

BEHAVIOR AND MODELING OF SEMI - RIGID STEEL BEAM TO COLUMN CONNECTIONS

By

Temesgen Wondimu

*A Thesis Submitted to the Graduate School of the Addis Ababa University in
Partial Fulfillment of the Requirements for the Degree of
Master of Science in Civil Engineering*

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LIST OF SYMBOLS

θ_r	relative deformation angle of the connection
ε_{ij}	coupling coefficient
ξ	damping ratio
β	interaction factor
φ	mode shape vector
Ω	natural frequency
$\bar{\theta}$	non dimensional connection rotation
θ_p	plastic rotation capacity of the connection
γ_i	participation factor for mode i
ψ_i	the i^{th} mode shape
$[u, w, \phi]$	nodal displacements in the x, y and z directions respectively
α	scaling factor for the purpose of numerical stability
\bar{m}	non-dimensional parameters used in the classification of connections
[D]	vector describing the excitation direction
[M]	mass matrix
A_i	mode coefficient for the i^{th} mode
C	correction matrix accounting for semi rigidity effects
C1, C2, C3...	standardization constants
d	width of the column
d	beam depth to which the connection is attached.
D_k	curve fitting parameter
E	Young's modulus
h	lever arm
$H[\theta]$	Heaviside's step function
I_b	moment of inertia of the beam
K'	total stiffness matrix for a semi-rigidly connected member
K_0	elastic element stiffness matrix of the original fully rigidly connected member:

K_{cwc}	stiffness of the panel zone (column web) under shear compression
K_{cws}	stiffness of the panel zone (column web) under shear
K_{eq}	resultant of the stiffness coming from column- web in tension, column right flange in bending, end plate in bending, angles, bolts in tension and reinforcement in case of composite structures
K_G	geometrical stiffness matrix
$K_{j,ini}$	resultant initial stiffness of all components
M	bending moment
M_p	full plastic moment capacity of the beam
n	shape parameter
N	total number of expanded modes
r	fixity factor
R_a	total modal response
R_k	instantaneous connection stiffness
R_{ki}	initial connection stiffness
R_{kp}	connection strain-hardening stiffness
T	transformation matrix

ABSTRACT

In most steel frame designs the beam to column connections are assumed to be rigid or pinned. Rigid joints, where no relative rotation occurs between the connected members, transfer not only substantial bending moments, but also shear and axial forces. On the other extreme, pinned joints are characterized by almost free rotation movement between the connected elements that prevent the transmission of bending moments. Despite these facts, the great majority of joints doesn't exhibit such idealized behavior.

A substantial effort has been made in recent years to characterize the behavior of semi rigid connections. Most design codes included methods and formulas to determine both their resistance and stiffness. EC3 and EC4, for instance, allow the use of springs attached to the end of the beams at both sides of the joints. In order to account for the panel shear deformation the code allows the use of interaction parameter, called the β factor. But since the definition of the β factor implies an approximation of internal forces at the joint, it requires an iterative process at the time of the global analysis of the structure.

In order to avoid this iterative process various researchers proposed new elements accounting for various deformation modes of the connections. Although the new proposed element by Bayo et.al accurately characterizes the behavior of semi-rigid connections based on the EC component method it is not appropriate to use in commonly available software.

In this thesis a new component-based two-node-connection element is proposed. By using the proposed two-node element the static and dynamic response behaviors of a semi-rigid frame as compared to a rigid frame of the same geometry and cross section are studied using a general purpose finite element package ANSYS.

The study showed that connection flexibility tends to increase vibration periods and hence reduces the internal stresses due to a given earthquake action in the frame elements as compared to those in rigid frames.

CHAPTER ONE

1.1. INTRODUCTION

Traditionally, steel frame design assumes that beam-to-column joints are rigid or pinned. Rigid joints, where no relative rotations occur between the connected members, transfer not only substantial bending moments, but also shear and axial forces. On the other extreme, pinned joints are characterized by almost free rotation movement between the connected elements that prevents the transmission of bending moments. Despite these facts, it is largely recognized that the great majority of joints doesn't exhibit such idealized behavior. This is explained by the fact that in semi – rigid frames the internal force distributions, lateral displacement magnitudes, collapse modes are functions of joint flexibility.

Extensive studies have been carried out over the past twenty five years to estimate the actual behavior of such joints. Innumerable studies have been produced on composite and steel semi-rigid connections, covering the state of the art [1], numerical studies and experimental tests [2 - 8].

The fundamental results of these investigations led to code specifications that provided structural engineers with adequate procedures to evaluate the moment rotation characteristics of semi rigid connections. A good example of this new design trend is available in Eurocode3 [17]. Despite the substantial increase in structural design knowledge, the semi-rigid connection design is still facing resistance from structural engineers. This is explained by:

1. Lack of detailed information on the advantages of semi-rigid design philosophy.

Semi-rigid frame design has many advantages. These include:

- a. *Economy*

Investigations showed that for a four story semi-rigid floor system up to 15% economy in terms of steel weight was reached even when compared to the traditional most

economical solution. As these structures are used in large scale construction the achieved 15% weight reduction can generate an even greater economy [10].

b. Ductility

The components of partial strength joints can deform in a ductile manner in the case of strong earthquake. In the common case where beam sizes are governed by drift or alike design criteria, rather than flexural strength requirements, ductile partial strength connections allow the formation of a desirable beam hinging global frame mechanism. The possibility of using partial strength connections as the main energy dissipative mechanism for the seismic resistance of frames has now been recognized in modern design codes [12, 13, and 19].

2. Complexity and lack of effective tools for global analysis of semi-rigid frames

The non linear characteristics of a steel beam-to-column connection play a very important role for frame global analysis. Although modern structural steel and composite codes [Eurocode3 &4] include procedures and formulas to define both the stiffness and resistance of semi rigid connections there is no explicit method to model and analyze frames with these connections.

3. Lack of sufficient experimental data

The design and construction of steel, as compared to others, is not yet at its outset in our country which leads to lack of experience. Besides the behavior of semi rigid connections is very dependent on experimental investigations which is not possible in our country due to the cost associated. Hence structural engineers usually face the problem of experimentally validated connection data bases which, in consequence, hinders the planning of structural steel frames of semi rigid nature.

1.2. SCOPE AND OBJECTIVE OF THE THESIS

The main objective and scope of the thesis include:

1. To discuss the moment rotation behavior of commonly used beam to column semi rigid connections

Moment rotation behavior of semi-rigid connections is central to the analysis and design of the joints as well as the frames. In recent years several researchers have published papers on moment-rotation relationship of various semi rigid connections for use in the analysis and design of semi rigid frames [2, 7].

Because of the complexity of the failure modes and large number of variables involved associated with connections, experimental evidence is unlikely ever to be able to thoroughly examine all aspects of the problem. Beside the cost involved is high for countries like ours. An alternative numerical approach is finite element method. In this thesis the moment-rotation relationship is discussed in detail to enhance the understanding of semi-rigid joint behavior.

2. To model and analyze semi-rigid frames

Traditionally connections can be modeled either pinned or fixed in structural engineering software. But for semi rigid frames connection modeling is not that easy. The most common method is to use springs attached at the end of beams at both sides of the joint, the springs being characterized by various models. A new and simplified modeling approach based on EC component method is proposed. In addition the static and dynamic behavior of semi-rigid frames as compared to rigid ones is discussed.

CHAPTER TWO

MOMENT–ROTATION RELATIONSHIP OF SEMI-RIGID CONNECTIONS

2.1. CONNECTION CLASSIFICATION

Much research and development work has focused primarily on the analysis and design of frames based on idealization of the joints as either fully rigid or pinned. In reality, the actual behavior of the structure is as much dependent on the connection and joint characteristics as on the individual component elements making up the structure. Ample research evidence exists which establishes that the observed joint behavior is substantially different from the assumed idealized models. Depending on the stiffness, strength, and deformation capacity, connections in structural frame work can influence the behavior of the structure in several ways. Under static loads, connection deformations contribute to the vertical deflections of beams and lateral drift of the frame. The moment resistance of the connections will influence the internal force distribution and local and global stability of the frame.

Realizing the potential influence of connections on frame performance, various design codes [17, 18, 22 and 23] has introduced provisions to allow designers to consider explicitly the behavior of connections in the design of structural steel frames.

Most design codes, list three types of connections for designing multi - story frames.

1. *Type I or “Rigid framing”*. This construction assumes that the beam to column connections have sufficient rigidity to maintain the original geometric angle between interesting members. Rigid connections are assumed for elastic analysis. Type I connections are sometimes referred to as moment connections.
2. *Type II or “simple framing”*. This construction assumes that, when the structure is loaded with gravity loads, the beam and girder connections transfer only vertical shear reactions without bending moment. The connections are allowed to rotate freely without any restraint. This type of connection is also called shear connection.

3. *Type III or “semi rigid framing”*. This construction assumes that the connections can transfer vertical shear and also have adequate stiffness and capacity to transfer some moment.

The AISC-LRFD specifications (1986) designate two types of constructions in their provisions: Type FR (fully restrained) and Type PR (partially restrained). Type FR corresponds to EBCS-3 Type I and Type PR includes EBCS-3 Type 2 and 3.

To be pertinent, the rigidity of the connection should be defined with respect to the rigidity of the connecting member. For general application to a wide range of beam-to-column connection, Bjorhorde et al. [2] introduced a non-dimensional system of classification that compares the connection stiffness to the beam stiffness. In defining the beam stiffness, a reference beam length $5d$ is used, where d is the beam depth to which the connection is attached.

The non-dimensional parameters used in the classification of connections are:

$$\bar{m} = \frac{M}{M_p}, \quad \bar{\theta} = \frac{\theta_r}{\theta_p} \quad (2.1)$$

where:

θ_r is the relative deformation angle of the connection,

$$\theta_p = \frac{M_p}{(EI_b/5d)},$$

I_b - moment of inertia of the beam and

M_p - full plastic moment capacity of the beam.

The classification is based on the strength and stiffness of the connections with the boundary regions shown in Fig 2.1. The different regions in Fig 2.1 are defined as:

1. Rigid connection

$$\bar{m} \geq \begin{cases} 0.7 & \text{in terms of strength} \\ 2.5\bar{\theta} & \text{in terms of stiffness} \end{cases} \quad (2.2)$$

2. semi- rigid connection

$$\begin{cases} 0.7 > \bar{m} > 0.2 & \text{in terms of strength} \\ 2.5\bar{\theta} > \bar{m} > 0.5\bar{\theta} & \text{in terms of stiffness} \end{cases} \quad (2.3)$$

3. Flexible connection

$$\bar{m} \leq \begin{cases} 0.2 & \text{in terms of strength} \\ 0.5\bar{\theta} & \text{in terms of stiffness} \end{cases} \quad (2.4)$$

Bjorhorde et al. [2] also have proposed an expression for calculating the required rotation capacity of the connection based on a reference beam length and by curve fitting with test data. This simplified expression is written as:

$$\bar{m} = \left(\frac{5.4 - 2\bar{\theta}}{3} \right) \quad (2.5)$$

According to this equation, the required rotation capacity of the beam-to-column connection depends on the ratio of the ultimate moment capacity of the connection to the fully plastic moment of the beam, and it is inversely proportional to the initial connection stiffness. In other words the smaller the initial connection stiffness, the larger the necessary rotation capacity.

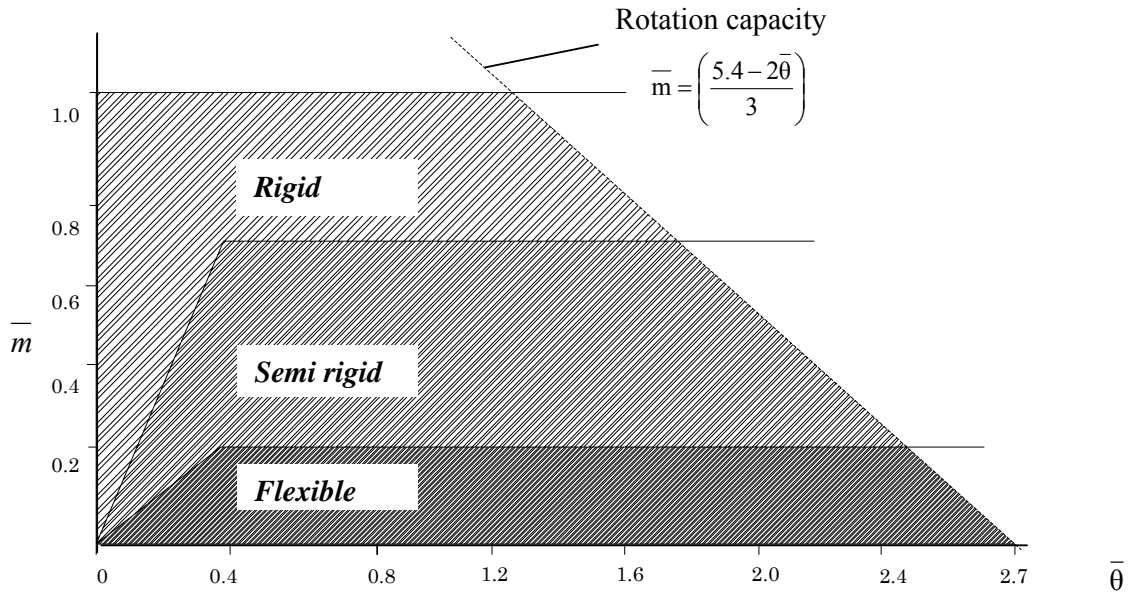


Fig. 2.1 Classification of connections according to Bjorhorde et al. (1990)

Although relatively better investigations were made on bare steel connections most standards give little guidance on the property and design of composite connections. Composite connections can be defined as a connection between a composite member and any other member in which reinforcement is intended to contribute to the resistance of the connection. This chapter is devoted to previous researches undertaken in order to provide a complete set of behavior, analysis and design rules of semi-rigid connections.

2.2. MODELING OF SEMI RIGID CONNECTIONS

2.2.1 Based on Available Experimental Test Data

The most commonly used approach to describe $M-\theta_r$ curve is to curve fit the experimental data with simple expressions. Several analytical models have been developed to represent connection flexibility using available experimental test data.

Early models used the initial connection stiffness as the key parameter in a linear $M-\theta_r$ model [1, 2]. Although the linear model is very easy to use, it has a serious disadvantage. It is suitable for only a small range of the initial relative rotation. A closer approximation of true connection behavior can be obtained by using either a bilinear model or a piecewise linear model. In these models, the abrupt changes in connection stiffness at the transition points make their practical use difficult. Jones et al. [1] proposed a cubic -B- spline model to obtain a more suitable function. However, this model requires a large number of sampling data during the formulation process. Frye and Morris [2] have reported a polynomial model to evaluate the behavior of the several types of connections. In this model, the $M-\theta_r$ behavior is represented by an odd power polynomial.

Some of the importance of connection models will be discussed in more detail in the following section.

a. Frye-Morris polynomial Model

The most popular model for structural analysis is the polynomial function proposed by Frye and Morris (1975). The Frye-Morris model was developed based on a procedure formulated

by Sommer (1969). They used the method of least square to determine the constants of the polynomial. This model has the general form shown in Eq. (2.9).

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \quad (2.9)$$

where K is the standardization parameter dependent upon the connection type and geometry, and C_1 , C_2 , and C_3 are curve fitting constants.

This model represents the $M-\theta_r$ behavior reasonably well. The main drawback is that the nature of the polynomial is to peak and trough within a certain range. Then, the first derivative of this function, which indicated the tangent and connection stiffness, may become negative at some value of connection moment M. This is physically unacceptable. This negative stiffness makes structural analysis difficult if the analysis scheme with tangent connection stiffness is used. Relative moment-rotation curves of extensively used semi-rigid connections are shown in Fig 2.2

The curve fitting constants C_1 , C_2 , and C_3 and the standardization constant K for each connection type are summarized in Table 2.1

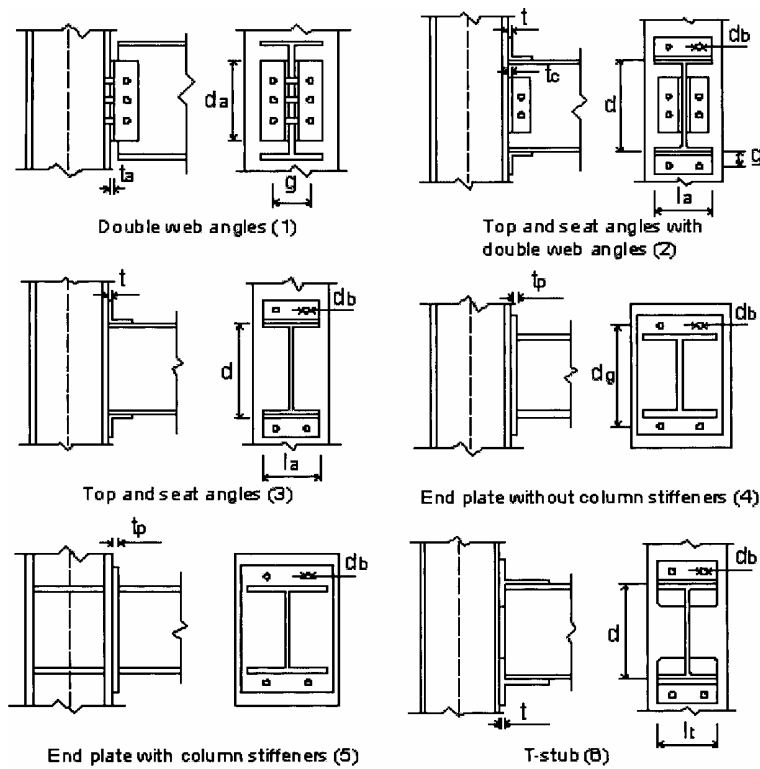


Fig. 2.2 Semi rigid connection types and size parameters

Table 2.1: Curve fitting Constants and Standardization constants for the Frye-Morris Polynomial Model

Connection types	Curve-fitting constants	Standardization constants
Single web-angle connection	$C_1 = 4.28 \times 10^{-3}$ $C_2 = 1.45 \times 10^{-9}$ $C_3 = 1.51 \times 10^{-16}$	$K = d_a^{-2.4} t_a^{-1.81} g^{0.15}$
Double web-angle connection	$C_1 = 3.66 \times 10^{-4}$ $C_2 = 1.15 \times 10^{-6}$ $C_3 = 4.57 \times 10^{-8}$	$K = d_a^{-2.4} t_a^{-1.81} g^{0.15}$
Top- and seat-angle with double web-angle connection	$C_1 = 2.23 \times 10^{-5}$ $C_2 = 1.85 \times 10^{-8}$ $C_3 = 3.19 \times 10^{-12}$	$K = d^{-1.287} t^{-1.128} t_c^{-0.415} l_a^{-0.694} g^{1.350}$
Top- and seat-angle without double web-angle connection	$C_1 = 8.46 \times 10^{-4}$ $C_2 = 1.01 \times 10^{-4}$ $C_3 = 1.24 \times 10^{-8}$	$K = d^{-1.5} t^{0.5} l_a^{-0.7} d_b^{-1.1}$
End-plate connection without column stiffeners	$C_1 = 1.83 \times 10^{-3}$ $C_2 = -1.04 \times 10^{-4}$ $C_3 = 6.38 \times 10^{-6}$	$K = d_g^{-2.4} t_p^{-0.4} t_r^{-1.5}$
End-plate connection with column stiffeners	$C_1 = 1.79 \times 10^{-3}$ $C_2 = -1.76 \times 10^{-4}$ $C_3 = 2.04 \times 10^{-4}$	$K = d_g^{-2.4} t_p^{-0.6}$
T-stub connection	$C_1 = 2.1 \times 10^{-4}$ $C_2 = 6.2 \times 10^{-6}$ $C_3 = -7.6 \times 10^{-9}$	$K = d^{-1.5} t^{-0.5} l_t^{-0.7} d_b^{-1.1}$
Header-plate connection	$C_1 = 5.10 \times 10^{-5}$ $C_2 = 6.20 \times 10^{-10}$ $C_3 = 2.40 \times 10^{-13}$	$K = t_p^{-1.6} g^{1.6} d_p^{-2.3} t_w^{-0.5}$

^a All size parameters are in inches.

b. Modified exponential Model

The Chen-Lui exponential Model has been refined to accommodate linear components by Kishi and Chen [2] and is referred to herein as the modified exponential model. This model is represented by a function of the following form

$$M = M_o + \sum_{j=1}^m C_j \left\{ 1 - \exp\left(-\frac{|\theta_r|}{2j\alpha}\right) \right\} + \sum_{k=1}^n D_k (|\theta_r| - |\theta_k|) H[|\theta_r| - |\theta_k|] \quad (2.10)$$

where:

M_o the initial connection moment

α scaling factor for the purpose of numerical stability

C_j, D_k curve fitting parameters,

θ_k the starting rotation of the k^{th} linear component given from the experimental $M-\theta_r$ curve

$H[\theta]$ Heaviside's step function (unity for $\theta \geq 0$, zero for $\theta < 0$).

Using the linear interpolation technique for the original $M-\theta_r$ data, the weight function for each $M-\theta_r$ datum is nearly equal. The constants C_j and D_k for the exponential and linear terms of the function are determined by a linear regression analysis.

The instantaneous connection stiffness R_k at an arbitrary relative rotation $|\theta_r|$ can be evaluated by differentiating Eq.(2.10) with respect to $|\theta_r|$.

When the connection is loaded, we have

$$R_k = R_{kt} = \left. \frac{dM}{d|\theta_r|} \right|_{|\theta_r|=|\theta_r|} = \sum_m \frac{C_j}{2j\alpha} \exp\left(-\frac{|\theta_r|}{2j\alpha}\right) + \sum_n D_k H[|\theta_r| - |\theta_k|] \quad (2.11)$$

When the connection is unloaded, we have

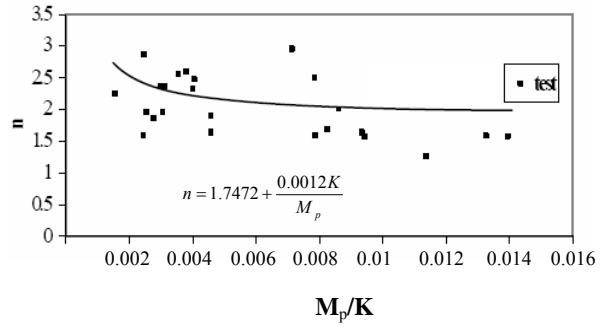
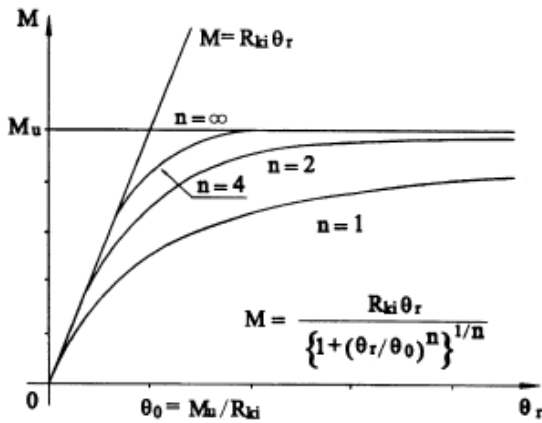
$$R_k = R_{ki} = \left. \frac{dM}{d|\theta_r|} \right|_{|\theta_r|=0} + \sum_m \frac{C_j}{2j\alpha} + D_k H[|\theta_k|]_{k=1} \quad (2.12)$$

This model has the following merits:

- the formulation is relatively simple
- it can deal with connection loading and unloading for the full range of relative rotation in a second order structural analysis with secant connection stiffness
- the abrupt changing of the connection stiffness among the sampling data is only general from inherent experimental characteristics.

c. Three Parameter Power Model

The modified exponential model mentioned above is a curve fitting equation obtained by using the least-mean-square technique for the experimental test data. From a different view point, Chen and Kishi and Kishi et al. [2] developed another procedure to predict the moment-rotation characteristics of steel beam-to-column connections. In this procedure, the initial connection stiffness and ultimate moment capacity of the connection are determined by a simple analytical model. Using those values so obtained, a three parameter power model given by Richard and Abbott (1975) was adopted to represent the moment-rotation relationship of the connection.



(a) Generalized moment rotation curve

(b) Shape factor

Fig. 2.3. Three parameter power model

The generalized form of this model is

$$M = \frac{R_{ki} \theta_r}{\left[1 + \left(\frac{\theta_r}{\theta_0} \right)^n \right]^{1/n}} \tag{2.13}$$

- where R_{ki} initial connection stiffness
- M_u ultimate moment capacity
- θ_0 a reference plastic rotation= M_u/R_{ki}
- n shape parameter.

Eq. 2.13 has the shape shown in Fig. 2.3. From this figure, it is recognized that the larger the power index n , the steeper the curve. The shape parameter n , can be determined by using the method of least squares for the differences between the predicted moments and the experimental test data

d. Four Parameter Power Model

A four parameter equation, first proposed by Richard and Abbott [15], has proven to be quite versatile for representing a wide range of connection responses. This equation may be expressed in the form

$$M = \frac{R_l \theta_r}{\left[1 + \left(\frac{\theta_r}{\theta_0}\right)^n\right]^{1/n}} + R_{kp} \theta_r \quad (2.14)$$

where:

M connection moment

R_{kp} connection strain-hardening stiffness

M_u connection ultimate moment capacity

θ_r connection rotation

$R_l = R_{ki} - R_{kp}$

θ_0 reference rotation = M_u / R_{ki}

R_{ki} initial connection stiffness

n shape parameter

One of the salient merits of this model is that the associated tangent stiffness of a semi-rigid connection can be readily obtained through differentiation of Eq. 2.14 with respect to θ_r :

$$R_t = \frac{R_l}{\left[1 + \left(\frac{\theta_r}{\theta_0}\right)^n\right]^{(n+1)/n}} + R_{kp} \quad (2.15)$$

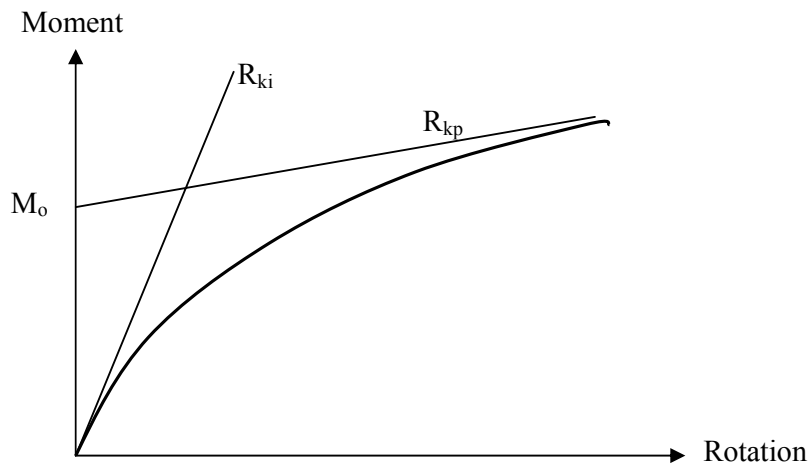


Fig.2.4 Moment rotation relationship for a four-parameter model of semi rigid connections

If the initial tangent stiffness, R_{ki} , is specified as a single independent parameter, the other three parameters needed in Eq. (2.14) are determined by

$$R_{kp} = 0.0282R_{ki}$$

$n =$ shape factor determined from Fig.2.3

$$M_u = 0.00338R_{ki}$$

which were provided by Hsieh and Deierlein [15] through a condensation process of collected experimental data in order to eliminate scale variations.

2.2.2 Finite Element Modeling of Semi-Rigid Connections

The finite element representation of semi rigidly connected structural members is proposed by Xu [15]. The member is assumed to be consisting of the originally fully rigidly connected member and zero length spring elements (with spring constant equal to the appropriate secant stiffness of the semi-rigid connection it simulates) attached to each member end.

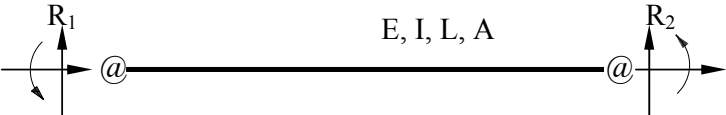


Fig. 2.5 Finite Element modeling semi-rigid connections

Consider the beam member shown below with the end springs. θ_{ra} and θ_{rb} represent the relative spring rotations of both ends and k_a and k_b are the corresponding spring stiffness expressed as:

$$k_a = Ma / \theta_{ra} ; k_b = Mb / \theta_{rb} \tag{2.18a}$$

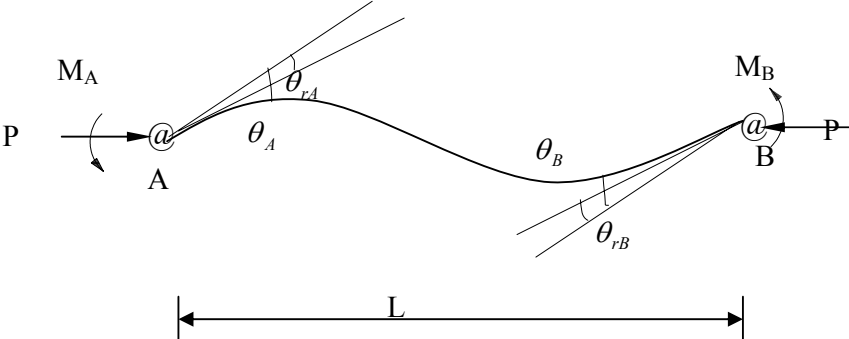


Fig. 2.6 Beam member with rotational springs

The relationship between end-moments and end-rotations of a beam can be written by replacing the end rotations θ_A and θ_B by $(\theta_A - \theta_{rA})$ and $(\theta_B - \theta_{rB})$ respectively, as follows [4]

$$M_A = \frac{EI}{L} \left[4 \left(\theta_A - \frac{M_A}{k_A} \right) + 2 \left(\theta_B - \frac{M_B}{k_B} \right) \right] \quad (2.18b)$$

$$M_B = \frac{EI}{L} \left[4 \left(\theta_B - \frac{M_B}{k_B} \right) + 2 \left(\theta_A - \frac{M_A}{k_A} \right) \right] \quad (2.18c)$$

The elastic element stiffness matrix of a semi-rigidly connected member, denoted by K' , can then be formulated as [6]

$$K' = K_0 C \quad (2.19)$$

where K_0 is the elastic element stiffness matrix of the original fully rigidly connected member:

$$\begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & -\frac{12EI}{L^3} & \frac{6EI}{L^2} \\ & & \frac{4EI}{L} & 0 & -\frac{6EI}{L^2} & \frac{2EI}{L} \\ & & & \frac{EA}{L} & 0 & 0 \\ & \text{symm} & & & \frac{12EI}{L^3} & -\frac{6EI}{L^2} \\ & & & & & \frac{4EI}{L} \end{bmatrix} \quad (2.20)$$

with E , L , I , A as elasticity modulus, length, cross-sectional moment of inertia, and cross sectional area of a given member, and C , a correction matrix accounting for semi rigidity effects:

$$C = \begin{bmatrix} 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{4r_2 - 2r_1 + r_1r_2}{4 - r_1r_2} & -\frac{2Lr_1(1-r_2)}{4 - r_1r_2} & 0 & 0 & 0 & 0 \\ 0 & \frac{6(r_1 - r_2)}{L(4 - r_1r_2)} & \frac{3r_1(2 - r_2)}{4 - r_1r_2} & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{4r_1 - 2r_2 + r_1r_2}{4 - r_1r_2} & \frac{2Lr_2(1-r_1)}{4 - r_1r_2} & \\ 0 & 0 & 0 & 0 & \frac{6(r_1 - r_2)}{L(4 - r_1r_2)} & \frac{3r_2(2 - r_1)}{4 - r_1r_2} \end{bmatrix} \quad (2.21)$$

where r_1 and r_2 are the “fixity factors” as defined by [15]:

$$r_j = \frac{1}{1 + \frac{3EI}{R_j L}}, \quad j = 1, 2 \quad (2.22)$$

where R_j is the appropriate secant stiffness, under current load level, of the semi-rigid connection at j^{th} end.

Physically speaking, the fixity factor indicates how stiff a semi-rigid connection is relative to the structural member to which it attaches. Two extreme cases are $r=1$ for fully rigid connections and $r=0$ for perfectly pinned connections. Obviously a realistic semi-rigid connection will have a value of $0 < r < 1$.

Note that the end moments need also to be modified due to the presence of semi rigid connections. For example, a structural member with semi-rigid connections at both ends and subjected to uniformly distributed transverse load of w has the following end moments [15]:

$$\begin{aligned}
M_1 &= \left(\frac{wL^2}{12} \right) \left[3r_1 \left(\frac{2-r_2}{4-r_1r_2} \right) \right] \\
M_2 &= \left(\frac{wL^2}{12} \right) \left[3r_2 \left(\frac{2-r_1}{4-r_1r_2} \right) \right]
\end{aligned} \tag{2.23}$$

Geometrical Non-Linearity Consideration

A geometrical stiffness matrix derived by Xu [15], which is specifically suited for semi rigidly connected members, takes the following form:

$$\begin{bmatrix}
0 & 0 & 0 & 0 & 0 & 0 \\
& G_{22} & G_{23} & 0 & G_{25} & G_{26} \\
& & G_{33} & 0 & G_{35} & G_{36} \\
& & & 0 & 0 & 0 \\
& \text{symm.} & & & G_{55} & G_{56} \\
& & & & & G_{66}
\end{bmatrix} \tag{2.24}$$

where:

$$\left. \begin{aligned}
G_{22} &= \frac{2N}{5L(4-r_1r_2)^2} (3r_1^2r_2^2 + r_1^2r_2 + r_1r_2^2 + 8r_1^2 + 8r_2^2 - 34r_1r_2 + 40) \\
G_{23} &= \frac{N}{10(4-r_1r_2)^2} (r_1^2r_2^2 - 12r_1^2r_2 + 16r_1r_2^2 + 32r_1^2 - 28r_1r_2) \\
G_{33} &= \frac{2NL}{5(4-r_1r_2)^2} (2r_1^2r_2^2 - 7r_1^2r_2 + 8r_1^2) \\
G_{36} &= \frac{-NL}{10(4-r_1r_2)^2} (7r_1^2r_2^2 - 16r_1^2r_2 - 16r_1r_2^2 + 28r_1r_2) \\
G_{26} &= \frac{N}{10(4-r_1r_2)^2} (r_1^2r_2^2 + 16r_1^2r_2 - 12r_1r_2^2 + 32r_2^2 - 28r_1r_2) \\
G_{66} &= \frac{2NL}{5(4-r_1r_2)^2} (2r_1^2r_2^2 - 7r_1^2r_2 + 8r_2^2) \\
G_{55} &= -G_{25} = G_{22}, \quad G_{35} = -G_{23}, \quad G_{56} = -G_{26}
\end{aligned} \right\} \tag{2.25}$$

Note that the geometrical stiffness matrix, K_G , for a fully rigidly connected member can be easily obtained by letting $r_1=r_2=1$;

$$K_G = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ & \frac{6N}{5L} & \frac{N}{10} & 0 & -\frac{6N}{5L} & \frac{N}{10} \\ & & \frac{2NL}{15} & 0 & -\frac{N}{10} & -\frac{NL}{30} \\ & & & 0 & 0 & 0 \\ & & & & \frac{6N}{5L} & -\frac{N}{10} \\ & & & & & \frac{2NL}{15} \end{bmatrix} \quad (2.26)$$

The total stiffness matrix for a semi-rigidly connected member considering geometrical non-linearity is then:

$$K' = K_0 C + KG \quad (2.27)$$

The main problem associated with this modeling approach is unless an analysis program is developed or the stiffness matrices accounting semi - rigid behavior is incorporated in the software used for structural analysis it is not easy to use. Besides it will not account for the panel shear deformation.

2.2.3. An Effective Component Based Method to Model Semi-Rigid Connections

The various modeling approaches discussed accurately represent the characteristics of connections at both sides of the joint, but don't take into account the panel deformations due to shear and compressive forces [3].

Early experimental studies showed the importance of panel shear deformations for stable energy dissipation under cyclic loading. Modeling of the panel is very important for the avoidance of local failure of columns under ultimate limit state.

EC3 and EC4 [17, 18] take in to account the deformation of the panel zone as one of the components of the joint. Further more, EC3 provides a sophisticated and elaborated tool based on the so called *component method* that allows one to obtain the strength and stiffness characteristics of all the different components of the joint, including the panel zone, shown in Fig. 2.7.

The component method has been preferred over the finite element method due to the ability of the former to better model local phenomena [3]. Under EC3 and EC4 the different components (springs) are assembled to form a resultant elastic-plastic spring element that models the connections at each side of the joint to be attached at the end of each beam.

a. Component joint modeling

EC3 proposes a mechanical model for the semi- rigid joint in which each component is modeled by an equivalent linear spring. Fig 2.7 shows a condensed model of the right side connections in which K_{eq2} is the resultant of the stiffness coming from the following components:

column- web in tension, column right flange in bending, end plate in bending, angles, bolts in tension and reinforcement in case of composite structures. The springs K_{cws} and K_{cwc} represent the stiffness of the panel zone (column web) under shear and compression respectively.

These springs are assembled to form a single bi-linear (elastic-plastic) rotational spring that models the connection and is attached at the intersection between beam and column for global analysis. The bilinear model is characterized by an initial stiffness $K_{j,ini}$ (resultant of all the components) and moment resistance $M_{j,Rd}$ which are defined by the following expressions:

$$K_{j,ini} = \frac{h^2}{\frac{1}{K_{cws}} + \frac{1}{K_{cwc}} + \frac{1}{K_{eq}}} \quad (2.28)$$

$$M_{j,Rd} = \sum F_{Lt,i,Rd} h_i \quad (2.29)$$

where h is the lever arm and $F_{Lt,i,Rd}$ is the resistance of each component[17].

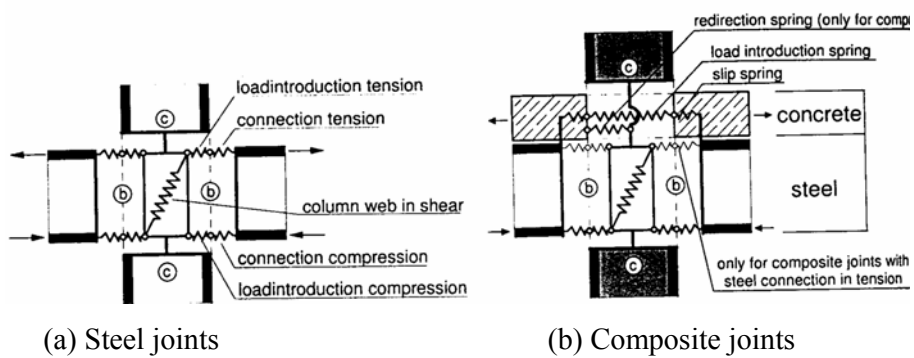


Fig 2.7 General models for steel and composite joints

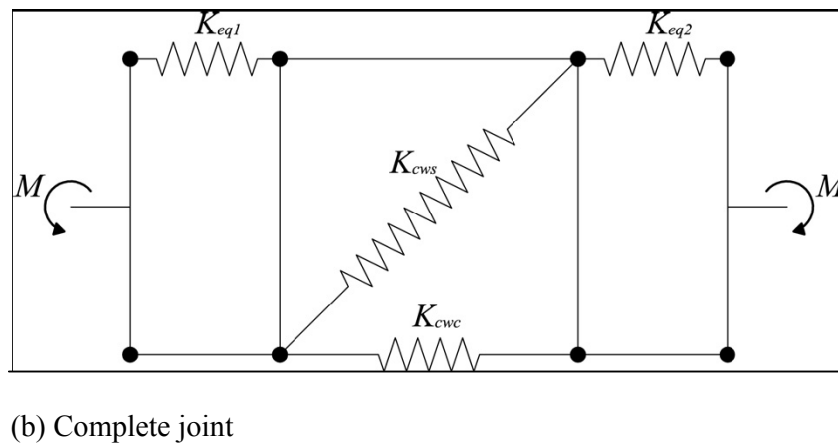
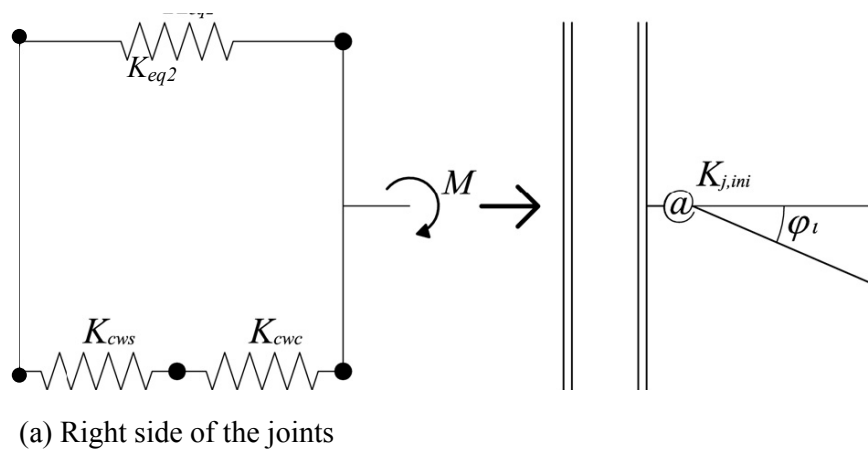


Fig. 2.8. Spring model of the right side and complete semi-rigid joint

- Stresses in the column panel zone: the β factor

The shear force acting on the panel zone is defined as [3]:

$$V_{cw} = \frac{M_1}{h} \left[\left(1 + \frac{M_2}{M_1} \right) - \frac{h}{2M_1} (V_{c1} + V_{c2}) \right] = \frac{M_1}{h} \beta_1 \quad (2.30a)$$

$$V_{cw} = \frac{M_2}{h} \left[\left(1 + \frac{M_1}{M_2} \right) - \frac{h}{2M_2} (V_{c1} + V_{c2}) \right] = \frac{M_2}{h} \beta_2 \quad (2.30b)$$

where M_1 and M_2 are the bending moments acting at the left and right side of the joint, and V_{c1} and V_{c2} , are the shear forces in the upper and lower columns, respectively.

The expressions in brackets of Eqns (2.30a) and (b) constitute the interaction factors β that are used in EC3 to define the shear forces acting on the panel zone.

EC3 further simplifies Eq. 2.30 by omitting the beneficial effect of the shear forces V_{c1} and V_{c2} [17], and by limiting β to positive values less than 2, as in the following equation:

$$\beta_1 = \left| 1 + \frac{M_2}{M_1} \right| \leq 2 \quad \text{and} \quad \beta_2 = \left| 1 + \frac{M_1}{M_2} \right| \leq 2 \quad (2.31)$$

These β factors affect the values of the joint stiffness $K_{j,ini}$ and moment resistance $M_{j,Rd}$. However, the bending moments M_1 and M_2 and shear forces V_{c1} and V_{c2} coming from different elements (beams and columns) are not known in advance, and are only known after a global analysis. Therefore, an initial guess of these values and a subsequent iteration procedure are needed to update the joint characteristics and obtain the precise final internal forces and moments.

b. Using Finite Dimensioned Four Node Elastic-Plastic Joint Element

E. Bayo et.al, 2006 [3] proposed an EC3 and EC4 component based finite dimensioned elastic-plastic 4-node joint element which takes into account the actual size of the joint, its deformation characteristics (components) including those of the panel zone, local phenomenon and all the internal forces that concur at the joint. As a consequence, this new element avoids the use of the β -factor and the inherent initial guessing and iterative process

that it requires. Since the element is attached to the adjacent beams and columns the eccentricities of the internal forces coming from them are also taken into account.

- **Interior joint**

The model of an interior joint starts from the scheme of the components shown in Fig. 2.8. The springs K_{eq1} and K_{eq2} model the left and right connection stiffness respectively. The springs K_{cws} and K_{cwc} represent the stiffness of the panel zone (column web) under shear and compression respectively, which are usually the most critical components of the joint. All these values may be obtained by means of formulas described in EC3 and EC4. Fig. 2.9 shows the new base element with 4 nodes and 9 degrees of freedom, proposed by Bayo. et.al, in which the rigid body modes have been restrained.

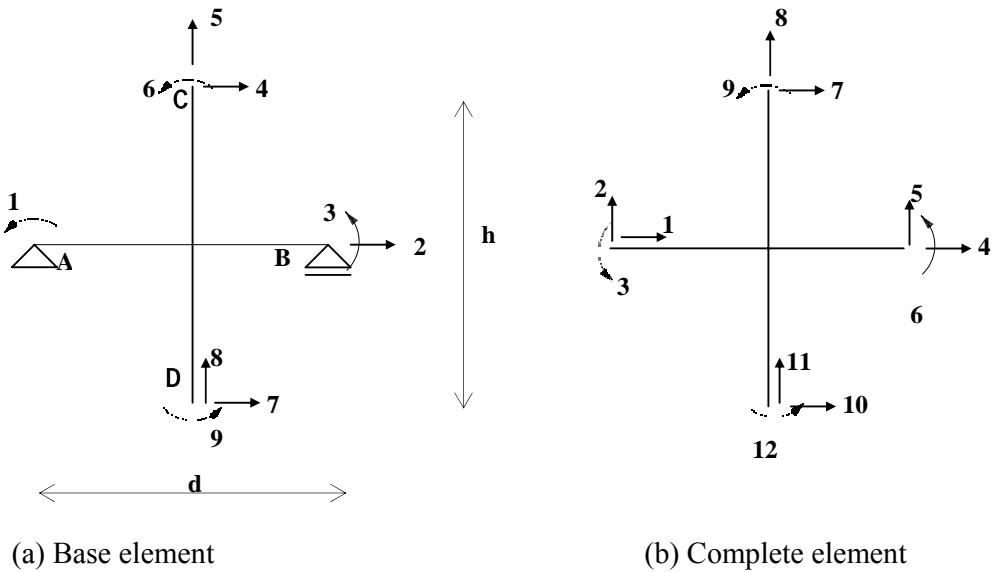


Fig. 2.9 Degrees of freedom of the interior joint

The dimension d is equal to the width of the column, and the height h corresponds to the lever arm defined in EC3, Part 1.8 [17].

This base element will be used to obtain the basic flexibility and stiffness matrices. Afterwards, the contribution of the rigid body modes will be added to obtain the complete stiffness matrix in terms of the 12 degrees of freedom shown in Fig. 2.9b.

It should be noticed that all the forces and moments coming from the adjacent beams and columns concur at the joint at the points A, B, C and D. Therefore the complete force field in the panel zone is known with no need of the β factor. Also, since the real dimensions of the joints are being considered the eccentric moments are automatically taken into account.

In order to develop the flexibility matrix the following deformation modes are considered (some of these models are depicted in Fig 2.10):

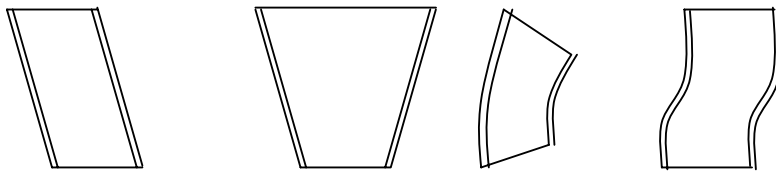


Fig. 2.10. Deformation modes corresponding to shear, compression and bending of the panel zone.

- Shear deformation of the panel zone
- Tension and compression of the panel zone
- Column flange and end-plate under bending
- Elasticity and configuration of the bolts
- Bending deformation of the column
- Axial deformation of the column

Due to the stiffening effect of the column flanges the segment AB (see Fig. 2.9) is supposed to be rigid under the shear forces coming from the beams. Thus, considering the 9 degrees of freedom of element of Fig 2.9 (left) and the deformation modes, the resulting non-zero elements of the upper triangular part of the flexibility matrix are derived and shown in Eq. 2.32.

The bilinear behavior of the springs is defined according to the values given by EC3.

Once the flexibility matrix has been obtained the basic stiffness matrix may be calculated through matrix inversion:

$$\mathbf{K}_b = \mathbf{F}^{-1}$$

Following standard procedures in matrix structural analysis [25] the rigid body modes may be added by means of a transformation matrix \mathbf{T} , which is shown in Eq 2.33.

$$\begin{aligned}
F_{11} &= \frac{1}{h^2} \left(\frac{1}{K_{cws}} + \frac{1}{K_{cwc}} + \frac{1}{K_{eq1}} \right) & F_{13} &= \frac{1}{h^2} \left(\frac{1}{K_{cws}} \right) \\
F_{14} &= -\frac{1}{2h} \left(\frac{1}{K_{cws}} \right) & F_{17} &= \frac{1}{2h} \left(\frac{1}{K_{cws}} \right) \\
F_{22} &= \frac{1}{K_{eq1}} + \frac{1}{K_{eq2}} & F_{24} &= \frac{1}{2K_{eq1}} + \frac{1}{2K_{eq2}} \\
F_{27} &= \frac{1}{2K_{eq1}} + \frac{1}{2K_{eq2}} & F_{33} &= \frac{1}{h^2} \left(\frac{1}{K_{cws}} + \frac{1}{K_{cwc}} + \frac{1}{K_{eq2}} \right) \\
F_{34} &= -\frac{1}{2h} \left(\frac{1}{K_{cws}} \right) & F_{37} &= \frac{1}{2h} \left(\frac{1}{K_{cws}} \right) \\
F_{44} &= \frac{1}{4K_{cws}} + \frac{1}{2K_{eq1}} + \frac{1}{2K_{eq2}} + \frac{h^3}{24EI_c} & F_{46} &= -\frac{h^2}{8EI_c} \\
F_{47} &= -\frac{1}{4K_{cws}} + \frac{1}{2K_{eq1}} + \frac{1}{2K_{eq2}} + \frac{h^3}{24EI_c} & F_{55} &= \frac{h}{2EA_c} \\
F_{66} &= \frac{h}{2EI_c} & F_{77} &= \frac{1}{4K_{cws}} + \frac{1}{2K_{eq1}} + \frac{1}{2K_{eq2}} + \cdot \\
F_{88} &= \frac{h}{2EA_c} & F_{99} &= \frac{h}{2EI_c}
\end{aligned} \tag{2.32}$$

where A_c and I_c represent the area and moment of inertia of the column respectively.

$$T = \begin{bmatrix} 0 & -1 & 0 & -1 & 0 & 0 & -1 & 0 & 0 \\ \frac{1}{d} & 0 & -\frac{1}{d} & -\frac{h}{2d} & -\frac{1}{2} & \frac{1}{d} & \frac{h}{2d} & -\frac{1}{2} & \frac{1}{d} \\ 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ -\frac{1}{d} & 0 & \frac{1}{d} & \frac{h}{2d} & -\frac{1}{2} & -\frac{1}{d} & -\frac{h}{2d} & \frac{1}{2} & -\frac{1}{d} \\ 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (2.33)$$

Thus, the basic stiffness \mathbf{K}_b may be transformed to obtain the general stiffness matrix in terms of the 12 degrees of freedom (Fig 2.10 right) as follows:

$$\mathbf{K} = \mathbf{TK}_b\mathbf{T}^T \quad (2.34)$$

- *External joints*

The external and corner joints will only have 9 and 6 degrees of freedom, respectively, as shown in Fig. 2.11

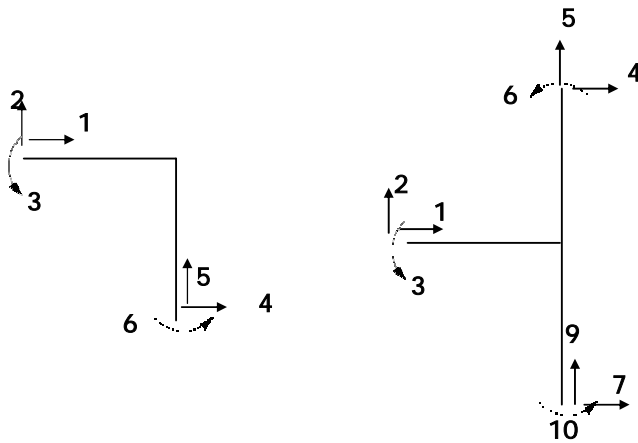


Fig. 2.11 Degrees of freedom of the corner and exterior elements.

Their stiffness matrices may be obtained from the 12 degrees of freedom stiffness matrix, \mathbf{K} , by means of the static condensation algorithm that helps eliminating those degrees of freedom that are not necessary. The general 12 degrees of freedom stiffness matrix may be partitioned as follows:

$$\mathbf{K} = \begin{bmatrix} \mathbf{K}_{dd} & \mathbf{K}_{de} \\ \mathbf{K}_{ed} & \mathbf{K}_{ee} \end{bmatrix} \quad (2.35)$$

where the sub-index d stands for desired, and sub-index e for eliminated degree of freedom, respectively. Then the condensed stiffness matrix may be written as [3]:

$$\mathbf{K}_{dd}^* = \mathbf{K}_{dd} - \mathbf{K}_{de} \mathbf{K}_{ee}^{-1} \mathbf{K}_{ed} \quad (2.36)$$

2.2.4. Mechanical Modeling of Semi Rigid Joints

The joint modeling approaches discussed in the previous sections are not usually suitable for use in available software for the analysis of framed structures. Many researchers proposed mechanical models for use in locally available software [1], accounting the various joint deformation modes.

The general behavior of semi rigid joint based on the component method of Eurocode3 was described in the previous section. The model used to generally describe the connection was shown in Fig 2.7.

The mechanical model shown below was first proposed by Tschemmerneegg et.al [1]. It was based on the general model shown in Fig. 2.7 and on the following deformation modes:

- Deformation caused by pure bending
- Deformation caused by pure shear
- Local deformations of the connection and the column regions opposite to the beam flanges

With this model, the bending and shear deformations of the joint zone are obtained simply by defining the correct cross section inertia moment and shear area for the members C_i-C_m and C_m-C_s , for the members B_i-B_m , B_m-B_s , D_i-D_m , D_m-D_s , and also for the members B_m-C_m , C_m-D_m , the cross section inertia moment and shear area should be infinite. The axial flexibility of the members B_i-C_i and C_i-D_i , must correspond to the sum of the local flexibilities of the connection and of the column, on the regions opposite to the lower flanges of the left and right beams, respectively. The same is valid for members B_s-C_s and C_s-D_s , but with the upper instead of the lower flanges of the beams. The members B_m-C_m and C_m-D_m must transmit only the beam shear forces to the column (the left and right beam bending moments and axial forces are transmitted by B_i-C_i/B_s-C_s and C_i-D_i/C_s-D_s , respectively); to provide for this, the link between members B_m-C_m and B_i-B_s , at B_m , and the link between members C_m-D_m and D_i-D_s , at D_m , only transmit vertical force.

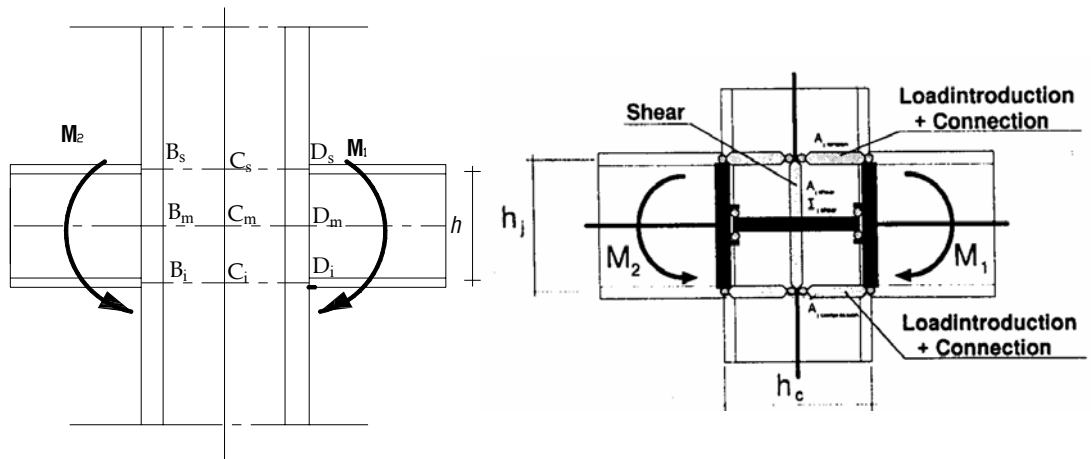


Fig 2.12 Mechanical semi rigid joint model proposed by Tschemmernegg et.al

CHAPTER THREE

BEHAVIOR AND MODELING OF SEMI RIGID FRAMES.

3.1 INTRODUCTION

The effect of semi rigid connections on the behavior of steel frames and their potential economical benefit is well recognized. The beam to column connection rotational behaviors directly affect the frame stability as it increases drift of the frame and causes a decrease in effective stiffness of the member which is connected to the joint. An increase in frame drift will multiply the second order (P- Δ) effect of beam-column members and thus will affect the over all stability of the frame. Hence, the non linear features of the beam to column connections have important function in structural steel design.

In order to incorporate the non – linear behavior of semi rigid connections in semi rigid frame analysis various modeling approaches have been proposed, as discussed in chapter two. Most of the approaches are based on experimental results. Various design codes, EC3 and EC4 for example, recently included in their edition methods to include the effect of semi rigid connections in the global analysis of partial strength frames.

The lack of a large and parameterized experimental data base doesn't allow for generating standardized functions. Thus, there is a need to be able to analytically generate a reliable moment rotation response of semi rigid connections which can be incorporated in the analysis and design of partial strength frames.

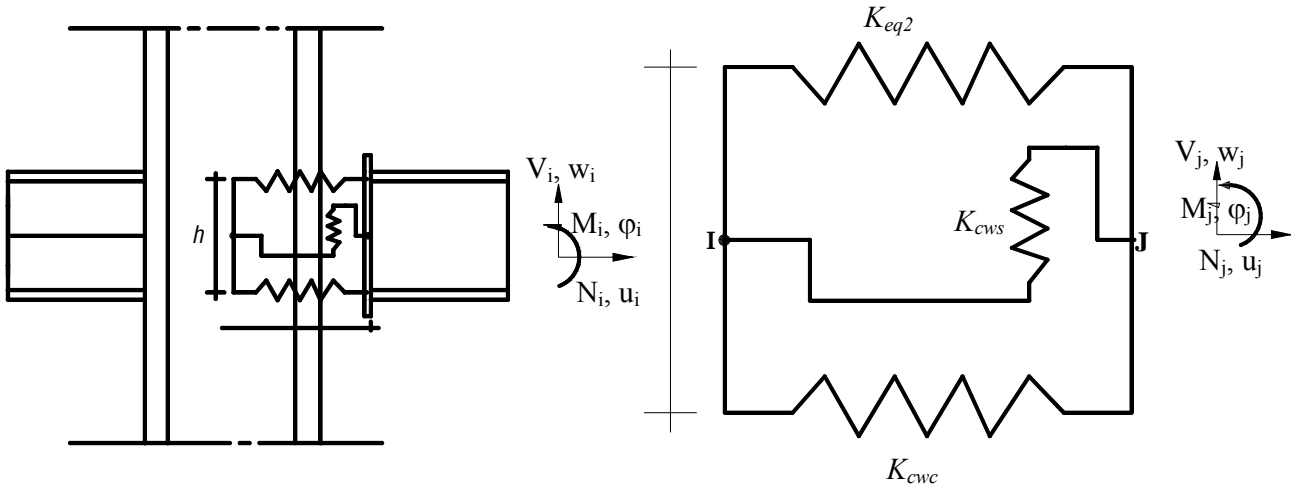
Lately E. Bayo et.al proposed a new four node element based on Eurocode component based method to model internal as well as external joints for the global analysis of steel and composite frames.

This new modeling scheme is not suitable to use in the standard available structural software. A two node simplified element whose stiffness matrix can be derived subsequently has been adopted in this study to investigate the static and dynamic behavior of partial strength frames.

ANSYS has an inbuilt element defined in its library called MATRIX27 characterized by six degrees of freedom ($U_x, U_y, U_z, \theta_x, \theta_y, \theta_z$) at each node. The stiffness coefficients of the two node element proposed in this study has been used as an input, appropriately, as a real constant for MATRIX27. A reference frame is analyzed by using the proposed two node element and the result is compared with the four node element proposed by Bayo et.al and the EC component based method β - factor. The proposed model gave an adequate result.

3.2. PROPOSED SIMPLIFIED EFFECTIVE COMPONENT METHOD JOINT MODELING

The four node element proposed in the previous section is not suitable for using in the commercial soft wares. In order to use the standard element library MATRIX27 of ANSYS 9 a two node element is proposed as shown below. The shear spring introduced represents the web panel deformation of the column web. This shear spring is assumed to be rigid in the horizontal direction but flexible in the vertical and rotational directions.



(a) Position of the connection element within the joint (b) Two node basic spring model

Fig. 3.1. Position and simplified effective modeling of semi rigid connections

In order to derive the stiffness matrix of the connection element, the simplest case of the element with only three springs is considered, as shown in Fig. 3.1 (b). As in the original

component method it is assumed that a joint will primarily deform in its plane, and all other out of plane and torsional degrees of freedom (DOF) are assumed to be rigidly connected. This reduces the problem to a two node element with three degrees of freedom per node, suitable to use the stiffness matrix element MATRIX27 available in the ANSYS library. If now each of these DOF is moved individually, as shown in Fig. 2.14, and the resulting spring forces are calculated, it is possible, with the help of some geometrical considerations and the spring characteristics, to derive the stiffness matrix of the basic connection element.

The resulting stiffness matrix for the basic two dimensional connection element, is shown in equation (3.1) below.

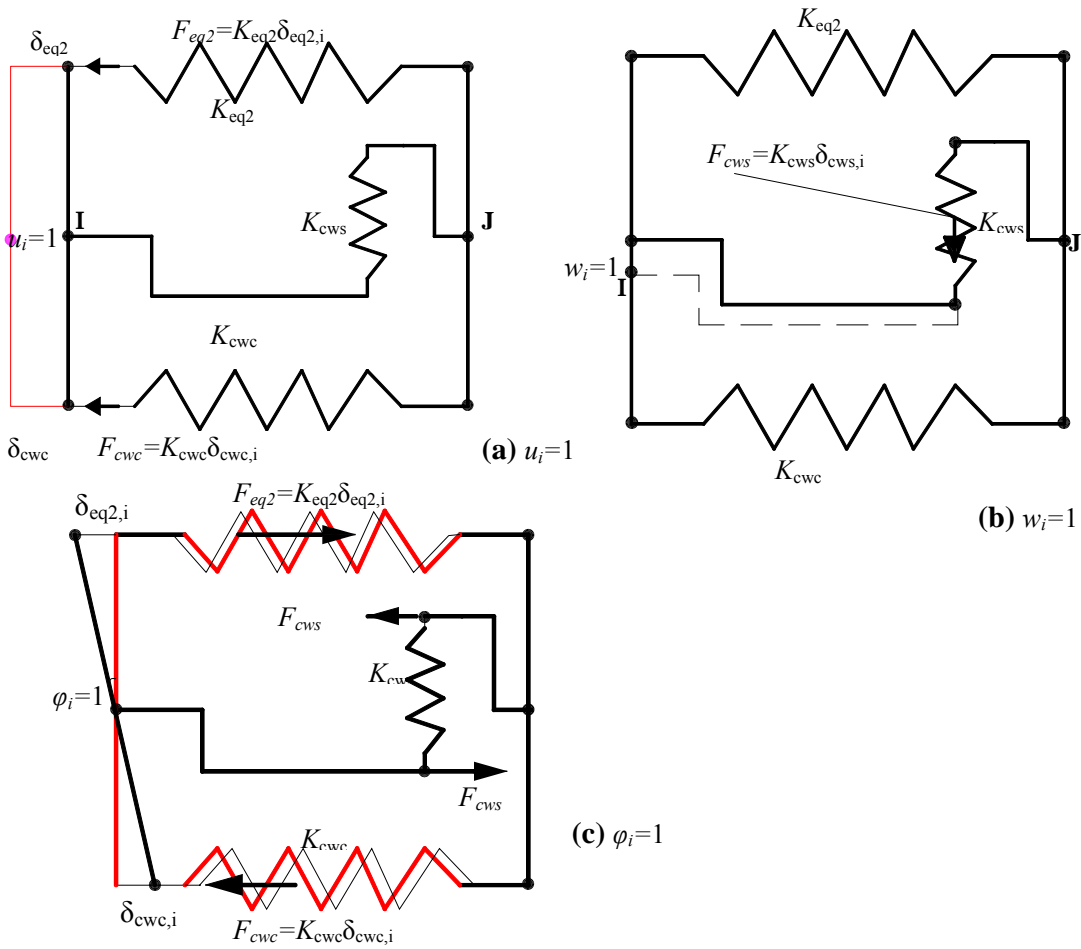


Fig.3.2. Stiffness derivation process for node I of the basic connection element.

$$\left[\begin{array}{cccccc}
(K_{cwc} + K_{eq2}) & 0 & -\frac{h}{2}(K_{eq2} - K_{cwc}) & -(K_{cwc} + K_{eq2}) & 0 & \frac{h}{2}(K_{eq2} - K_{cwc}) \\
0 & K_{cwc} & 0 & 0 & -K_{cwc} & 0 \\
-\frac{h}{2}(K_{eq2} - K_{cwc}) & 0 & \left(\frac{h^2}{\frac{1}{K_{cwc}} + \frac{1}{K_{cws}} + \frac{1}{K_{eq2}}} \right) & \frac{h}{2}(K_{eq2} - K_{cwc}) & 0 & -\left(\frac{h^2}{\frac{1}{K_{cwc}} + \frac{1}{K_{cws}} + \frac{1}{K_{eq2}}} \right) \\
-(K_{cwc} + K_{eq2}) & 0 & \frac{h}{2}(K_{eq2} - K_{cwc}) & (K_{cwc} + K_{eq2}) & 0 & -\frac{h}{2}(K_{eq2} - K_{cwc}) \\
0 & -K_{cwc} & 0 & 0 & K_{cwc} & 0 \\
\frac{h}{2}(K_{eq2} - K_{cwc}) & 0 & -\left(\frac{h^2}{\frac{1}{K_{cwc}} + \frac{1}{K_{cws}} + \frac{1}{K_{eq2}}} \right) & -\frac{h}{2}(K_{eq2} - K_{cwc}) & 0 & \left(\frac{h^2}{\frac{1}{K_{cwc}} + \frac{1}{K_{cws}} + \frac{1}{K_{eq2}}} \right)
\end{array} \right] \quad (3.1)$$

3.3 MODELLING OF SEMI RIGID FRAMES FOR STATIC ANALYSIS

3.3.1. The Investigated Structure

The investigated steel frame is a two bay three story system with geometric layout shown below. For comparison purposes the same frame with rigid connections has been investigated. In order to check the validation of the proposed element used in this thesis two story frame of the same connection characteristics shown in Fig 3.3 is analyzed. The characteristics of the connections computed based on EC3 component based method, are depicted in Table 3.2 and the locations of the joints are illustrated in Fig.3.3 and 3.4. ANSYS library element MATRIX 27 is used to model the proposed two node element of Fig 3.1.

The results are tabulated in Table 3.1. As can be seen from the table the proposed model give relatively good result as compared to EC3 approximate analysis and the four node accurate element proposed by Bayo et.al

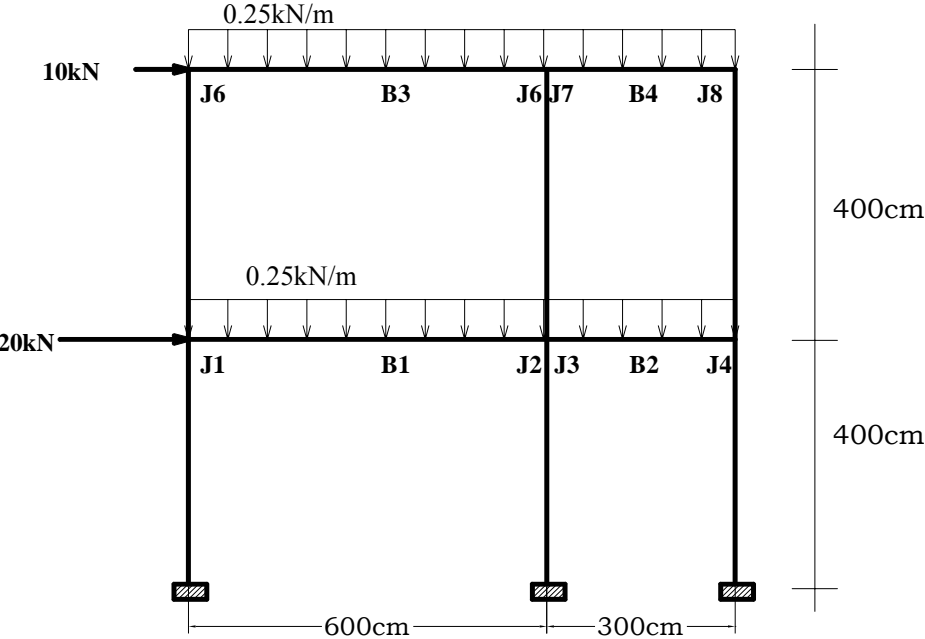


Fig. 3.3 Reference frame

Table 3.1. Comparison of results obtained by the proposed model, EC3 and the model of Bayo et al.

Location	Bayo et al. (1) [kNm]	Proposed model (2) [kNm]	using β of EC3 (3) [kNm]	Rigid [kNm]	$ \Delta $ wrt* (3)	$ \Delta $ wrt* (1)
J1	16.37	22.24	18.81	27.54	3.43	5.87
J2	78.11	75.28	71.78	92.04	3.50	2.83
J3	16.87	8.92	9.38	18.56	0.46	7.47
J4	18.67	23.34	19.83	23.69	3.51	4.67
J5	17.79	24.80	19.71	28.41	5.09	7.01
J6	63.77	58.37	57.18	74.35	1.19	5.40
J7	28.72	18.45	21.00	32.42	2.55	10.14
J8	8.33	13.15	9.26	11.10	3.89	4.82
B1	57.88	63.79	59.83	52.71	3.96	5.91
B2	6.72	12.01	9.90	7.01	2.11	5.29
B3	64.35	71.21	66.68	61.12	4.53	6.86
B4	5.98	12.34	9.37	13.00	2.97	6.36

* wrt – with respect to

Fig. 3.4 shows the frame to be investigated in the subsequent sections of this thesis. The static loads shown are arbitrarily chosen. The member cross sections and joint characteristics are the same as the reference frame of Fig. 3.3.

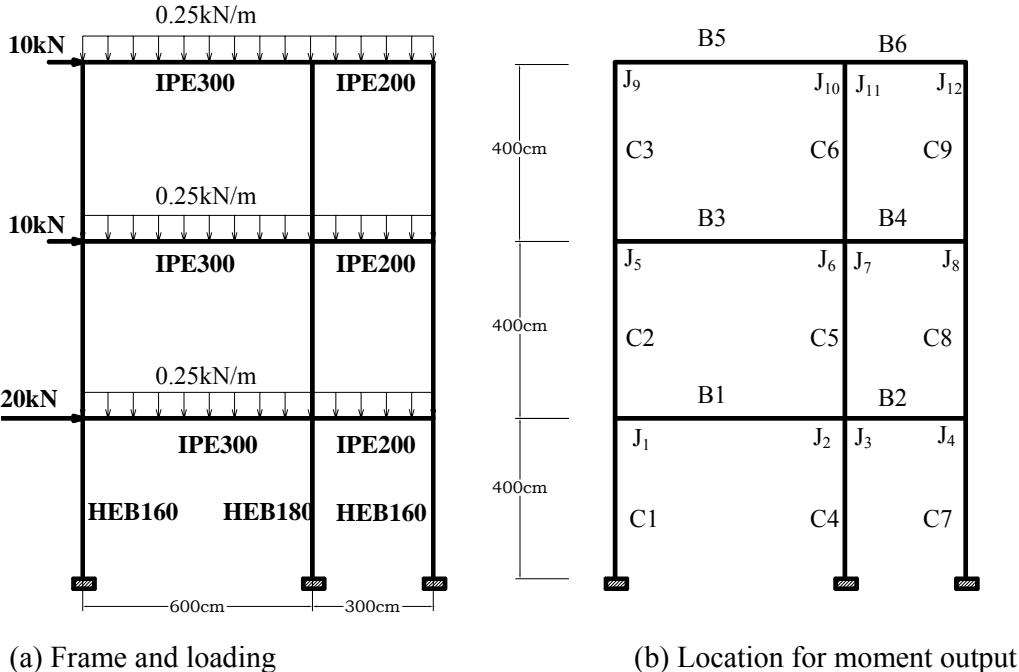


Fig. 3.4 The investigated frame

Table 3.2 Characteristics of the semi rigid connection used for frame in Figs 3.2 and 3.3.

Joint	Connection Characteristics					
	K_{cws} [kN/cm]	K_{cwc} [kN/cm]	K_{eq} [kN/cm]	Resistance [kN.cm]	h [cm]	d [cm]
J1	4852	20875	10790	7342	29.35	16
J2	5583	19420	10760	8306	29.35	18
J3	8432	29290	10130	5413	19.70	18
J4	7330	20465	10160	4824	19.70	16
J5	4852	20875	10790	7342	29.35	16
J6	5583	19420	10760	8306	29.35	18
J7	8432	29290	10130	5413	19.70	18
J8	7330	20465	10160	4824	19.70	16
J9	4852	20875	10790	7342	29.35	16
J10	5583	19420	10760	8306	29.35	18
J11	8432	29290	10130	5413	19.70	18
J12	7330	20465	10160	4824	19.70	16

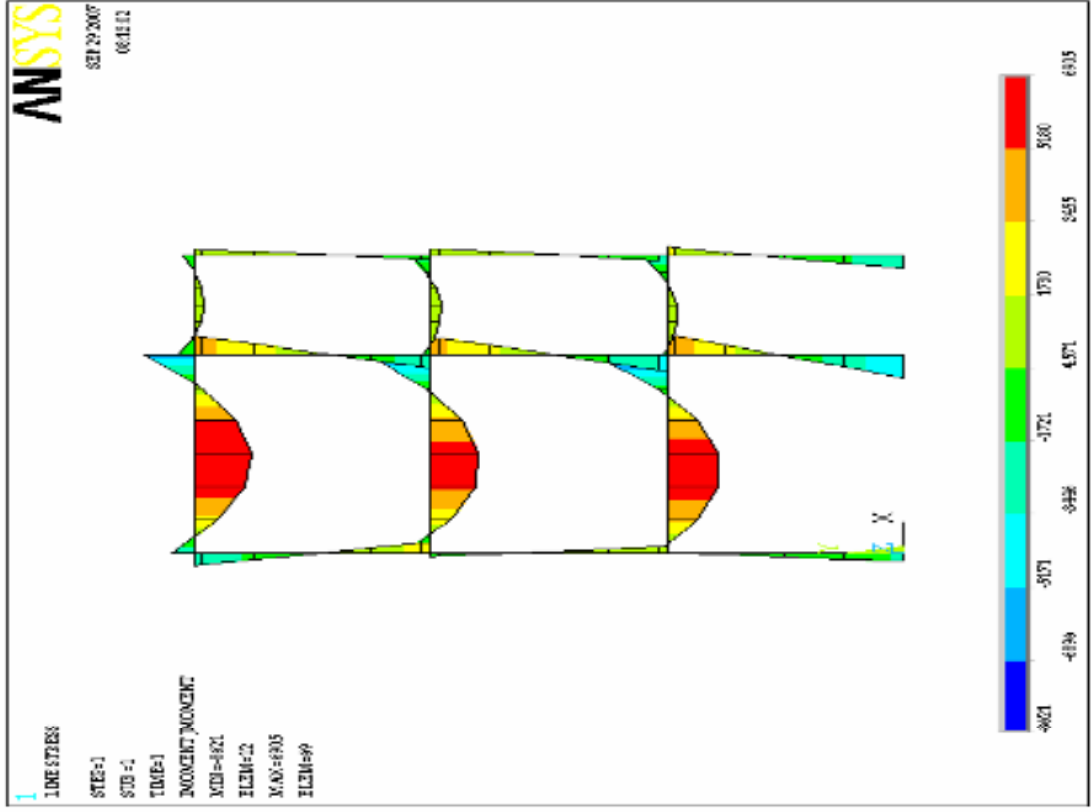
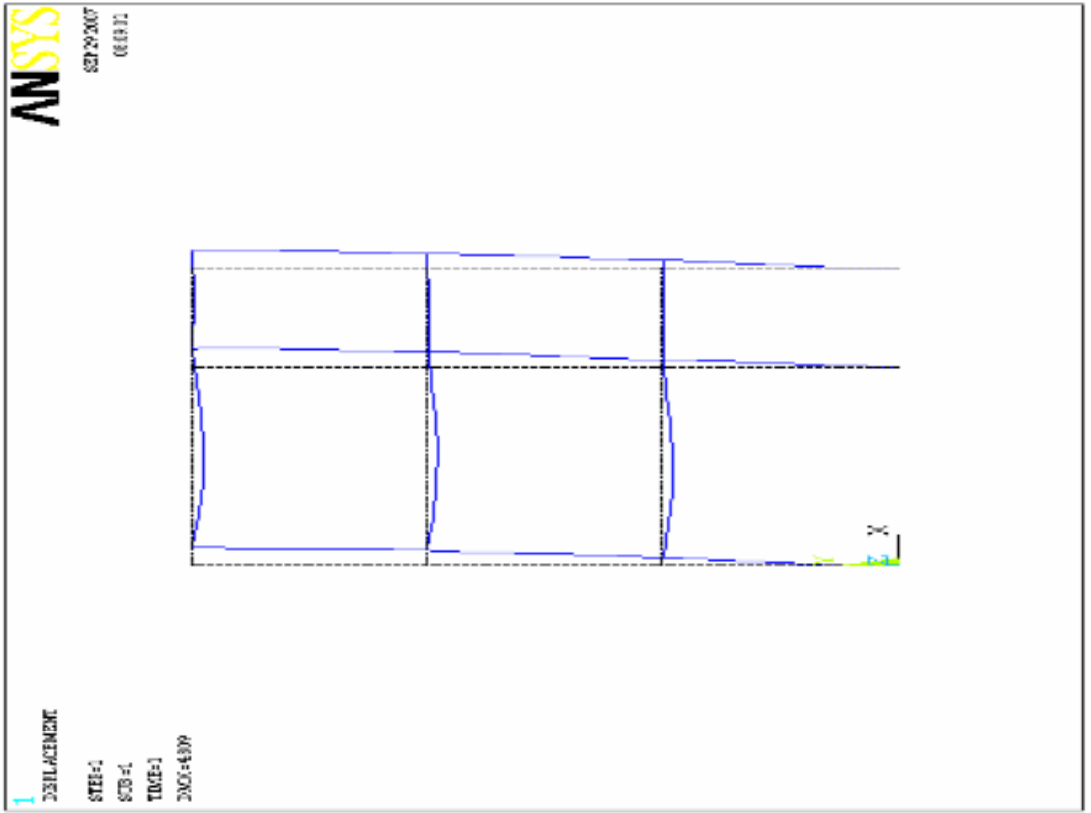
3.3.2 Finite Element Analysis of the Frame

Using the two node element of Fig. 3.1 and equation (3.1) the stiffness matrix of the element can be computed. The stiffness values are input as real constants in the element MATRIX27.

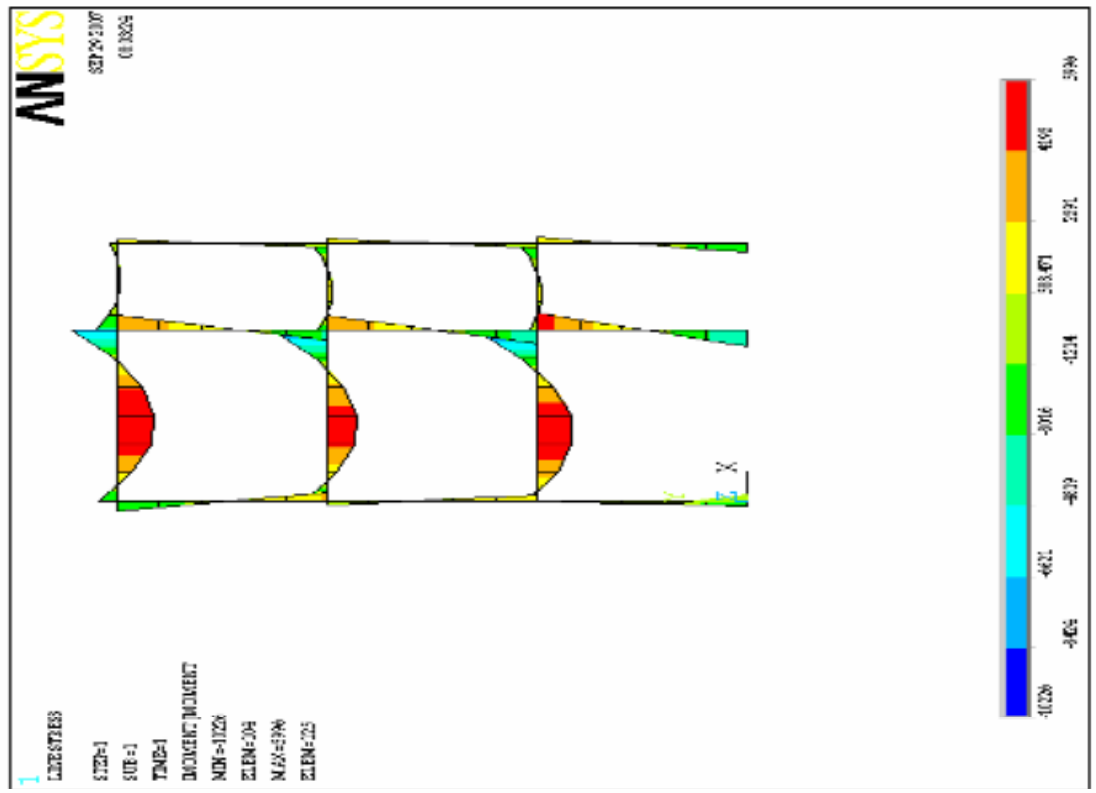
The connection region of the column has been allowed for shear deformation. The output result is given in Table 3.3. Comparison of moments (kNm) at the joints and beam mid-spans is shown. The results indicate that the moments at the semi rigid joints got reduced significantly which will reduce the cost of connection detailing. Though there is an increase in moment at mid spans usually steel sections, especially if laterally restrained, will be capable of resisting these moments as compared to the support region. It can be concluded from Table 3.2 that semi rigid connections will result in economical design solution.

Table 3.3 Comparisons of moments (kNm) at the joints and beam mid spans for semi-rigid and rigid connections.

Location	Rigid [kNm]	Semi Rigid [kNm]	$ \Delta $ [kNm]	% increase
J1	17.00	13.71	3.29	19.35
J2	102.26	86.21	16.05	15.69
J3	12.05	4.65	7.40	61.41
J4	31.72	29.32	2.40	7.57
J5	37.69	30.74	6.95	18.44
J6	89.96	75.61	14.35	15.95
J7	15.92	7.90	8.02	50.38
J8	24.51	23.27	1.24	5.06
J9	30.62	27.46	3.16	10.32
J10	74.46	59.44	15.02	20.17
J11	31.99	18.72	13.27	41.48
J12	12.10	13.44	-1.34	11.07
B1	52.87	62.54	9.67	18.29
B2	6.24	11.14	4.90	78.53
B3	48.68	59.33	10.65	21.88
B4	7.91	12.54	4.63	58.53
B5	59.96	69.05	9.09	15.16
B6	6.08	12.04	5.96	98.03
δ_{\max} (cm)	3.90	4.81	0.91	23.18



(a) Semi-rigid frame



(b) Rigid frame

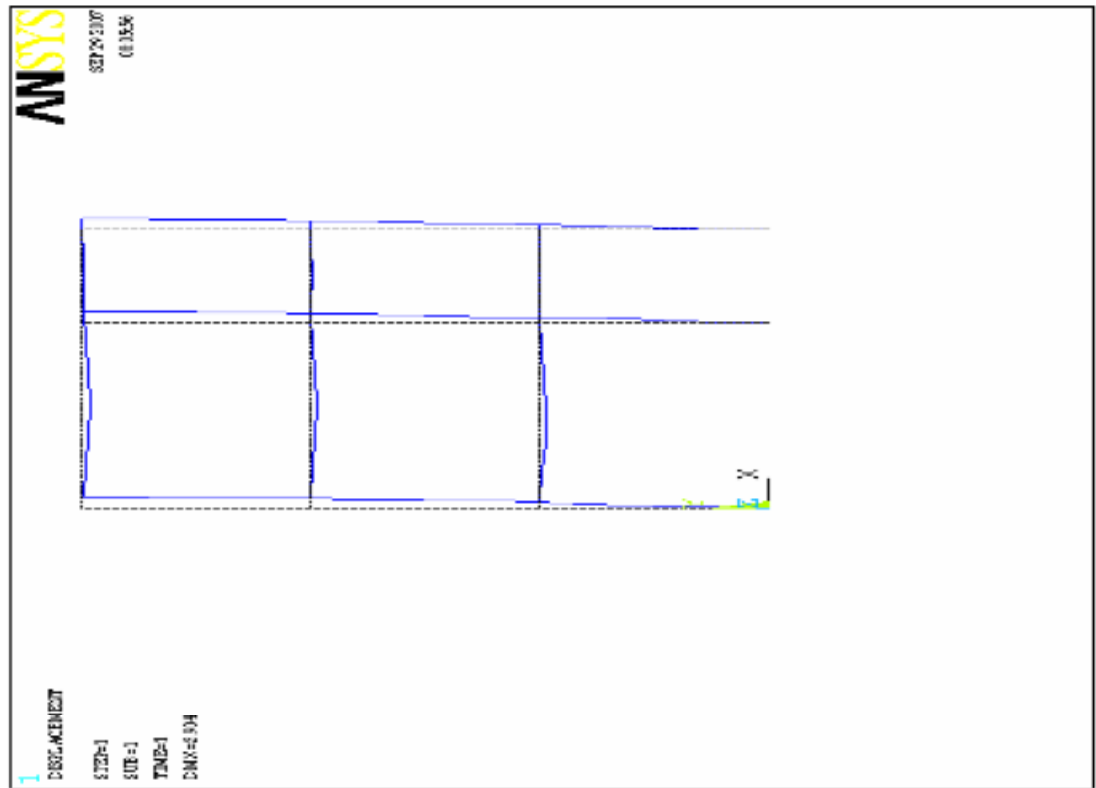


Fig. 3.5 Static analysis results of semi rigid and rigid frames

3.4. DYNAMIC ANALYSIS AND RESPONSE OF SEMI RIGID FRAMES

3.4.1. Introduction

The 1994 Northridge earthquake (USA) has shown the vulnerability of rigid connections in steel moment resisting frames subject to severe earthquake ground motions. Since then, as an alternative connection type, semi rigid connections are considered for the retrofit and new design of steel moment resisting frames in high seismic regions [20].

In seismic design, all steel frames are expected to suffer large inelastic deformation to dissipate energy during strong ground motion [12, 25]. In the current design codes no analysis or design guidance is given for semi rigid frames although the possibility of using them as the main energy dissipative mechanism has been recognized.

In this thesis the analysis and response of a typical semi rigid frame depicted in Fig3.4 is carried out using a general finite element package, ANSYS. For comparison purpose the same frame but rigidly connected is studied. The frames are subjected to the same earthquake and founded on soil class B. The peak ground acceleration is 0.1g (seismic hazard zone 4 as per EBCS-8) and the normalized elastic acceleration response- spectrum corresponding to the stated soil condition is used [21].

3.4.2. Method of Analysis and Analysis Results

For the given frame geometry, connection stiffness and member size shown in Fig.3.2 and Table3.1 modal and spectral analysis is carried out. Modal analysis is used to determine the vibration characteristics (natural frequencies and mode shapes) of the frames. The natural frequencies and mode shapes are important parameters in the design of a structure for dynamic loading conditions. They are also required if you want to do a spectrum analysis or a mode superposition harmonic or transient analysis.

3.4.2.1 Modal analysis

Modal analysis in the ANSYS family of products is a linear analysis. Any non-linearities, such as plasticity and contact (gap) elements, are ignored even if they are defined. A choice can be made from several mode-extraction methods: Block Lanczos, subspace, Power Dynamics, reduced, unsymmetrical, damped, and QR damped. The damped and QR damped methods allow to include damping in the structure. The QR Damped method also allows for unsymmetrical damping and stiffness matrices [23].

The basic equation solved in a typical undamped modal analysis is the classical eigen value problem:

$$[\mathbf{K}]\{\varphi_i\} = \omega_i^2 [\mathbf{M}]\{\varphi_i\} \quad (3.2)$$

where :

$[\mathbf{K}]$ stiffness matrix

$\{\varphi_i\}$ mode shape vector (eigen vector) of mode i

Ω_i natural frequency of mode i (ω_i^2 is the eigen value)

$[\mathbf{M}]$ mass matrix

Many numerical methods are available to solve the above equation. ANSYS offers these methods:

- Block Lanczos method
- Subspace method
- Power Dynamics method
- Damped method
- Reduced(Householder) method
- Unsymmetric method
- QR damped method

The damped, unsymmetric, and QR damped methods are not available in the ANSYS Professional program.

a. Block Lanczos method

It uses the Lanczos algorithm where the Lanczos recursion is performed with a block of vectors. This method is as accurate as the subspace method, but faster. The Block Lanczos method is especially powerful when searching for eigen frequencies in a given part of the eigen value spectrum of a given system. The convergence rate of the eigen frequencies will be about the same when extracting modes in the midrange and higher end of the spectrum as when extracting the lowest modes. Therefore, when you use a shift frequency (FREQB) to extract n modes beyond the starting value of FREQB, the algorithm extracts the n modes beyond FREQB at about the same speed as it extracts the lowest n modes.

b. Subspace method

The *subspace* method uses the subspace iteration technique, which internally uses the generalized Jacobi iteration algorithm. It is highly accurate because it uses the full \mathbf{K} and \mathbf{M} matrices. For the same reason, however, the subspace method is slower than the reduced method. This method is typically used in cases where high accuracy is required or where selecting master DOF is not practical.

c. Power dynamics method

The Power Dynamics method internally uses the subspace iterations, but uses the PCG iterative solver. This method may be significantly faster than either the subspace or the Block Lanczos methods, but may not converge if the model contains poorly-shaped elements, or if the matrix is ill-conditioned. This method is especially useful in very large models (100,000+ DOFs) to obtain a solution for the first few modes. It is not recommended to use this method if you will be running a subsequent spectrum analysis. The Power Dynamics method does not perform a Sturm sequence check (that is, it does not check for missing modes), which might affect problems with multiple repeated frequencies. This method always uses lumped mass approximation.

d. Reduced method

The reduced method uses the HBI algorithm (Householder-Bisection-Inverse iteration) to calculate the eigen values and eigenvectors. It is relatively fast because it works with a small

subset of degrees of freedom called master DOF. Using master DOF leads to an exact \mathbf{K} matrix but an approximate \mathbf{M} matrix (usually with some loss in mass). The accuracy of the results, therefore, depends on how well \mathbf{M} is approximated, which in turn depends on the number and location of masters.

e. Unsymmetrical method

The unsymmetric method, which also uses the full \mathbf{K} and \mathbf{M} matrices, is meant for problems where the stiffness and mass matrices are unsymmetric (for example, acoustic fluid-structure interaction problems). It uses the Lanczos algorithm which calculates complex eigenvalues and eigenvectors if the system is non-conservative (for example, a shaft mounted on bearings). The real part of the eigenvalue represents the natural frequency and the imaginary part is a measure of the stability of the system - a negative value means the system is stable, whereas a positive value means the system is unstable. Sturm sequence checking is not available for this method. Therefore, missed modes are a possibility at the higher end of the frequencies extracted.

f. Damped method

The *damped* method is meant for problems where damping cannot be ignored, such as rotor dynamics applications. It uses full matrices (\mathbf{K} , \mathbf{M} , and the damping matrix \mathbf{C}). It uses the Lanczos algorithm and calculates complex eigen values and eigenvectors (as described below). Sturm sequence checking is not available for this method. Therefore, missed modes are a possibility at the higher end of the frequencies extracted.

g. Damped method-real and imaginary parts of the eigen-value

The imaginary part of the eigen value, Ω , represents the steady-state circular frequency of the system. The real part of the eigen value, σ , represents the stability of the system. If σ is less than zero, then the displacement amplitude will decay exponentially, in accordance with $\text{EXP}(\sigma)$. If σ is greater than zero, then the amplitude will increase exponentially. (Or, in other words, negative σ gives an exponentially decreasing, or stable, response; and positive σ gives an exponentially increasing, or unstable, response.) If there is no damping, the real component

of the eigen value will be zero. The eigen value results reported by ANSYS are actually divided by 2π . This gives the frequency in Hz (cycles/second). In other words:

$$\text{Imaginary part of eigen value, as reported} = \frac{\Omega}{2\pi}$$

$$\text{Real part of eigen value, as reported} = \frac{\sigma}{2\pi}$$

h. Damped method-real and imaginary parts of the eigenvector

In a damped system, the response at different nodes can be out of phase. At any given node, the amplitude will be the vector sum of the real and imaginary components of the eigenvector.

i. QR damped method

The QR damped method combines the advantages of the Block Lanczos method with the complex Hessenberg method. The key concept is to approximately represent the first few complex damped eigen values by modal transformation using a small number of eigenvectors of the undamped system. After the undamped mode shapes are evaluated by using the real eigen solution (Block Lanczos method), the equations of motion are transformed to these modal coordinates.

Using the QR algorithm, a smaller eigenvalue problem is then solved in the modal subspace. This approach gives good results for lightly damped systems and can also be applicable to any arbitrary damping type (proportional or non-proportional symmetric damping or nonsymmetrical gyroscopic damping matrix). This approach also supports nonsymmetrical stiffness if present in the model.

The sub space method is used in this study for its accuracy.

3.4.2.2. Spectral analysis

Two types of spectrum analyses are supported by ANSYS: the deterministic response spectrum method and the nondeterministic random vibration method. Both excitation at the

support and excitation away from the support are allowed. The three response spectrum methods are the single-point, multiple-point and dynamic design analysis method. The random vibration method uses the power spectral density (PSD) approach.

a. Assumptions and restrictions

1. The structure is linear.
2. For single-point response spectrum analysis and dynamic design analysis method, the structure is excited by a spectrum of known direction and frequency components, acting uniformly on all support points or on specified unsupported master degrees of freedom (DOFs).
3. For multi-point response spectrum and power spectral density analyses, the structure may be excited by different input spectra at different support points or unsupported nodes. Up to ten different simultaneous input spectra are allowed.

b. Description of analysis

The spectrum analysis capability is a separate analysis type and it must be preceded by a modal analysis. If mode combinations are needed, the required modes must also be expanded, as described in modal analysis.

The four options available are the single-point response spectrum method (SPRS), the dynamic design analysis method (DDAM – essentially a response spectrum analysis in which the spectrum is obtained from a series of empirical equations and shock design tables provided in the U.S. Naval Research Laboratory Report NRL-1396.), the random vibration method (PSD – statistical measure defined as the limiting mean-square value of a random variable. It is used in *random vibration analyses* in which the instantaneous magnitudes of the response can be specified only by probability distribution functions that show the probability of the magnitude taking a particular value.) and the multiple-point response spectrum method (MPRS – similar to PSD but with no correlation between spectra). Each option is discussed in detail in ANSYS Release 9.0 documentation [23]. The only method available in the ANSYS Professional program is the single-point response spectrum, discussed in detail subsequently.

3.4.2.2.1 Single-point response spectrum

Both excitation at the support (base excitation) and excitation away from the support (force excitation) are allowed for the single-point response spectrum analysis (SPRS). The table below summarizes these options as well as the input associated with each.

Table 3.4 Types of Spectrum Loading

	Excitation Option			
	Excitation at support		Excitation away from support	
Spectrum input	Response spectrum table		Amplitude multiplier table	
Orientation of load	Direction vector		x, y, z direction at each node	
Distribution of loads	Constant on all support points		Amplitude in x, y, or z directions	
Type of input	Velocity	Acceleration	Displacement	Force

i. Damping

Damping is evaluated for each mode and is defined as:

$$\xi'_i = \frac{\beta\omega_i}{2} + \xi_c + \frac{\sum_{j=1}^{N_m} \beta_j^m E_j^s}{\sum_{j=1}^{N_m} E_j^s} + \xi_i^m \quad (3.3)$$

where :

ξ'_i effective damping ratio for mode i

β beta damping

ω_i undamped natural circular frequency of the ith mode

ξ_c damping ratio (input as RATIO, DMPRAT command)

N_m number of materials

β_j^m damping constant stiffness matrix multiplier for material j

$$E_j^s = \frac{1}{2} \{\varphi_i\}^T [K_j] \{\varphi_i\} = \text{strain energy}$$

$\{\varphi_i\}$ displacement vector for mode i

$[K_j]$ stiffness matrix of part of structure of material j

ξ_i^m modal damping ratio of mode i

ii. Participation factors and mode coefficients

The participation factors for the given excitation directions are defined as:

$$\gamma_i = \{\varphi_i\}^T [M] \{D\} \quad \text{for the base excitation option} \quad (3.4a)$$

$$\gamma_i = \{\varphi_i\}^T \{F\} \quad \text{for the force excitation option} \quad (3.4b)$$

where :

- γ_i participation factor for the i th mode
- $\{\phi\}_i$ normalized eigenvector
- $[D]$ vector describing the excitation direction
- $\{F\}$ input force vector

The vector describing the excitation direction has the form:

$$\{D\} = [T]\{e\} \quad (3.4c)$$

where:

$$\{D\} = [D_1^a \quad D_2^a \quad D_3^a \quad D_{4\dots}^a]$$

D_j^a = excitation at DOF j in direction a ; a may be either x, y, z , or rotations about one of these axes

$$[T] = \begin{bmatrix} 1 & 0 & 0 & 0 & (z - z_0) & (-y + y_0) \\ 0 & 1 & 0 & (-z + z_0) & 0 & (x - x_0) \\ 0 & 0 & 1 & (y - y_0) & (-x + x_0) & 0 \\ 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix} \quad (3.4d)$$

where:

x, y, z global Cartesian coordinates of a point on the geometry

x_0, y_0, z_0 global Cartesian coordinates of point about which rotations are done (reference point)

$\{e\}$ six possible unit vectors

We can calculate the statically equivalent actions at j due to rigid-body displacements of the reference point using the concept of translation of axes [T]. For spectrum analysis, the determination of D_a values may be obtained accurately from [23].

iii. *Combination of modes*

The modal displacements, velocity and acceleration may be combined in different ways to obtain the response of the structure. For all excitations but the PSD this would be the maximum response, and for the PSD excitation, this would be the $1-\sigma$ (standard deviation) relative response. The response includes DOF response as well as element results and reaction forces if computed in the expansion operations

In the case of the single-point response spectrum method or the dynamic-design analysis method options of the spectrum analysis, it is possible to expand only those modes whose significance factor exceeds the significant threshold value. Note that the mode coefficients must be available at the time the modes are expanded.

Only those modes having significant amplitude (mode coefficient) are chosen for mode combination. A mode having a coefficient of greater than a given value of the maximum mode coefficient (all modes are scanned) is considered significant.

The spectrum option provides five options for the combination of modes. They are:

- Complete Quadratic Combination Method (CQC)
- Grouping Method (GRP)
- Double Sum Method (DSUM)
- SRSS Method
- NRL-SUM Method (NRL SUM)

These methods generate coefficients for the combination of mode shapes. This combination is done by a generalization of the method of the square root of the sum of the squares which has the form:

$$R_a = \left(\sum_{i=1}^N \sum_{j=1}^N \varepsilon_{ij} R_i R_j \right)^{\frac{1}{2}} \quad (3.5)$$

where:

R_a total modal response

N total number of expanded modes

ε_{ij} coupling coefficient.

The value of $\varepsilon_{ij} = 0.0$ implies modes i and j are independent and approaches 1.0 as the dependency increases

$R_i = A_i \Psi_i$ modal response in the i^{th} mode

$R_j = A_j \Psi_j$ modal response in the j^{th} mode

A_i mode coefficient for the i^{th} mode

A_j mode coefficient for the j^{th} mode

Ψ_i the i^{th} mode shape

Ψ_j the j^{th} mode shape

Ψ_i and Ψ_j may be the DOF response, reactions, or stresses. The DOF response, reactions, or stresses may be displacement, velocity or acceleration.

- ***Complete Quadratic Combination Method***

This method is based on Wilson, et al

$$R_a = \left(\sum_{i=1}^N \sum_{j=1}^N k \varepsilon_{ij} R_i R_j \right)^{\frac{1}{2}} \quad (3.6)$$

where :

$$k = \begin{cases} 1 & \text{for } i = j \\ 2 & \text{for } i \neq j \end{cases}$$

$$\varepsilon_{ij} = \frac{8(\xi'_i \xi'_j)^{\frac{1}{2}} (\xi'_i + r \xi'_j) r^{\frac{3}{2}}}{(1-r^2)^2 + 4\xi'_i \xi'_j r(1+r^2) + 4(\xi_i'^2 + \xi_j'^2) r^2}$$

$$r = \frac{\omega_j}{\omega_i}$$

- **Grouping Method**

This method is from the NRC Regulatory Guide. For this case, equation 3.4 specializes to:

$$R_a = \left(\sum_{i=1}^N \sum_{j=1}^N \varepsilon_{ij} |R_i R_j| \right)^{\frac{1}{2}} \quad (3.7)$$

where :

$$\varepsilon_{ij} = \begin{cases} 1.0 & \text{if } \left| \frac{\omega_j - \omega_i}{\omega_i} \right| \leq 0.1 \\ 0.0 & \text{if } \left| \frac{\omega_j - \omega_i}{\omega_i} \right| > 0.1 \end{cases}$$

Closely spaced modes are divided into groups that include all modes having frequencies lying between the lowest frequency in the group and a frequency 10% higher. No one frequency is to be in more than one group.

- **Double Sum Method**

The Double Sum Method also is from the NRC Regulatory Guide. For this case, [equation 3.4](#) specializes to:

$$R_a = \left(\sum_{i=1}^N \sum_{j=1}^N \varepsilon_{ij} |R_i R_j| \right)^{\frac{1}{2}} \quad (3.8a)$$

where :

$$\varepsilon_{ij} = \frac{1}{1 + \left(\frac{\omega_i' - \omega_j'}{\xi_i'' \omega_i + \xi_j'' \omega_j} \right)^2}$$

ω_i' = damped natural circular frequency of the ith mode

ω_i = undamped natural circular frequency of the ith mode

ξ_i'' = modified damping ratio of the ith mode

The damped natural frequency is computed as :

$$\omega_i' = \omega_i \left(1 - (\xi_i')^2 \right)^{\frac{1}{2}}$$

The modified damping ratio ξ_i'' is defined to account for the earthquake duration time:

$$\xi_i'' = \xi_i' + \frac{2}{t_d \omega_i} \quad (3.8b)$$

where:

t_d = earthquake duration time, fixed at 10 units of time

- **SRSS Method**

The SRSS (Square Root of the Sum of the Squares) Method is from the NRC Regulatory Guide. For this case, Eq. 3.4 reduces to:

$$R_a = \left(\sum_{i=1}^N (R_i)^2 \right)^{\frac{1}{2}} \quad (3.9)$$

- ***NRL-SUM Method***

The NRL-SUM (Naval Research Laboratory Sum) method (O'Hara and Belsheim) calculates the maximum modal response as:

$$R_a = |R_{a1}| + \left(\sum_{i=2}^N (R_{ai})^2 \right)^{\frac{1}{2}} \quad (3.10)$$

where:

$|R_{a1}|$ absolute value of the largest modal displacement, stress or reaction at the point

R_{ai} displacement, stress or reaction contributions of the same point from other modes.

3.4.2.2.2 Analysis results

- ***Free vibration response***

The main objective of the thesis is to investigate how connection flexibility influences the dynamic response of semi rigid frames. As shown in the previous sections the frames under investigation are three storey frames of the same geometry, cross section and material properties but one rigidly connected and the other being flexible.

Modal analysis is carried out using the subspace method and the results are shown in Table 3.5. As can be seen readily from Table 3.5, connection flexibility increases the natural period of vibration of the frame, especially in the lower modes, while it tends to decrease the frequency. Thus semi rigid frames will be subjected to lower lateral seismic loads as compared to rigid ones.

Table 3.5: Modal Analysis results, natural period and frequency of rigid and semi-rigid frames.

Mode	Rigid Frame		Semi-rigid Frame		Increase in period	% increase
	ω_i (rad/sec)	T_n (sec)	ω_i (rad/sec)	T_n (sec)		
1	3.641	1.726	3.195	1.966	0.241	13.945
2	11.594	0.542	10.608	0.592	0.050	9.295
3	19.886	0.316	19.263	0.326	0.010	3.234
4	31.206	0.201	29.979	0.210	0.008	4.093
5	35.944	0.175	33.440	0.188	0.013	7.488

where:

ω natural frequency

T_n natural period

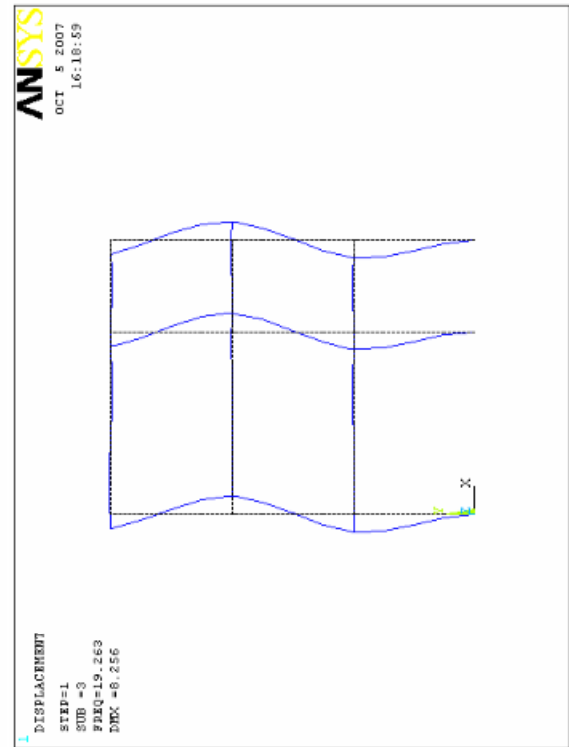
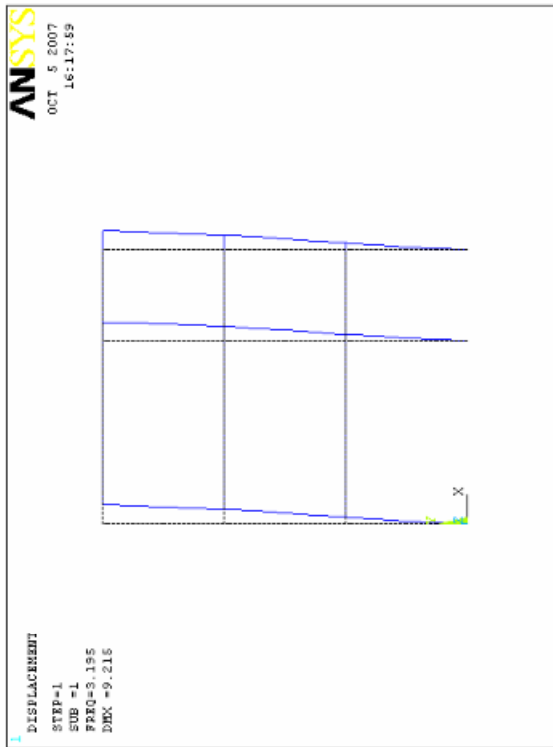
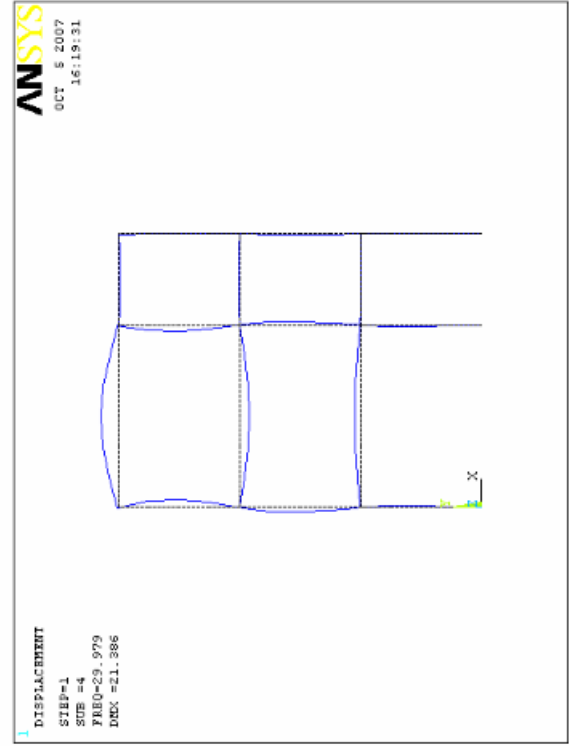
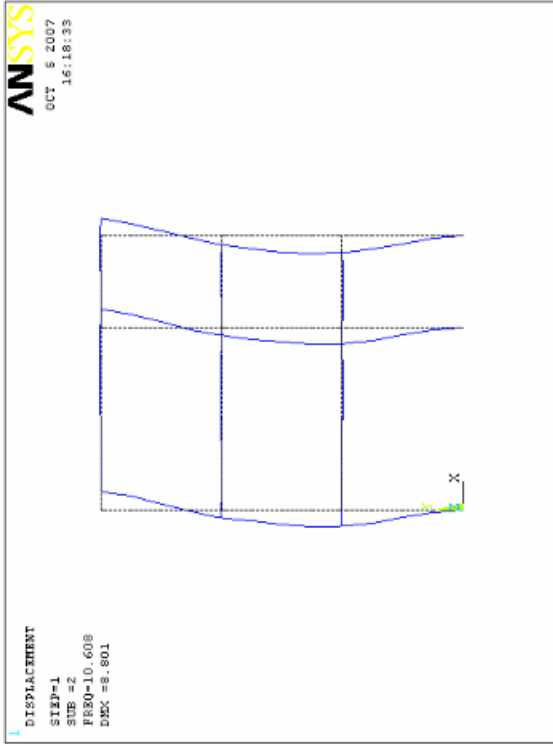


Fig.3.6 The first four modal shape of semi rigid frame.

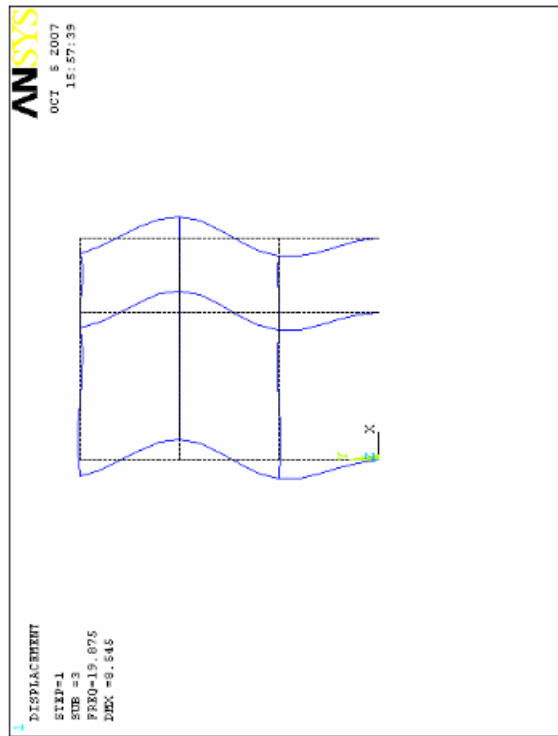
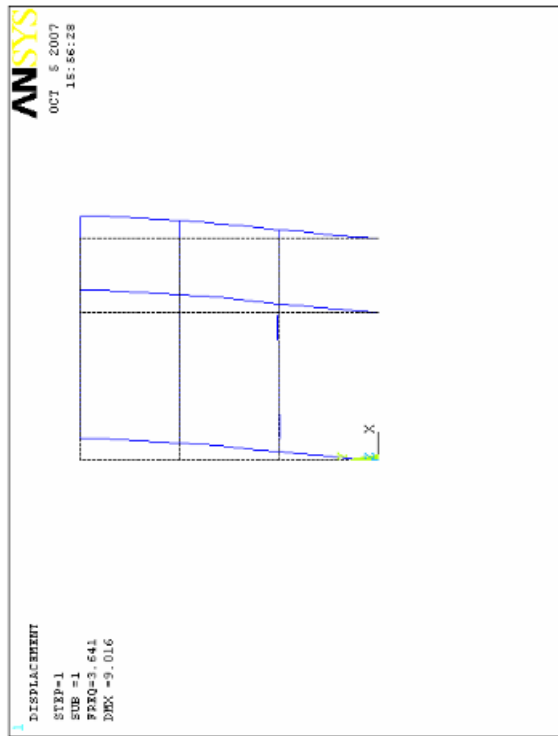
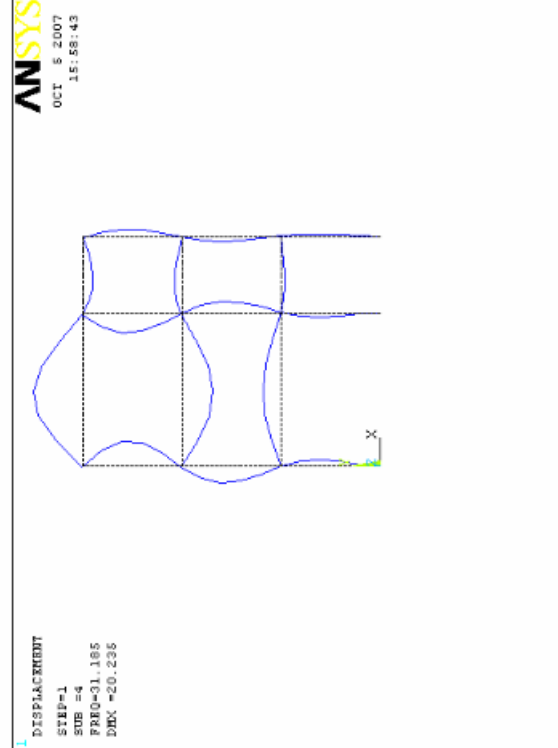
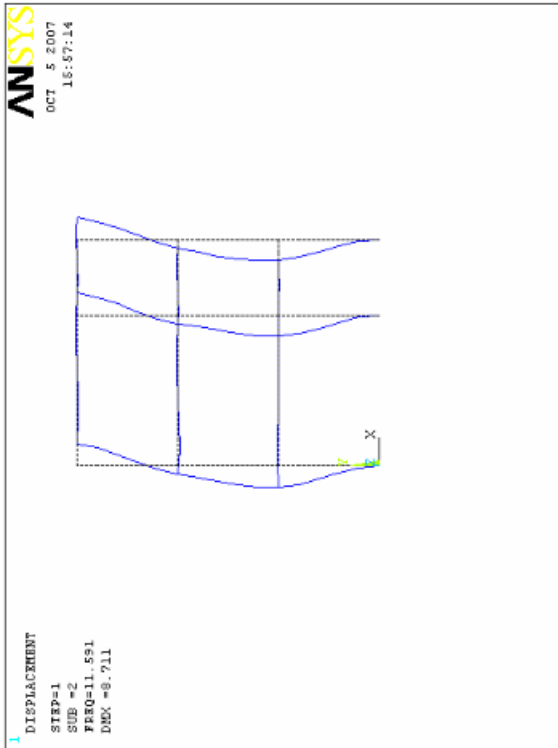


Fig.3.7 The first four modal shape of rigid frame

- *Earthquake responses*

In order to study the earthquake response of the frames using elastic design spectrum of EBCS-8 soil class B and seismic hazard zone of 4 is taken. Both the semi rigid and rigid frames are subjected to the same earthquake and the responses are tabulated as shown in table below.

As can be seen from the table the shear forces and internal moments in rigid frames are more than the semi rigid frame for the same earthquake. The higher the natural period of the frame the lower the action effect of the earthquake, which resulted in the lower shear force and bending moments.

Table 3:6 Spectral analysis results: story shear, drift, joint moment and span moment

Story level	Story Shear		Inter story drift	
	Semi rigid [kN]	Rigid [kN]	Semi rigid [cm]	Rigid [cm]
0	26.14	53.73	0.00	0.00
1	21.86	43.29	3.14	3.37
2	12.24	21.39	3.89	3.72
3	12.24	21.39	2.18	1.93

Table 3.6 (cont'd)

Location	Moment		Δ [kNm]	% increase
	Rigid [kNm]	Semi- Rigid [kNm]		
J1	72.61	53.47	19.14	35.80
J2	71.54	55.37	16.17	29.20
J3	43.55	26.08	17.47	66.99
J4	49.38	32.94	16.44	49.91
J5	52.78	41.43	11.35	27.40
J6	52.32	42.93	9.39	21.87
J7	30.78	19.90	10.88	54.67
J8	34.17	24.57	9.60	39.07
J9	17.83	16.06	1.77	11.02
J10	17.94	16.85	1.09	6.47
J11	10.45	7.92	2.53	31.94
J12	11.89	10.11	1.78	17.61
B1	23.54	19.14	4.40	22.99
B2	18.41	13.28	5.13	38.63
B3	17.33	14.85	2.48	16.70
B4	12.53	9.75	2.78	28.51
B5	6.05	5.90	0.15	2.54
B6	4.44	4.10	0.34	8.29
C1	50.44	42.44	8.00	18.85
C2	35.57	28.09	7.48	26.63
C3	15.95	12.01	3.94	32.81
C4	78.31	64.97	13.32	20.50
C5	56.4	42.57	13.83	32.49
C6	24.52	17.43	7.09	40.68
C7	45.33	37.76	7.57	20.05
C8	22.48	16.62	5.86	35.26
C9	8.44	4.97	3.47	69.82

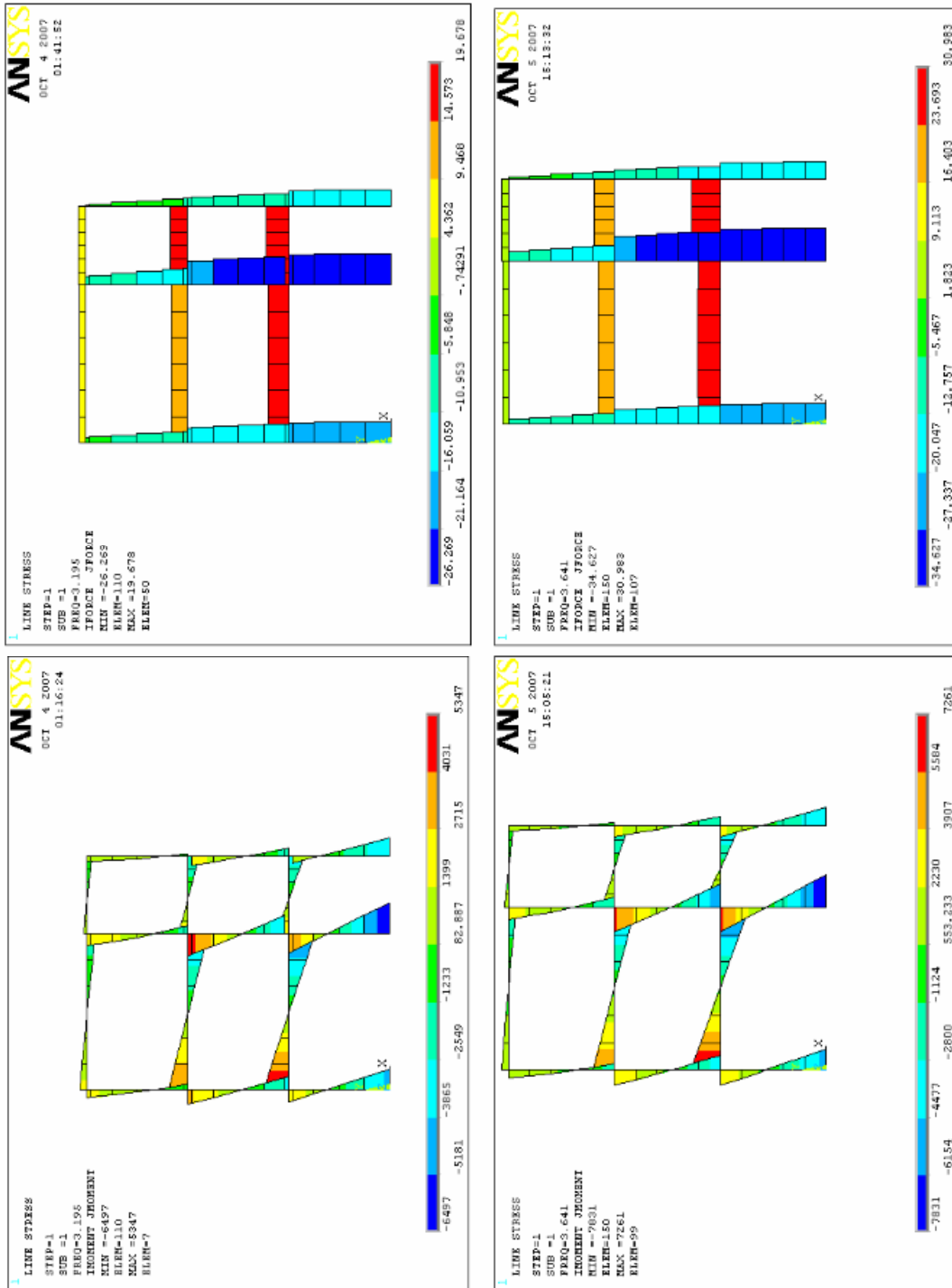


Fig.3.8 Moment and shear diagram of semi rigid and rigid frames subjected to an earthquake respectively

CHAPTER FOUR

CONCLUSION AND RECOMMENDATIONS

4.1. CONCLUSIONS

In this study a two node simplified connection element is proposed. The panel shear deformation of the joint has been taken care of by allowing the joint region of the column to undergo shear deformation. The proposed model has given a good result as compared to the EC3 component method, using the interaction parameter β – factor. Though there are some discrepancies between the results obtained by Bayo et.al[3] and the proposed model at some locations, it will be suitable for design purposes.

The static analysis response of the frames has indicated that a reduction in the joint moment is accompanied by an increase in the span moments. Reducing joint moments is advantageous as detailing, modeling and design of joints is the most cumbersome part of steel frame design. In composite construction beams are usually laterally restrained and have sufficient strength to sustain design loads in their span than connection region. This will make semi rigid connections an economical design solution.

From the dynamic analysis response of the two frames it can be concluded that joint flexibility will increase the natural period, and hence decrease the natural frequency, of the frame especially in the fundamental mode. Correspondingly the stresses produced due to an earthquake excitation in the semi rigid frame elements are significantly lower than the rigid frame elements. This indicates that semi rigid frame design will give a good solution in earthquake prone areas. The large deformation capability of semi rigid frames will increase their energy dissipating capacity, provided the lateral deformation is within the limits.

4.2. RECOMMENDATIONS

In this thesis the planar static and dynamic response behavior of semi rigid frame is studied using a simplified two node connection modeling based on the EC component method. The model is easy and suitable for use in day to day semi rigid steel frame design. In the model the real dimension of the connection elements has not been taken into account. It is clear that further and extensive research work needs to be done in order to accurately incorporate the behavior of semi rigid connections in the analysis and design of semi rigid frames.

The following are among the areas of semi rigid connections which need further research.

1. Efficient semi rigid connection modeling in 3-D
2. Development of computer program incorporating accurately the behavior of semi rigid connections.
3. Extensive investigation of semi rigid connections in order to provide design methods, and detailing and fabrication criteria for their use in seismic structures.
4. Quantification of typical rotation demand on composite semi rigid connections.

ANSYS MATRIX27 ELEMENT

A.1. MATRIX27 ELEMENT DESCRIPTION

MATRIX27 represents an arbitrary element whose geometry is undefined but whose elastic kinematic response can be specified by stiffness, damping, or mass coefficients. The matrix is assumed to relate two nodes, each with six degrees of freedom per node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z axes. See [MATRIX27](#) in the *ANSYS, Inc. Theory Reference* for more details about this element. Other similar, but less general, elements are the spring-damper element ([COMBIN14](#)), and the mass element ([MASS21](#)).

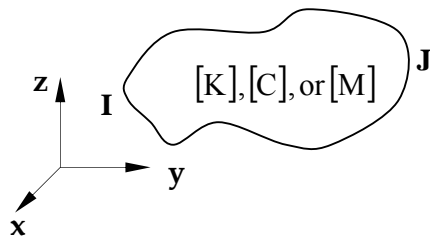


Figure A.1 MATRIX27 Schematic

A.2. MATRIX27 INPUT DATA

The node locations and the coordinate system for this element are shown in Fig. A.1: "MATRIX27 Schematic". The element is defined by two nodes and the matrix coefficients. The stiffness, damping, or mass matrix constants are input as real constants. The units of the stiffness constants are Force/Length or Force×Length/Radian and the damping constants, Force×Time/Length and Force×Length×Time/Radian. The mass constants should have units of Force×Time²/Length or Force×Time²×Length/Radian.

All matrices generated by this element are 12 by 12. The degrees of freedom are ordered as $U_x, U_y, U_z, ROT_x, ROT_y, ROT_z$ for node I followed by the same for node J. If one node is not used, simply let all rows and columns relating to that node default to zero.

The matrix constants should be input according to the matrix diagrams shown in "[MATRIX27 Output Data](#)". For example, if a simple spring of stiffness K in the nodal x direction is desired, the input constants would be $C_1 = C_{58} = K$ and $C_7 = -K$ for KEYOPT (2) = 0 and KEYOPT (3) = 4.

A summary of the element input is given below.

MATRIX27 Input Summary

Nodes

I, J

Degrees of Freedom

UX, UY, UZ, ROTX, ROTY, ROTZ

Real Constants

C1, C2, ... C78 - Define the upper triangular portion of the matrix

C79, C80, ... C144 - Define the lower triangular portion of an unsymmetrical matrix (required only if KEYOPT(2) = 1)

Material Properties

DAMP

Surface Loads

None

Body Loads

None

Special Features

Birth and death

KEYOPT (2)

Matrix formulation:

0 --

Symmetric matrices

1 --

Unsymmetric matrices

KEYOPT (3)

Real constant input data:

2 --

Defines a 12 x 12 mass matrix

4 --

Defines a 12 x 12 stiffness matrix

5 --

Defines a 12 x 12 damping matrix

KEYOPT (4)

Element matrix output:

0 --

Do not print element matrix

1 --

Print element matrix at beginning of solution phase

A.3. MATRIX27 OUTPUT DATA

The solution output associated with the element consists of node displacements included in the overall nodal solution. There is no element solution output associated with the element.

KEYOPT (4) = 1 causes the element matrix to be printed (for the first sub-step of the first load step only).

For KEYOPT(2) = 0, the symmetric matrix has the form:

$$\left[\begin{array}{cccccc|cccccc} C_1 & C_2 & C_3 & C_4 & C_5 & C_6 & C_7 & C_8 & C_9 & C_{10} & C_{11} & C_{12} \\ C_2 & C_{13} & C_{14} & C_{15} & - & - & - & - & - & - & C_{22} & C_{23} \\ C_3 & C_{14} & C_{24} & C_{25} & - & - & - & - & - & - & - & C_{33} \\ C_4 & C_{15} & C_{25} & C_{34} & - & - & - & - & - & - & - & C_{42} \\ C_5 & - & - & - & C_{43} & - & - & - & - & - & - & C_{50} \\ C_6 & - & - & - & - & C_{51} & - & - & - & - & - & C_{57} \\ \hline C_7 & - & - & - & - & - & C_{58} & - & - & - & - & C_{63} \\ C_8 & - & - & - & - & - & - & C_{64} & - & - & - & C_{68} \\ C_9 & - & - & - & - & - & - & - & C_{69} & - & - & C_{72} \\ C_{10} & - & - & - & - & - & - & - & - & C_{73} & - & C_{75} \\ C_{11} & - & - & - & - & - & - & - & - & - & C_{76} & C_{77} \\ C_{12} & C_{23} & C_{33} & C_{42} & C_{50} & C_{57} & C_{63} & C_{68} & C_{72} & C_{75} & C_{77} & C_{78} \end{array} \right]$$

For KEYOPT (2) = 1, the unsymmetric matrix has the form:

$$\left[\begin{array}{cccccc|cccccc} C_1 & C_2 & C_3 & C_4 & C_5 & C_6 & C_7 & C_8 & C_9 & C_{10} & C_{11} & C_{12} \\ C_{79} & C_{13} & C_{14} & - & - & - & - & - & - & - & C_{22} & C_{23} \\ C_{80} & C_{81} & C_{24} & - & - & - & - & - & - & - & - & C_{33} \\ C_{82} & C_{83} & C_{84} & C_{34} & - & - & - & - & - & - & - & C_{42} \\ C_{85} & C_{86} & - & C_{88} & C_{43} & - & - & - & - & - & - & C_{50} \\ C_{89} & - & - & - & C_{93} & C_{51} & - & - & - & - & - & C_{57} \\ \hline C_{94} & - & - & - & - & C_{99} & C_{58} & - & - & - & - & C_{63} \\ C_{100} & - & - & - & - & - & C_{106} & C_{64} & - & - & - & C_{68} \\ C_{107} & - & - & - & - & - & - & C_{14} & C_{69} & - & - & C_{72} \\ C_{115} & - & - & - & - & - & - & - & C_{123} & C_{73} & - & C_{75} \\ C_{124} & - & - & - & - & - & - & - & C_{132} & C_{133} & C_{76} & C_{77} \\ C_{134} & C_{135} & C_{136} & C_{137} & C_{138} & C_{139} & C_{140} & C_{141} & C_{142} & C_{143} & C_{144} & C_{78} \end{array} \right]$$

A.4. MATRIX27 ASSUMPTIONS AND RESTRICTIONS

- Nodes may be coincident or no coincident.
- Since element matrices should normally not be negative definite, a note is printed for those cases where this can be easily detected.
- With a lumped mass matrix all off-diagonal terms must be zero.
- The matrix terms are associated with the nodal degrees of freedom and are assumed to act in the nodal coordinate directions

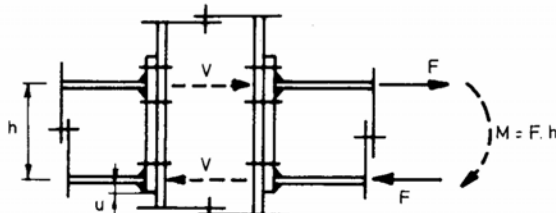
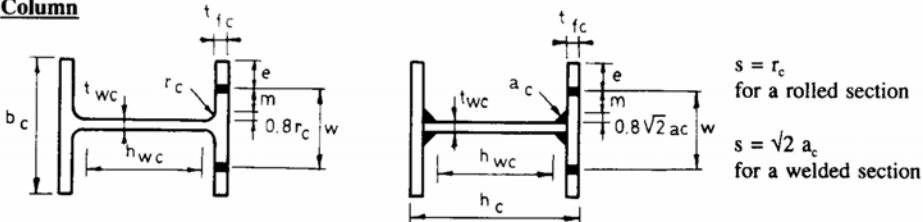
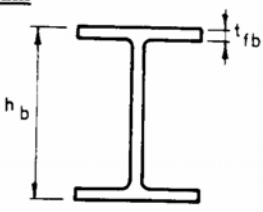
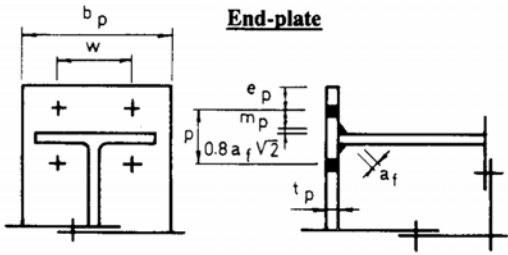
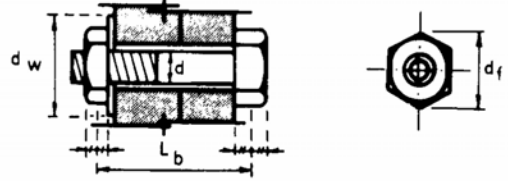
MATRIX27 Product Restrictions

When used in the product(s) listed below, the stated product-specific restrictions apply to this element in addition to the general assumptions and restrictions given in the previous section.

- Damping and unsymmetric matrices are not allowed.
- Real constants C79 through C144, for unsymmetrical matrices, are not applicable.
- The birth and death special feature is not allowed.
- KEYOPT (2) can only be set to 0 (default). KEYOPT (3) = 5 is not allowed.
- The DAMP material property is not allowed.

STIFFNESS COEFFICIENTS OF VARIOUS COMPONENTS OF THE END PLATE JOINT [1, 17]

Table B1

<i>Mechanical characteristics</i>		
	<u>Yield stresses</u>	<u>Ultimate stresses</u>
Beam web	f_{ywb}	-
Beam flange	f_{yfb}	-
Column web	f_{ywc}	-
Column flange	f_{yfc}	f_{ufc}
End-plate	f_{yp}	f_{up}
Bolts	-	f_{ub}
If hot-rolled profiles : $f_{ywc} = f_{yfc}$ and $f_{ywb} = f_{yfb}$		
<i>Geometrical characteristics</i>		
<p><u>Joint</u></p> 		
<p><u>Column</u></p>  <p> $s = r_c$ for a rolled section $s = \sqrt{2} a_c$ for a welded section </p> <p> $A_v = A_c - 2 b_c t_{fc} + (t_{wc} + 2 r_c) t_{fc}$ with $A_c =$ column section area $A_v = (h - 2 t_{fc}) t_{wc}$ </p>		
<p><u>Beam</u></p> 		
<p><u>End-plate</u></p> 		
<p><u>Bolts</u></p> <p>d_w : see figure or = d_f if no washer</p> <p>A_s : resistance area of the bolts</p> 		

STIFFNESS	RESISTANCE
------------------	-------------------

Column web panel in shear	$k_1 = \frac{0,385 A_v}{\xi h}$	$F_{Rd,1} = \frac{V_{wc,Rd}}{\xi} \quad \text{with} \quad V_{wc,Rd} = \frac{0,9 A_v f_{ycw}}{\sqrt{3} \gamma_{Mo}}$ <p> $\xi = 1$ for one-sided joint configurations; 0 for double sided joint configurations symmetrically loaded; 1 for double-sided configurations non-symmetrically loaded with balanced moments; 2 for double-sided joint configurations non-symmetrically loaded with unbalanced moments. For other values, see 1.2.2.1 in chapter 1. </p>
Column web in compression	$k_2 = \frac{0,7 b_{eff,wc,c} t_{wc}}{h_{wc}}$	$F_{Rd,2} = \eta \rho_c b_{eff,wc,c} t_{wc} f_{ycw} / \gamma_{Mo}$ <p> with $\eta = \min [1,0 ; 1,25 - 0,5 \frac{\sigma_n}{f_{ycw}}] (*)$ </p> $\rho_c = \sqrt{\frac{1}{1 + 1,3(\xi b_{eff,wc,c} t_{wc} / A_v)^2}}$ <p> σ_n : normal stresses in the column web at the root of the fillet radius </p> $b_{eff,wc,c} = t_{fb} + a_f \sqrt{2} + t_p + \min(u ; a_f \sqrt{2} + t_p) + 5(t_{fc} + s)$
Beam flange in compression	$k_3 = \infty$	$F_{Rd,3} = M_{c,Rd} / (h_b - t_{fb})$ <p> $M_{c,Rd}$: beam design moment resistance </p>
Bolts in tension	$k_4 = 3,2 \frac{A_s}{L_b}$	$F_{Rd,4} = 4 B_{t,Rd} \quad \text{with} \quad B_{t,Rd} = F_{t,Rd}$ $F_{t,Rd} = \frac{0,9 f_{ub} A_s}{\gamma_{Mb}}$
Column web in tension	$k_5 = \frac{0,7 b_{eff,wc,t} t_{wc}}{h_{wc}}$	$F_{Rd,5} = \rho_t b_{eff,wc,t} t_{wc} f_{ycw} / \gamma_{Mo}$ <p> with $\rho_t = \sqrt{\frac{1}{1 + 1,3(\xi b_{eff,wc,t} t_{wc} / A_v)^2}}$ </p> $b_{eff,wc,t} = \min [4\pi m ; 8m + 2,5e ; p + 4m + 1,25e]$

TableB2 (cont'd)

<p>Column flange in bending</p>	$k_6 = \frac{0,85 l_{eff,fc,t} t_{fc}^3}{m^3}$	$F_{Rd,6} = \min [F_{fc,Rd,1} ; F_{fc,Rd,2}]$ $F_{fc,Rd,1} = \frac{(8n - 2e_w) l_{eff,fc,t} m_{pl,fc}}{2mn - e_w(m+n)}$ $F_{fc,Rd,2} = \frac{2 l_{eff,fc,t} m_{pl,fc} + 4 B_{t,Rd} n}{m + n}$ $n = \min [e ; 1,25m ; (b_p - w)/2]$ $e_w = d_w / 4$ $m_{pl,fc} = 0,25 t_{fc}^2 f_{yc} / \gamma_{Mo}$ $l_{eff,fc,t} = b_{eff,wc,t}$
<p>End-plate in bending</p>	$k_7 = \frac{0,85 l_{eff,p,t} t_p^3}{m_p^3}$	$F_{Rd,7} = \min [F_{ep,Rd,1} ; F_{ep,Rd,2}]$ $F_{ep,Rd,1} = \frac{(8n_p - 2e_w) l_{eff,p,t} m_{pl,p}}{2m_p n_p - e_w(m_p + n_p)}$ $F_{ep,Rd,2} = \frac{2 l_{eff,p,t} m_{pl,p} + 4 B_{t,Rd} n_p}{m_p + n_p}$ $n_p = \min [e_p ; 1,25m_p]$ $m_{pl,p} = 0,25 t_p^2 f_{yp} / \gamma_{Mo}$ $l_{eff,p,t} = \min [4\pi m_p ; 8m_p + 2,5e_p ; w + 4m_p + 1,25e_p ; b_p]$
<p>JOINT</p>	<p>Initial stiffness :</p> $S_{joint,init.} = E h^2 / \sum_{i=1}^7 1/k_i$ <p>Nominal stiffness :</p> $S_{joint} = S_{joint,init.} / 2$	$F_{Rd} = \min [F_{Rd,i}]$ <p>Plastic design moment resistance :</p> $M_{Rd} = F_{Rd} h$ <p>Elastic moment resistance :</p> $\frac{2}{3} M_{Rd}$

APPENDIX C

REAL CONSTANTS / STIFFNESS COEFFICIENTS/ FOR THE JOINTS

J1

$$\begin{bmatrix} 31665 & 0 & 147997.4 & -31665 & 0 & -147997 \\ 0 & 4852 & 0 & 0 & -4852 & 0 \\ 147997.4 & 0 & 2484754 & -147997 & 0 & -2484754 \\ -31665 & 0 & -147997 & 31665 & 0 & 147997.4 \\ 0 & -4852 & 0 & 0 & 4852 & 0 \\ -147997 & 0 & -2484754 & 147997.4 & 0 & 2484754 \end{bmatrix}$$

J2

$$\begin{bmatrix} 30180 & 0 & 127085.5 & -30180 & 0 & -127086 \\ 0 & 5583 & 0 & 0 & -5583 & 0 \\ 127085.5 & 0 & 2662448 & -127086 & 0 & -2662448 \\ -30180 & 0 & -127086 & 30180 & 0 & 127085.5 \\ 0 & -5583 & 0 & 0 & 5583 & 0 \\ -127086 & 0 & -2662448 & 127085.5 & 0 & 2662448 \end{bmatrix}$$

J3

$$\begin{bmatrix} 39420 & 0 & 188726 & -39420 & 0 & -188726 \\ 0 & 8432 & 0 & 0 & -8432 & 0 \\ 188726 & 0 & 1543385 & -188726 & 0 & -1543385 \\ -39420 & 0 & -188726 & 39420 & 0 & 188726 \\ 0 & -8432 & 0 & 0 & 8432 & 0 \\ -188726 & 0 & -1543385 & 188726 & 0 & 1543385 \end{bmatrix}$$

J4

30625	0	101504.3	-30625	0	-101504
0	7330	0	0	-7330	0
101504.3	0	1367888	-101504	0	-1367888
-30625	0	-101504	30625	0	101504.3
0	-7330	0	0	7330	0
-101504	0	-1367888	101504.3	0	1367888

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CANDIDATE’S DECLARATION

I hereby declare that the work which is being presented in this thesis entitles **“Behavior and Modeling of Semi-rigid Steel Beam-to-column Connections”** is original work of my own, has not been presented for a degree in any other university and that all sources of material used for the thesis have been duly acknowledged.

Temesgen Wondimu
(Candidate)

Date

This is so certify that the above declaration made by the candidates is correct to the best of my knowledge.

Dr. Shifferaw Taye
(Thesis Advisor)

Date