a comparison was achieved between the variables of each case.

The semi-rigid connections were modeled as linear elastic rotational spring, using COMBIN14 element which permit to change its rotational stiffness value. The material of frames is assumed to be in elastic stage. Arc-length technique was used to predict the non-linear behavior of large displacement arising from buckling.

The numerical analysis contains two basic parts, the first part includes modeling of three frame types with semi-rigid connections as well as the rigid connection case and the numerical results of these frames were compared with that of the previous studies to check the validity and accuracy of these finite element models. The results of the numerical study showed a good agreement with that of the previous studies. In the second part, the effects of the semi-rigid beam-columns connections were studied on different frame cases.

The numerical results showed that; the effect of changing the beam-column connections from rigid to semi rigid with rotational joint stiffness 25EI/L to 15EI/L and 10EI/L leads to decrease the initial peak load of the frames of fixed-fixed supports with percentages 3.36 %, 5.6% and 8.95% respectively as compared with that of the rigid connection frame, While, the initial peak load not affected by changing the joint stiffness for fix-pin and pin-pin support cases.

Also the results of analysis showed that, the effect of changing joint stiffness on vertical displacement in three-storey frame is significant more than in two-storey. Increasing depth of beam and modulus of elasticity lead to increase the initial peak load and decrease vertical displacement and make the frame more affected by changing joint stiffness. Bracing the frame by single diagonal and presence of lateral load make the frame less affected by changing joint stiffness.

Thus, the semi-rigid connections should be considered in analysis and design of the steel frame to obtain more realistic results.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

List of Symbols

A	Area of the bar
a	Curve fitting parameter
b	Curve fitting parameter
C_1	Curve-Fitting constant
C_2	Curve-Fitting constant
C_3	Curve-Fitting constant
d	distance
dV	infinitesimal volume of the element
dw/dx	Slope of curvature
E	Modulus of elasticity
F	Axial force in the bars
g_1	gusset plate length
g_2	gusset plate length
h	The rise of the frame
I	Moment of inertia
i	subscript representing the current equilibrium
jd	user-defined desired number of iterations for convergence
J_{i-1}	actual number of iterations required for convergence in the (i-1) load step
K	Rotational stiffness of a connection
k	Standardization constant
L	Length of beam
L_e	Effective length
L_i	Arc-length
M	Bending Moment
n	total number of elements
P	Applied load
P_{cr}	Euler's buckling load
R	Radius of curvature
S	Length of compressed bars
W_{ext}	External work
W_{int}	Internal work
λ	Rigidity index

ϕ	Curvature
Θ	Rotation between the beam and the column
$[K^e]$	Element stiffness matrix
[B]	strain-nodal displacement relation matrix
[D]	constitutive matrix
[K]	overall structural stiffness matrix
$[K_i^T]$	Tangent stiffness matrix
[N]	shape function matrix
$\{\Delta u\}$	tangent displacement (mm)
{F}	nodal forces applied to the element
$\{F^a\}$	vector of applied loads (total external force vector)
$\{F_i^{nr}\}$	Vector of restoring loads corresponding to the element internal loads.
{u}	local displacements)
{U}	global displacements
$\{\sigma\}$	Element of real stress vector
$\{\partial\varepsilon\}$	Element of virtual strain vector
Δ	Axial shortening of each bar
$\Delta\lambda_i^1$	initial increment of the load parameter

List of Abbreviations

AISC	American Institute of Steel Construction
ASD	Allowable Stress Design
EI	Flextural rigidity
FEM	Finite Element Method
FR	Fully Restrained
PR	Partially Restrained

CHAPTER ONE

1.1 Introduction

Structures which has slender elements, after reaching the applied loads to buckling loads value, the loads remain unchanged or it decrease, but the deformations continue to increase, for some cases, after a certain value of deformation, the structure begins to carry more loading to retain the continuity of deformations, and the second buckling occurs. This cycle may be repeat several times, this phenomenon is called post-buckling behavior (Benson and Hallquist, 1990). It important to estimate the post-buckling behavior for the structures having slender elements, since post buckling state means loss the structure stability related with large deformations and that lead to destruction the structures.

On the other hand, in recent years, the steel structures have been widely used. These structures usually consist of different elements such as; beams, columns, and connections between them. The connections between beam and columns are an integral element of the steel frames, and it has important effect on the performance of these structures. In most engineering practice, the connections are assumed to be fully rigid or perfect pin, but in actual practice, the connections are not providing ideal rigid or perfect pin. The fact that, the ideal rigid connections provide flexibility and ideal pinned provide rigidity. Therefore, the behavior of connections reality fall between rigid and ideal pin, and these connections should be considered as semi-rigid steel connections (Dave and Savaliya, 2010).

1.2Buckling and Post-buckling Behavior of Structures

For a long time, engineers have been interested in the buckling of structures, but they did not observe the behavior of structures after the onset of buckling. The agreeable idea was that the buckling load was the ultimate performance of the structure, that for the reasons of safety, the actual load had to be kept far below the critical load, and the research on post-buckling behavior had no practical significance (Van der Neut, 1956).

The problem of buckling in steel structures has grown in the last two decades due to several interrelated developments. The need to provide large span of structure without bracing or intermediate support and also to provide very small ratio of weight to the unit area of structure due to economic considerations, made the buckling capacity of these structures to determine for their design.

1.2.1 Buckling

Buckling is an instability state of structural member which leads to a failure mode. When a slender structure is loaded in compression, for small loads it deforms without any noticeable change in the geometry and load carrying capacity. At the point of critical load value, the structure suddenly experiences a large deformation and may lose its ability to carry load. This stage is the buckling stage. (Deshpande, 2011).

1.2.2 Type of Buckling

1.2.2.1 Euler buckling

Euler buckling is possibly the most widely encountered and studied buckling mode, both in theory and in practice. When an imperfect strut or column is subjected to an axial force, it has a tendency to deflect laterally along its length. For practical purposes, the bifurcation and the plastic collapse loads are only the upper bound solutions to the actual failure load.

For the simplest case of a column under compressive axial load, the equation for Euler's buckling load can be obtained analytically as:

$$P_{cr}=\frac{\pi^2 EI}{L_e^2} \quad \text{…….. (1.1)}$$

In which Le is the effective length of a column, P is the axial force and π is a constant equal to 3.14159 (Chan and Chui, 2000).

1.2.2.2 Lateral buckling of beams

When a member is under the predominant action of bending moment about its principal major axis, it may deflect and bend about its minor axis when the applied moment is close to its critical moment. The elastic buckling moment can be obtained by the solution of the differential equilibrium equation (Trahair, 1965) or the energy equation using the principle of total potential energy (Chan and Kitipornchai, 1987a). Figure 1.1 sketches the lateral buckling analysis of a cantilever beam of rectangular cross section.

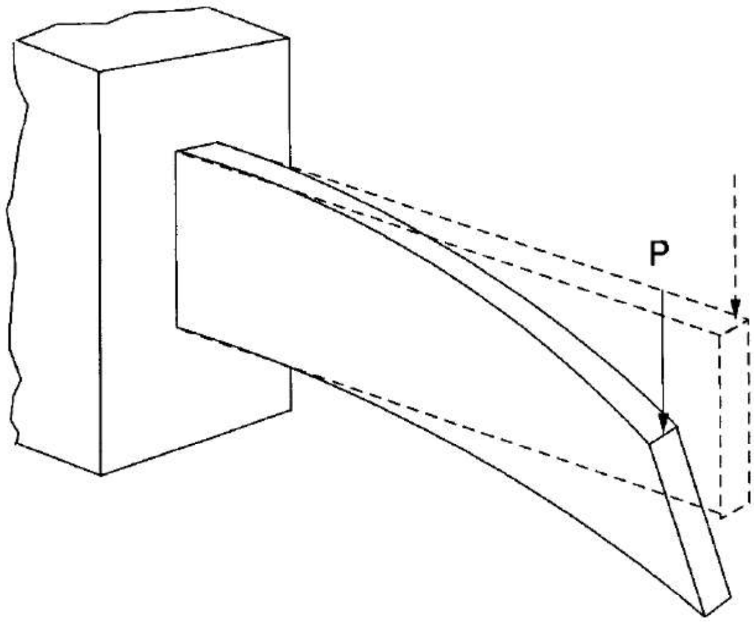

Figure (1.1) Buckling of a cantilever of rectangular cross-section (Chan and Chui, 2000)

1.2.2.3 Torsional buckling

A short thin-walled column under axial force may buckle and twist torsionally. This mode of buckling may be found in short and open section such as angle and cruciform, which is used as the supporting base of space frames shown in Figure (1.2) (Chan and Chui, 2000).

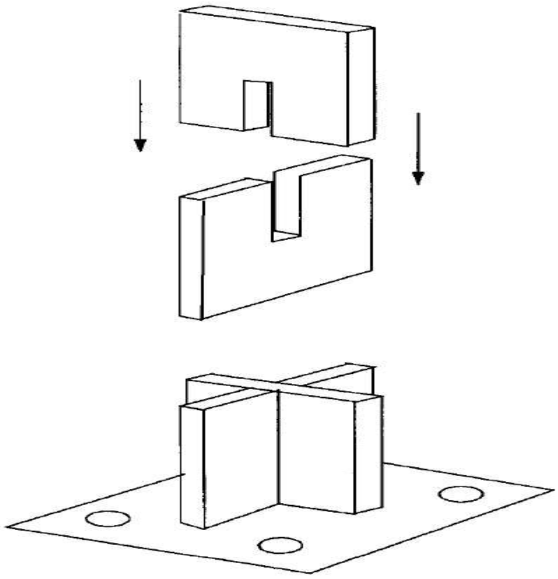

Figure (1.2) Cruciform for support base (Chan and Chui, 2000)

Short thin-walled columns under axial force may twist torsionally before reaching the squash load. Kitipornchai and Lee (1984) conducted experiments on the elasto-plastic buckling behavior of angle with varying length and Chan and Kitipornchai (1987b) proposed a finite element approach to the elastic buckling analysis of angle struts. The elasto-plastic buckling load was also predicted by Chan (1989) using the finite element approach which compared well with the test results by Usami (1971).

1.2.2.4 Local buckling

In the conventional theory of thin-walled beams, the cross section is assumed to remain undeformed when loaded to its ultimate strength. For the majority of hot-rolled sections, this assumption is valid and the result obtained on this basis is accurate. Using this assumption, theories developed for analysis of thin-walled beam-columns can be simplified and the time required for the analysis of this type of structures is greatly reduced. When the thickness of the plate components making up a section is thin, the cross section of the member may deform and exhibit local plate buckling before the member yields or buckles in any other modes previously described. Figure 1.3 shows the local plate buckling (Chan and Chui, 2000).

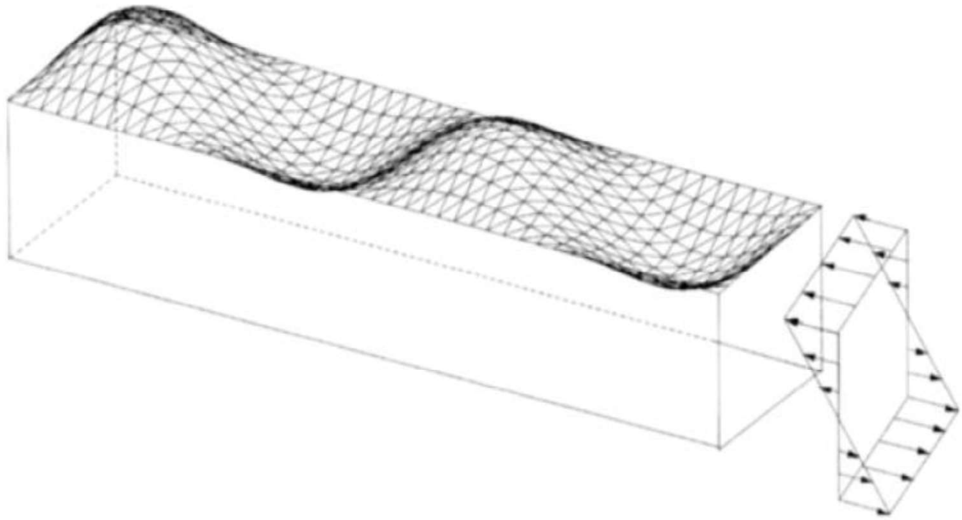

Figure (1.3) Local buckling of top flange of a box section (Chan and Chui, 2000)

1.2.3 Post-buckling

Post-buckling behavior means the behavior of structure after the first load limit point even when the geometry of structures is changed (Ramm, 1981).

A system is said stable if the small load causes a small response. Similarly a system is said to be unstable if small load causes large response. In terms of structural stability, as long as the structure remains in stable equilibrium, only one path is available for load-deflection curve satisfied the equilibrium condition. In addition to non-linear behavior in terms of stiffing or softening system, two phenomenon are of particular interest: occurrence of limit point and bifurcation of the path into several branches (Al-Mahdawi, 2001).

The instability of buckling classified as: bifurcation buckling and Limit load buckling. In the bifurcation buckling the deflections change from direction to another one. The load occurs at the bifurcation are a critical buckling load. A deflections path that occur before the bifurcation is called as a primary path and after bifurcations called a secondary or post-buckling (Deshpande, 2011).

In case of limit point, the process of solution can only continue beyond the limit point by decreasing applied load. Bifurcation a new equilibrium path occurs suddenly by branching, it is not only important to establish a bifurcation point exist, but it should be able to proceed with solution along the secondary branch.

To explain many of main characteristic of elastic buckling and post-buckling of structures, a simple frame is used. The frame consisting of two bars hinged together and at support as shown in Figure (1.4).

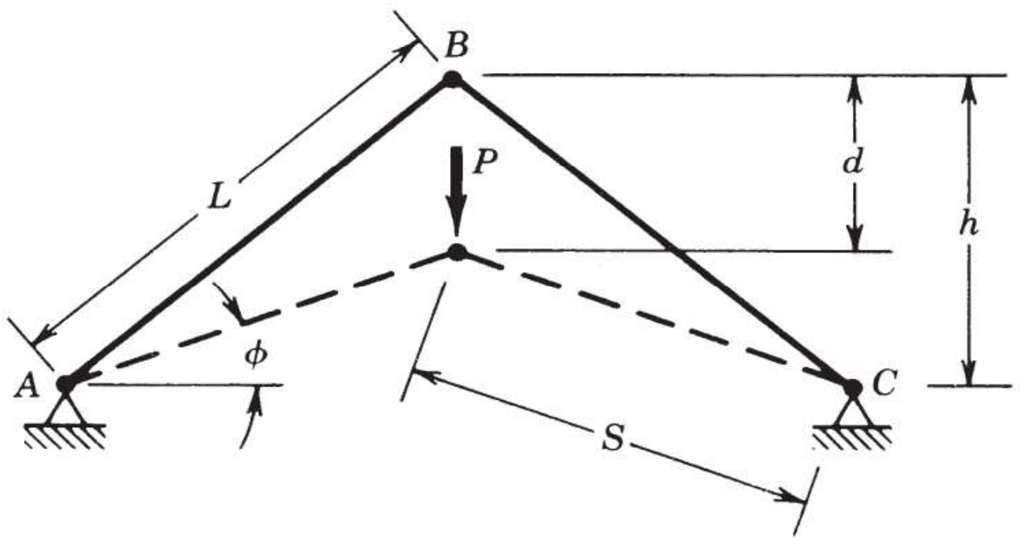

Figure (1.4) Limit-load buckling model (Ziemian,2010)

The load (P) acting on point B. The point B moved down with a distance (d). Axial force (F) that formed in the bars by applied loads P, expressed as:

$$\mathbf{F}= \frac{P}{2Sin\o} = \frac{PS}{2(h-d)} \qquad \text{…… (1.2)}$$

The members AB and BC shorten by an amount Δ, and can be given as:

$$\Delta=\frac{F}{K} =\frac{PS}{2K(h-d)} \qquad \text{…… (1.3)}$$

In which $S = \sqrt{L^2 + d^2 - 2dh}$ is the length of compressed bars.

where **K** is the stiffness of the bars given by:

$$\mathbf{K}=\frac{AE}{L} \qquad \text{……. (1.4)}$$

Substitution of Δ=L-S, $S=\sqrt{L^2 + d^2 - 2dh}$ and $F= \frac{PS}{2(h-d)}$ in equation (1.2) lead to:

$$L-\sqrt{L^2 + d^2 - 2dh} =\frac{P\sqrt{L^2 + d^2 - 2dh}}{2K(h-d)} \qquad \text{…… (1.5)}$$

If the rise (h) is assumed to be small compared with (L), equation (1.4) reduces to:

$$P = \frac{Kh3}{L^2} (2\delta - 3\delta^2 + \delta^3) \qquad \text{….. (1.6)}$$

In which $\delta = \frac{d}{h}$

The load-deformation relation corresponding to equation (1.6) is represented by solid curve in figure (1.5). It is evident that no bifurcation of equilibrium exists. Instead, the load and deformation increase simultaneously until a maximum or limit load reached at point (1), beyond this point the system becomes unstable.

If the rise (h) is large enough compared with (L), the axial force will reach their values causing the legs to buckle as hinged-hinged columns before the entire system reaches it is limit load at point (1). In that case buckling occurs as a result of a bifurcation of equilibrium at point (5) on the curve (Ziemian, 2010)

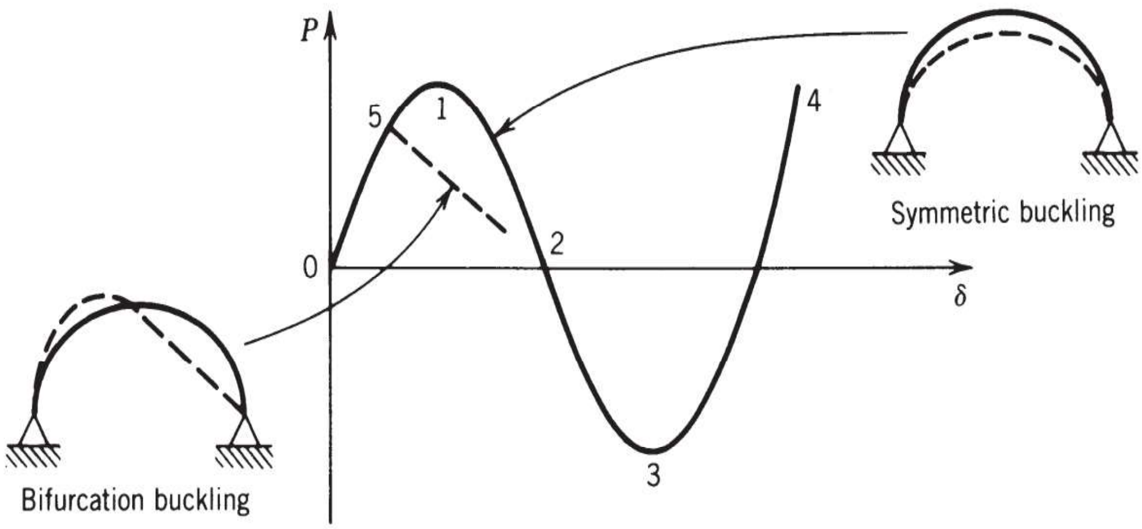

Figure (1.5) Load-deflection curve for limit-load buckling model (Ziemian,2010)

1.3 Semi-rigid Connections

Steel frames with beams and columns are most traditional system in construction. The structural performance of a steel frame is closely attached to the behavior of the beam-column connection. The connections are a medium that transferred the moments and forces from one member to another, such as from the beam to the column. In a plane frames, the beam-column connections transferred forces from beam to column include axial forces, shear forces and the bending moments. Two types of beam-column connection are considered in the design and analysis of steel frame. Steel connection may be assumed fully rigid or perfect pin. These assumptions lead to an incorrect estimation of structure behavior. In reality the connection is between the two assumptions and having some rotation stiffness (Khalifa et.al, 2011).

The stiffness of connection plays a significant role on ultimate carrying capacity of the frames. Ignoring stiffness of connections, leads to an idealized response of structure that may be unreal comparing to that of an actual behavior of structures.

Number of experiment investigations on the behavior of connections have demonstrate that a simple connections possess some amount of rotation stiffness, while the rigid connections possess some degree of flexibility (Nethercot et.al, 1998).

Bolted and welded connection rotate at an angle to the applied moment (Khalifa et.al, 2011), that leads to the fact that there are no rigid or pinned connections but there is a connection that has rotational stiffness with different values to make it behave either close to rigid or simple, this connection is called semi-rigid connections.

Figure (1.6) shows the realistic behavior of the beam-column connections (Chen and Lui, 1987). Where ϴ represent the rotation between the beam and the column, M is the moment and K is rotational stiffness.

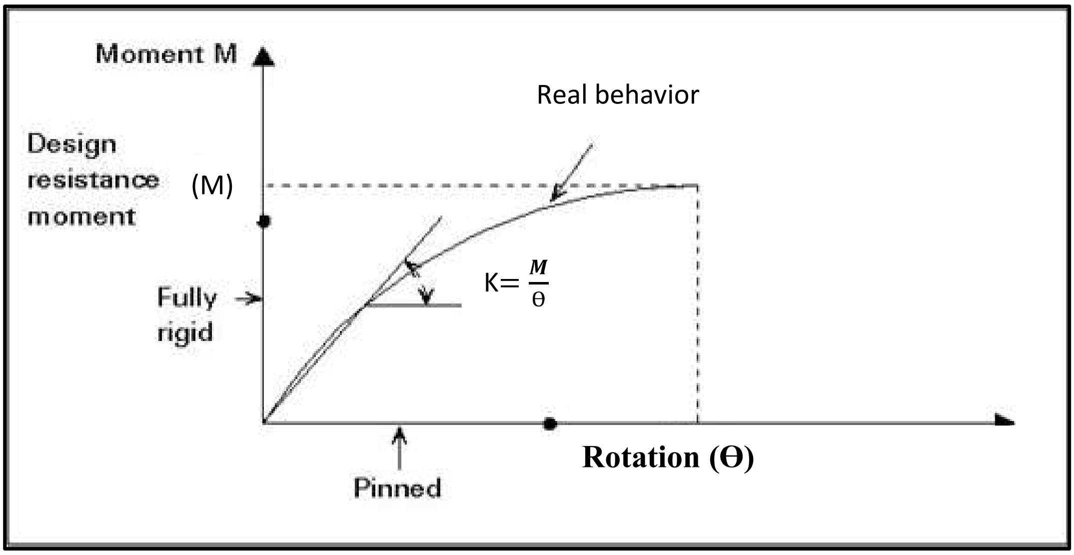

Figure (1.6) Moment-Rotation relationship of beam-column connection (Chen and Lui, 1987)

The behavior of pinned connection is represented by the ϴ-axis with M=0, the behavior of fully-rigid connection is represented by the M-axis with ϴ=0. The semi-rigid connections are represented by curve lying between these two extremes, allowing some moment to be transferred and some rotation to occur in a connection (Kruger et al, 1995).

1.4 Research Objectives

Safe and economical design of any structures can be achieved only if the behavior of these structures is adequately understood. These complications in the behavior of structures have necessitated employing highly sophisticated numerical analysis procedures, such as the finite element method, to ensure safety and economy of such structures. The finite element method is one of these methods, which applies numerical techniques for solving problems of complicated boundaries and complex material characteristics.

This thesis aims to present a numerical analysis using finite element method to study the effect of semi-rigid connection on post-buckling behavior of steel frame. The effect of semi-rigid beam-column connection in terms of rotational joint stiffness with different values was studied and compared on different frame cases, such as:

1. Support type (fixed-fixed, fixed-pined and pined-pined).
2. Number of frame stories.
3. State of bracing (braced and unbraced).
4. Presence of lateral load.
5. Length of frame span.
6. Depth of the frame-beam.
7. Modulus of elasticity of frame elements.

1.5 Research Scope

The scope of study for this research includes:

1. Post-buckling behavior of steel frames.
2. Two-dimensional plane frame.
3. Semi-rigid beam to column connection.

1.6 Research Justification

In the past, there were a lot of studies that dealt with the post buckling behavior of steel structure but most of these studies mainly considered the beam-column connection as a fully rigid. This assumption leads to an incorrect estimation of structural behavior. In reality the connections are between the two extremes and have some rotational stiffness. This study investigates the post buckling behavior of steel frames and considers the beam-column connection as a semi-rigid connection with different rotational joint stiffness.

1.7 Methodology and Limitation of Thesis

The numerical analysis of this research includes modeling of three frames with rigid and semi-rigid connection. The numerical results were compared with previous experimental and numerical results that achieved by others researches in order to check the validity and accuracy of these models. The second part of this thesis included studying the effect of semi-rigid beam-column connection on different frame cases.

1.8 Layout of the Thesis

This Thesis consists of six chapters, as follows:

Chapter one: Includes an introduction about buckling and post-buckling behavior, semi-rigid connections in steel frames, the objective of the research and justification, methodology and layout of the thesis.

Chapter two: Describes the connections types and discusses the literature review.

Chapter three: Describes the finite element models that developed to simulate the behavior of post-buckling with rigid and semi–rigid connection.

Chapter four: Presents verification of the finite element models.

Chapter five: Describes in detail the case study of steel frame with the effect of semi-rigid connection as well as the study on it.

Chapter six: Presents the conclusions, recommendations, and suggestions for future studies.