Design of non-linear semi-rigid steel frames with semi-rigid column bases

S. O. Degertekin
Civil Engineering Department, Dicle University, Muhendislik-Mimarlik Fakultesi
21280 Diyarbakir, Turkey

M. S. Hayalioglu
Civil Engineering Department, Dicle University, Muhendislik-Mimarlik Fakultesi
21280 Diyarbakir, Turkey

ABSTRACT

This article presents an analysis and design method for steel frames with semi-rigid connections and semi-rigid column bases. The analysis takes into account both the non-linear behaviour of beam-to-column connections and P-Δ effects of beam-column members. The Frye and Morris polynomial model is used for modelling of semi-rigid connections. The members are designed according to the specifications of American Institute of Steel Construction (AISC) Allowable Stress Design (ASD). The design process is interactive, and gives choices to the designer, to change member cross-sections and connection parameters for economical and practical reasons, interacting with computer. Two design examples with various type of connections are presented to demonstrate the efficiency of the method. The semi-rigid connection modelling yields more economical solutions than rigid connection modelling.The semi-rigid column base modelling also results in lighter frames. It is also shown that changes in the stiffness of the connections may result in economical solutions and alteration in the sways of the frames.

KEYWORDS
Allowable stress design; Non-linear analysis; Semi-rigid connections; Steel design; Unbraced frame;Semi-rigid column base

1 Introduction

Beam-to-column connections are assumed either perfectly pinned or fully rigid in most design of steel frames. This simplification leads to an incorrect estimation of frame behaviour. In fact, the connections are between the two extreme assumptions and possess some rotational stiffness. Full scale testing requires so as explaining the real behaviour of these connections. Bolted and welded beam-to-column connections rotate at an angle due to applied bending moment. This connection deformation has negative effect on 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) effects of beam-column members and thus will affect the overall stability of the frame. Hence, the non-linear features of beam-to-column connections have important function in structural steel design. As a result of experimental works done by several researchers, various semi-rigid connection modelling and their moment-rotation relationships are proposed. The main of these are linear, polynomial, cubic B spline, power and exponential models [1]. Some important research works have been reported for the analysis and design of semi-rigid frames [1-4].

AISC-ASD specification [5] describes three types of steel construction: rigid-frame, simple framing (unrestrained) and semi-rigid framing (partially restrained). This specification requires that the connections of the type of partially restrained construction have a flexibility intermediate in degree between the rigidity of Type 1 and the flexibility of Type 2, and this type of construction may necessitate non-elastic (non-linear) deformations of structural steel parts. On the other hand, Eurocode 3 [6] proposes three types connection: rigid; semi-rigid and normally pinned or flexible. Eurocode 3 gives clear demarcation lines with exact values among these types.

The aim of the present study is also to consider both semi-rigid beam-to-column connections and semi-rigid column bases in the design of steel frames according to the specifications of AISC-ASD and thus to account the non-linear behaviour due to connection characteristics and P-Δ effects of beam-column members. A polynomial model proposed by Frye and Morris [7] is adopted as semi-rigid connection model. In the present study, a computer-based analysis and design method is developed which is interactive in character, and allows the designer to change member sizes and connection parameters to search satisfactory designs. The effect of changes in connection stiffness on the design results is investigated. The design results of frames with semi-rigid column bases are also compared with those of frames with rigid bases.

2 Connection modelling

A connection rotates through angle θ_r caused by applied moment M. This is the angle between beam and column from their original position. Several moment-rotation relationships have been derived from experimental studies for modelling semi-rigid connections of steel frames. These relationships vary from linear model to exponential models and are non-linear in nature. Relative moment-rotation curves of extensively used semi-rigid connections are shown in Fig.1 [8]. The geometry and size parameters of six types of connections are shown in Fig.2 [8]. In the present work, a polynomial model offered by Frye and Morris [7] is used because of its easy application. This model is expressed by an odd power polynomial which is in the following form:

$$\theta_r = c_1(\kappa M)^1 + c_2(\kappa M)^3 + c_3(\kappa M)^5 \qquad (1)$$

where κ is standardization constant depends upon connection type and geometry; c_1, c_2, c_3 are the curve fitting constants. The values of these constants are given in Table 1 [9].

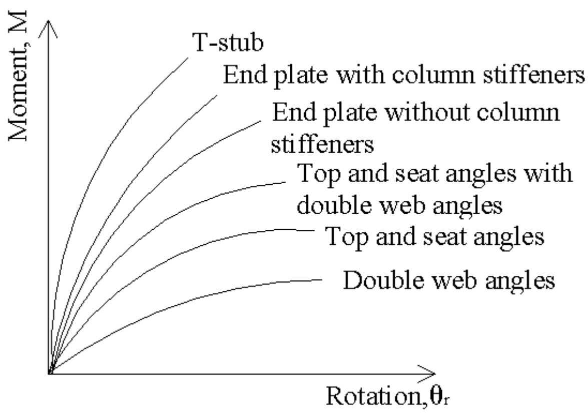

Fig. 1 - Moment-rotation curves of semi-rigid connections

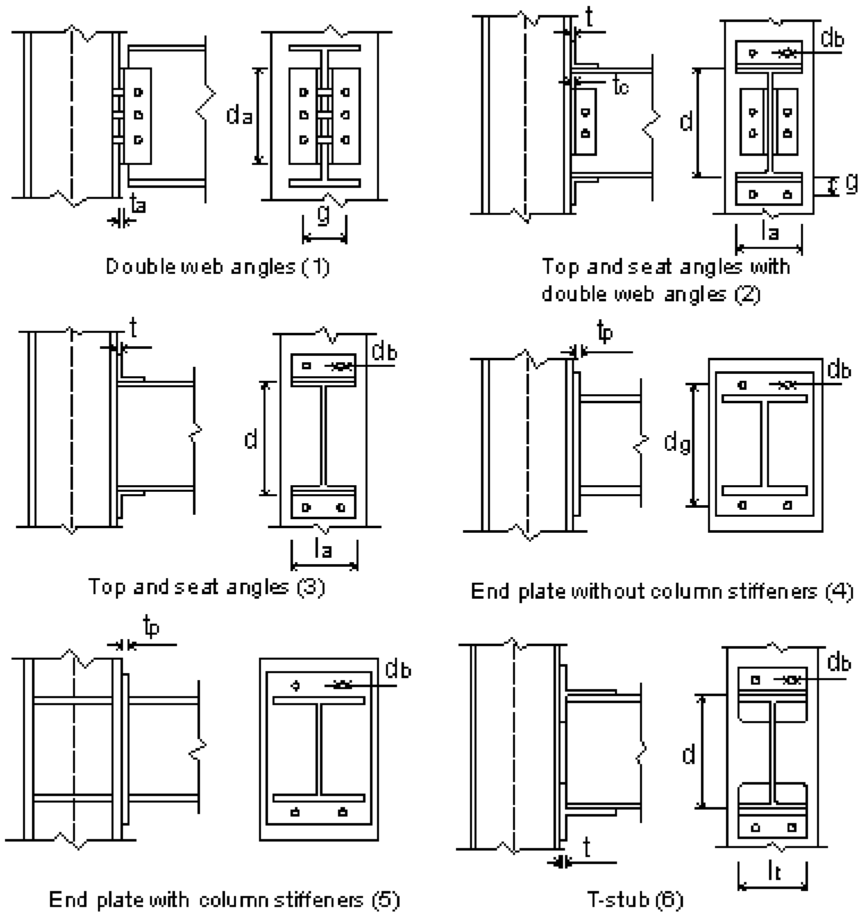

Fig. 2 - Semi-rigid connection types and size parameters (type-numbers are given in brackets)

Table 1 - Curve fitting constants and standardization constants for the Frye-Morris polynomial model

Connection types	Curve fitting constants	Standardization constants
1	$c_1 = 3.66\times10^{-4}$ $c_2 = 1.15\times10^{-6}$ $c_3 = 4.57\times10^{-8}$	$\kappa = d_a^{-2.4} t_a^{-1.81} g^{0.15}$
2	$c_1 = 2.23\times10^{-5}$ $c_2 = 1.85\times10^{-8}$ $c_3 = 3.19\times10^{-12}$	$\kappa = d^{-1.287} t^{-1.128} t_c^{-0.415} l_a^{-0.694} g^{1.35}$
3	$c_1 = 8.46\times10^{-4}$ $c_2 = 1.01\times10^{-4}$ $c_3 = 1.24\times10^{-8}$	$\kappa = d^{-1.5} t^{-0.5} l_a^{-0.7} d_b^{-1.5}$
4	$c_1 = 1.83\times10^{-3}$ $c_2 = 1.04\times10^{-4}$ $c_3 = 6.38\times10^{-6}$	$\kappa = d_g^{-2.4} t_p^{-0.4} d_b^{-1.5}$
5	$c_1 = 1.79\times10^{-3}$ $c_2 = 1.76\times10^{-4}$ $c_3 = 2.04\times10^{-4}$	$\kappa = d_g^{-2.4} t_p^{-0.6}$
6	$c_1 = 2.10\times10^{-4}$ $c_2 = 6.20\times10^{-6}$ $c_3 = -7.60\times10^{-9}$	$\kappa = d^{-1.5} t^{-0.5} l_t^{-0.7} d_b^{-1.1}$

3 Analysis of steel frames with semi-rigid connections

The design procedure requires that the displacements and stresses in the frame system be known. This is achieved through a non-linear analysis of the steel frame. The non-linear analysis of steel frames takes into account both the geometrical non-linearity of beam-column members and non-linearity due to end connection flexibility of beam members. The columns of frames are generally continuous and do not have any internal flexible connections. However, the beams possess semi-rigid end connections, but have small axial forces with a geometric non-linearity of little importance. In the present study, three types of members are adopted for easiness in the design of steel frames with semi-rigid connections:

1. Beam-column member: A plane-frame member modified to include geometric non-linearity effect (P-Δ effect).

2. Beam member with semi-rigid end connections: A plane-frame member modified to incorporate end connection flexibility.

3. Beam-column member with semi-rigid column base: A bottom storey column modified to include semi-rigid base.

End forces and end displacements of a plane-frame member in member (local) coordinates are shown in Fig.3.

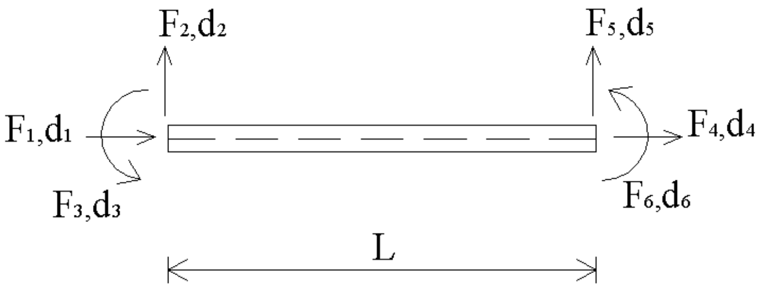

Fig. 3 - A plane-frame member with end forces and displacements

3.1 Beam-column member

The stiffness matrix of a beam-column member i in member (local) coordinates incorporating P-Δ effect can be expressed as follow:

$$[\bar{k}]_i = [k_E]_i + [k_p]_i \tag{2}$$

where $[k_E]_i$ is conventional linear-elastic stiffness matrix and $[k_p]_i$ is ‘geometrical stiffness matrix’ given as [10]

$$[k_p]_i = \frac{P}{L}\begin{bmatrix} 0 & & & & & \\ 0 & \frac{6}{5} & & & & \\ 0 & \frac{L}{10} & \frac{2L^2}{15} & & & \\ 0 & 0 & 0 & 0 & & \\ 0 & \frac{-6}{5} & -\frac{L}{10} & 0 & \frac{6}{5} & \\ 0 & \frac{L}{10} & \frac{-L^2}{30} & 0 & \frac{-L}{10} & \frac{2L^2}{15} \end{bmatrix} \quad (3)$$

where L is member length and P is the axial force in the member.

3.2 Beam member with semi-rigid end connections

Semi-rigid end connections of a beam can be represented by rotational springs as shown in Fig.4. θ_{rA} and θ_{rB} are the relative spring rotations of both ends and k_A and k_B are the corresponding spring stiffness expressed as:

$$k_A = \frac{M_A}{\theta_{rA}} \quad (4)$$

$$k_B = \frac{M_B}{\theta_{rB}} \quad (5)$$

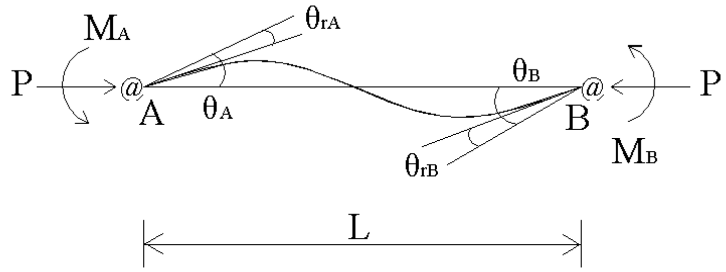

Fig. 4 - Beam member with rotational springs

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 (θ_A-θ_{rA}) and (θ_B-θ_{rB}) respectively, as follows:

$$M_A = \frac{EI}{L}\left[4\left(\theta_A - \frac{M_A}{k_A}\right) + 2\left(\theta_B - \frac{M_B}{k_B}\right)\right] \quad (6a)$$

$$M_B = \frac{EI}{L}\left[4\left(\theta_B - \frac{M_B}{k_B}\right) + 2\left(\theta_A - \frac{M_A}{k_A}\right)\right] \quad (6b)$$

where E is the modulus of elasticity and I is moment of inertia of the member. The Eqns. (6a) and (6b) can be expressed in the following form:

$$M_A = \frac{EI}{L}\left(r_{ii}\theta_A + r_{ij}\theta_B\right) \quad (7a)$$

$$M_B = \frac{EI}{L}\left(r_{ij}\theta_A + r_{jj}\theta_B\right) \tag{7b}$$

$$r_{ii} = \frac{1}{k_R}\left(4 + \frac{12EI}{Lk_B}\right) \tag{8a}$$

$$r_{jj} = \frac{1}{k_R}\left(4 + \frac{12EI}{Lk_A}\right) \tag{8b}$$

$$r_{ij} = \frac{2}{k_R} \tag{8c}$$

$$k_R = \left(1 + \frac{4EI}{Lk_A}\right)\left(1 + \frac{4EI}{Lk_B}\right) - \left(\frac{EI}{L}\right)^2\left(\frac{4}{k_A k_B}\right) \tag{8d}$$

Eqns. (7) are converted to the following stiffness matrix of a semi-rigid beam member with 6 degrees of freedom in local coordinates [11].

$$[k]_i = \begin{bmatrix} \frac{AE}{L} & & & & & \\ 0 & (r_{ii}+2r_{ij}+r_{jj})\frac{EI}{L^3} & & & & \\ 0 & (r_{ii}+r_{ij})\frac{EI}{L^2} & r_{ii}\frac{EI}{L} & & & \\ \frac{-AE}{L} & 0 & 0 & \frac{AE}{L} & & \\ 0 & -(r_{ii}+2r_{ij}+r_{jj})\frac{EI}{L^3} & -(r_{ii}+r_{ij})\frac{EI}{L^2} & 0 & (r_{ii}+2r_{ij}+r_{jj})\frac{EI}{L^3} & \\ 0 & (r_{ij}+r_{jj})\frac{EI}{L^2} & r_{ij}\frac{EI}{L} & 0 & -(r_{ij}+r_{jj})\frac{EI}{L^2} & r_{jj}\frac{EI}{L} \end{bmatrix} \tag{9}$$

where A is cross-sectional area of the member. Applying the known steps of the matrix displacement method, this matrix is obtained in global or structure coordinates for each member and structure stiffness matrix is constituted. The relationships between end-forces and end-displacements are also constructed according to the method. In the present work fixed-end forces which are derived in [2] are used for the beam members with semi-rigid end connections.

3.3 Beam-column member with semi-rigid column base

A column base with four bolt connection arrangements has been adopted as shown in Fig.5. The rotational stiffness of semi-rigid column base is given by Hensman and Nethercot [12] as

$$k_{base} = \frac{Ez^2 t}{20} \tag{10}$$

in which

$$z = r_b + \frac{H_c}{2} - \frac{t_f}{2} \tag{11}$$

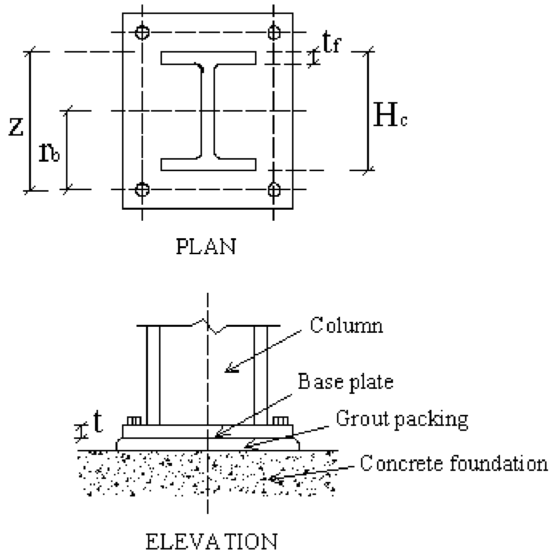

Fig. 5 - Simple semi-rigid column base detail

A linear spring model is used at the column feet; the rotations developed at the column base under serviceability load levels are assumed sufficiently small for use of the initial linear response to be sensible. The stiffness matrix of a beam-column member with semi-rigid column base is obtained by adding the matrices in Eqn. (2) and Eqn. (9) and replacing zero and k_{base} instead of $1/k_B$ and k_A respectively in the elements of the final matrix (the A end of the column is assumed to be column base).

3.4 Analysis procedure

The structure stiffness matrix is constructed by superimposing the member stiffness matrices contain geometric non-linearity and connection flexibility effects. This matrix is substituted in the structural equilibrium equations, which are non-linear and necessitate an iterative solution procedure. The applied loads are divided into a number of small-load increments and structural equilibrium equations are written in the incremental form:

$$[S]\{\Delta D\} = \{\Delta F\} \tag{12}$$

where $[S]$ is structure stiffness matrix, $\{\Delta F\}$ is incremental load vector, and $\{\Delta D\}$ is incremental displacement vector. The incremental Eqns.(12) are iteratively solved by a sequence of linear steps. The secant stiffness approach [10] is utilized for calculating the connection stiffness. The connection secant stiffness, SE, is defined as:

$$SE = \frac{\Delta M}{\Delta \theta_r} \tag{13}$$

where ΔM is the change in end moment during a load increment, $\Delta \theta_r$ is the change in relative spring rotation during the load increment. For each load increment, structure stiffness matrix is formed at the start of each iterative cycle. This requires calculation of the connection secant stiffness at the beginning of each cycle, and changing of the latest geometry and member end forces based on information from previous cycle. The convergent connection secant stiffness related to all load increments are shown in Fig.6. Convergence is obtained when the difference between joint displacements of two consecutive cycles falls below a specified tolerance.

A convergent solution of a load increment forms initial values for the next iteration and the iterative procedure goes on until all load increments are taken into account. The solutions for all load increments are added up to acquire a total non-linear response.

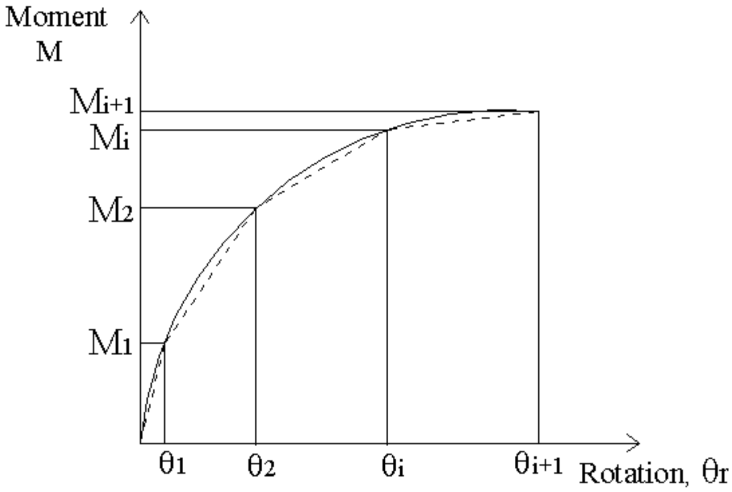

Fig. 6 - Connection secant stiffness through load increments

The above mentioned analysis procedure can be summarized through the following steps:

1. Divide applied loads into a series of small increments.

2. Carry out the linear analysis under first load increment and obtain the response of the frame, which is an initial estimate for the non-linear analysis.

3. Set up the member stiffness matrices $[k]_i$ and $[\bar{k}]_i$ for all members and assemble them in structure stiffness matrix $[S]$.

4. Solve the Eq. (12) for $\{\Delta D\}$ and then determine the incremental member end forces.

5. Obtain the connection secant stiffness by Eqn. (13).

6. Update the terms in the member stiffness matrices using the latest connection secant stiffness, and member forces. Update also structure geometry.

7. Repeat steps 3-6 until convergence is attained.

8. Calculate accumulated displacements and member end forces at convergence.

9. Continue the analysis with new load increments until all load increments are considered.

4 Design requirements

The interaction equations for the members of a steel frame under bending and axial stresses are of the form [5]:

For members subjected to both axial compression and bending stresses:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left(1 - \frac{f_a}{F'_{ex}}\right) F_{bx}} \leq 1.0 \qquad (14)$$

$$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} \leq 1.0 \quad (15)$$

When $f_a / F_a \leq 0.15$, Eq. (16) is permitted in lieu of Eqs. (14) and (15).

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \leq 1.0 \quad (16)$$

For members subjected to both axial tension and bending stresses:

$$\frac{f_a}{F_t} + \frac{f_{bx}}{F_{bx}} \leq 1.0 \quad (17)$$

In Eqs. (14)-(17), the subscript x, combined with subscripts b, m and e indicates the axis of bending about which a particular stress or design property applies. In Eqs. (14)-(16), F_a is axial compressive stress permitted in the existence of axial force alone, F_{bx} is compressive bending stress permitted in the existence of bending moment alone, F'_{ex} is Euler stress divided by a factor of safety, f_a is computed axial compressive stress, f_{bx} is computed compressive bending stress at the point under consideration, C_{mx} is a coefficient whose value is taken as 0.85 for compression members in unbraced frames, F_y is the yield stress of steel. In Eq. (17), f_a is the computed axial tensile stress, f_{bx} is computed bending tensile stress, F_{bx} is allowable bending stress which is equal to $0.66F_y$ and F_t is the governing allowable tensile stress. $F_a, F_{bx}, 0.60F_y, F'_{ex}$ and F_t are increased 1/3 in accordance with the specification when produced by wind or earthquake acting alone or in combination with the design dead and live loads. Definitions of the permitted and Euler stresses and other details are given in AISC-ASD specifications [5]

The computed stresses are determined from non-linear analysis of steel frames under dead and live loads in combination with wind or earthquake loads.

4.1 Effective column-length factor

Effective length factor (K-factor) of columns must be estimated to evaluate the stability of columns in frames with rigid and semi-rigid connections. The factor K is required to determine the permitted axial compressive stress F_a and Euler stress F'_{ex} in the design of frame members. The effective length factor K for the columns in an unbraced frame is determined from the following interaction equation [13].

$$\frac{G_A G_B (\pi/K)^2 - 36}{6(G_A + G_B)} = \frac{\pi/K}{\tan(\pi/K)} \quad (18)$$

where G_A and G_B are relative stiffness factors for A-th and B-th ends of columns and given as:

$$G = \frac{\sum I_c / L_c}{\sum I_g / L_g} \quad (19)$$

where the summation is taken over all members connected to the joint, and where I_c is moment of inertia of column section corresponding to plane of buckling, L_c is unbraced length of column, I_g is moment of inertia of beam/girder corresponding to plane of bending, and L_g is unbraced length of beam/girder .

In Eq.(18), it is assumed that the beams and girders are rigidly connected to columns at the joints. The beam/girder stiffness I_g/L_g in Eq.(19) is multiplied by the following factors to consider for different end connections:

The factor is 0.5 for far ends fixed; 0.67 for pinned, and $1/(1+6EI/L\times k)$ for flexibly connected, where k is spring stiffness of corresponding end.

5 Design procedure

Interacting with the computer, a design engineer can select member size founded on the value of interaction ratio given by Eqns. (14)-(17) compared with 1. An interaction ratio value greater than 1 implies that the member is insufficient and a larger section should be selected. Interaction ratios value smaller than 0.9 gives the implication that the design may be improved by selecting a reduced section. The engineer can also select and change interactively connection type and its size parameters to obtain adequate designs. The iterative and interactive design goes on until the engineer is convinced of his member and connection parameter selection.

The steps of the design of steel frames with semi-rigid connections are given in the following.

1. Assign the initial sections to the members of the frame from a specified list of standard sections and carry out the non-linear analysis of the frame under the applied loads by considering for non-linear behaviour of the semi-rigid connections and P-Δ effect.

2. Compute the member stresses using the member forces obtained from the non-linear analysis.

3. Check all members to satisfy the design requirements in Eqs.(14)-(17).

4. If the design is not satisfactory for any member, change the member size from the list for the insufficient or oversized member. Meanwhile, try various connection size parameters to achieve economic designs and control frame sway.

5. Repeat the procedure until satisfactory design is obtained.

6 Design examples

A computer program has been developed in the present study, which is implementation of the design procedure. Two design examples are presented to demonstrate the application of the design algorithm. The designs of semi-rigid frames are compared to the designs of rigid frames under the same design requirements. The designs of rigid frames are performed considering P-Δ effect of beam-column members. The material is grade A36 steel with a modulus of elasticity of 200000 MPa and yield stress of 248.2 MPa. Material density is 7850 kg/m^3. AISC (W) shapes are used as steel sections in all design examples considered in the present study. The numbers of semi-rigid connection types used in the designs are the same as the ones given in Fig.2.

6.1 Three-storey, two-bay frame

The dimensions, loading and numbering of members of three-storey two-bay frame are shown in Fig.7. The connection size parameters, which remain fixed during the design process, are given in Table 2 depending on the connection types.

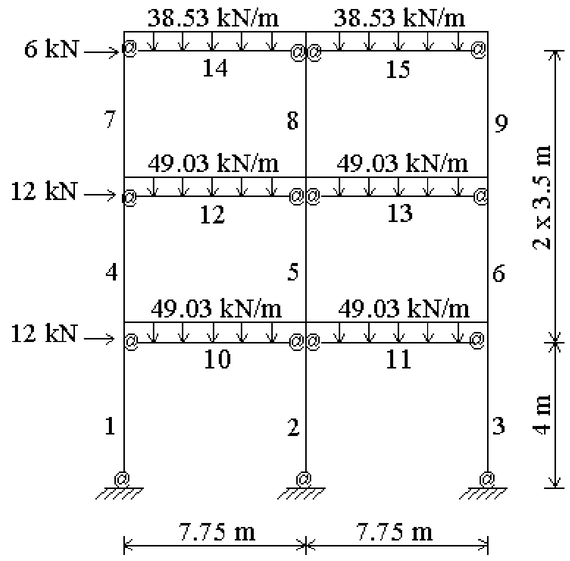

Fig. 7 - Three-storey, two-bay frame

Table 2 - The fixed connection size parameters for three-storey, two-bay frame

Connection Type	**Connection size parameters (cm)**
1	t_a =**2.54** g =**22.86**
2	t =**2.54** t_c =**2.54** g =**11.43**
3	t =**2.54** d_b =**2.54**
4	t_p =**2.54** d_b =**2.54**
5	t_p =**2.54** d_b =**2.54**
6	t =**2.223** d_b =**2.54**

Table 3 - Final design results of 3-storey, two-bay frame. (Frame weights and sways)

Semi-rigid connection types	Weight (kg)		Top storey sway (cm)	
	Semi-rigid connection	Rigid connection	Semi-rigid connection	Rigid connection
1	6797	6196	1.13	0.71
2	5728		0.79	
3	7413		1.16	
4	5976		1.00	
5	5976		1.00	
6	5778		0.83	

The results of the final designs for six types of semi-rigid connections and also rigid connection are given in Table 3 in the form of frame weight and top storey sway. 3.5-7.6 % lighter frames with the connection types of 2,4,5,6 and 9.7-19.7% heavier frames with the connection types of 1,3 are obtained when compared to rigidly connected frame. Top storey sway of semi-rigid frames increase by 12-64% over the sway of the rigidly connected frame. The final design sections and their maximum interaction ratio values for the six types of connection are presented in Table 4 and Table 5 respectively.

Table 4 - Final design sections for 3-storey, two-bay frame

Member no	Semi-rigid connection type						Rigid connection
	(1)	(2)	(3)	(4)	(5)	(6)	
1,4,7	W12×19	W12×19	W12×22	W12×19	W12×19	W12×22	W12×40
2,5,8	W12×40	W12×40	W12×40	W12×40	W12×40	W12×40	W12×40
3,6,9	W12×19	W12×26	W12×19	W12×22	W12×22	W12×26	W12×40
10-13	W12×87	W12×65	W12×96	W12×72	W12×72	W12×65	W12×65
14,15	W12×65	W12×58	W12×72	W12×58	W12×58	W12×58	W12×53

Table 5 - Maximum interaction ratio values for 3-storey, two-bay frame

Member no	Semi-rigid connection type						Rigid connection
	(1)	(2)	(3)	(4)	(5)	(6)	
1,4,7	1.000	0.991	0.909	0.932	0.932	0.936	0.883
2,5,8	0.905	0.903	0.896	0.910	0.911	0.906	0.907
3,6,9	1.000	0.992	0.986	0.974	0.980	0.978	0.930
10-13	0.974	0.978	0.940	1.000	1.000	0.892	0.926
14,15	0.981	0.936	0.860	0.993	0.987	0.865	0.998

For the final design of frame with connection type 2, the positive span-moments of the beam members are presented in Table 6 while the absolute maximum end-moments of the members are given in Table 7 . The results of Table 6 and Table 7 show that, in the frame with semi-rigid connections, the absolute maximum end-moments of beams decrease while the span moments of beams increase when compared to those of rigid frame. However, in the semi-rigid frame the overall maximum moments decrease in columns while they increase in small amount in most of the beams when compared to those of rigid frame.

Table 6 - Span moments in the beams of 3-storey, two-bay frame

Member no.	Semi-rigid connection Moment (kN-m)	Rigid connection Moment (kN-m)
10	282.42	142.95
11	277.18	138.09
12	278.86	135.55
13	275.40	133.89
14	223.80	115.25
15	220.57	114.23

Table 7 - Absolute maximum end-moments in 3-storey two-bay frame

Member no.	Semi-rigid connection Moment (kN-m)	Rigid connection Moment (kN-m)
1	11.08	60.40
2	27.70	25.72
3	42.28	88.55
4	32.99	106.94
5	17.16	16.30
6	47.35	118.63
7	39.31	131.48
8	4.96	6.80
9	50.10	139.28
10	130.04	288.15
11	92.64	253.79
12	116.38	267.66
13	98.22	247.02
14	92.76	219.83
15	87.80	213.03

6.2 Ten-storey, single-bay frame

Fig.8 shows configuration, dimensions, loading, and numbering of members. The fixed connection size parameters are presented in Table 8. The results of the final designs for six types of semi-rigid connections together with rigid connection are presented in Table 9 in the shape of weight and the sway of the top storey. 1.4-4.3 % lighter frames with the connection types of 1,2,4,5,6 are obtained in comparison with the rigidly connected frame. Semi-rigid frame drifts increase by 46-86 % over the rigid frame's sway.

The final design sections and their maximum interaction ratio values for the frame with type 4 connections together with the values for the rigid frame are given in Table 10.

Table 8 - The fixed connection size parameters for ten-storey, single-bay frame

Connection Type	Connection size parameters (cm)
1	t_a =**2.22** g =**15.24**
2	t =**1.91** t_c =**1.91** g =**6.35**
3	t =**2.54** d_b =**2.54**
4	t_p =**1.75** d_b =**2.54**
5	t_p =**1.75** d_b =**2.54**
6	t =**1.75** d_b =**2.22**

Table 9 - Final design results of ten-storey, single-bay frame. (Frame weights and sways)

Semi-rigid connection types	Weight (kg)		Top storey sway (cm)	
	Semi-rigid connection	Rigid connection	Semi-rigid connection	Rigid connection
1	11202	11542	13.46	9.12
2	11048		13.28	
3	12621		17.01	
4	11202		13.92	
5	11377		13.92	
6	11213		15.04	

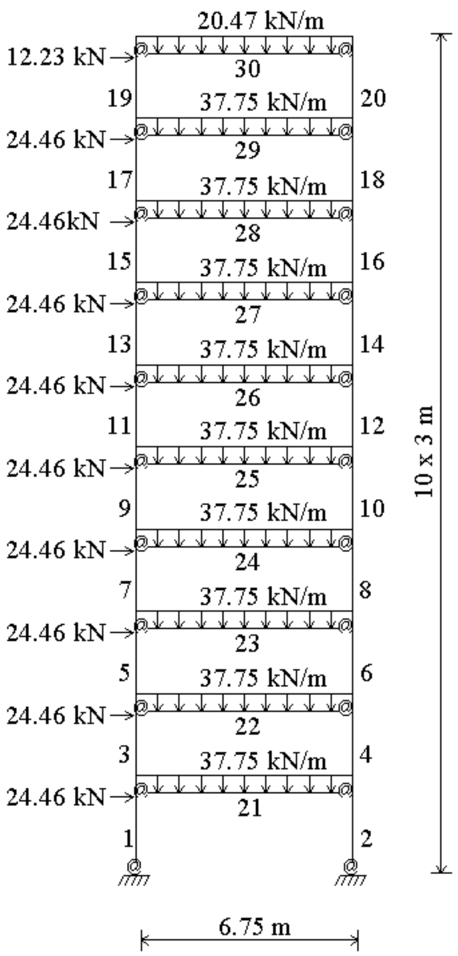

Fig. 8 - Ten-storey, single-bay frame

To examine the effect of the connection stiffness on the design of frames, the same frame with connection type 4 is designed with various connection size parameters and the results are presented in Table 11 in the form of frame weights and sways. The results of Table 11 indicate that, reducing of connection stiffness causes increase in both frame weight and sway.

The semi-rigid frames with rigid column bases are also designed and results are given in Table 12. According to these results the weights of frames with semi-rigid column bases decrease by 0.7-6.8% over the weights of frames with rigid bases depending on connection types. The semi-rigid column bases also cause increase in the sways by 0.3-23%.

Table 10 - Final design sections and their maximum interaction ratios for ten-storey, single-bay frame (for connection type 4)

Member no.	Semi-rigid connection		Rigid connection	
	Section	The ratio	Section	The ratio
1-4	W14×99	0.956	W14×99	1.000
5-8	W14×74	0.954	W14×82	0.920
9-12	W12×58	0.923	W12×65	0.950
13-16	W14×34	0.940	W14×43	0.958
17-20	W12×19	0.892	W12×26	0.999
21-23	W21×57	0.875	W21×68	0.912
24-26	W18×65	1.000	W18×60	0.961
27-29	W18×60	0.987	W18×50	0.978
30	W18×65	0.917	W18×55	0.986

Table 11. The effect of connection stiffness on the design of ten-storey, single-bay frame

Connection size parameters (cm)	Weight (kg)	Top storey sway (Cm)
t_p =2.540	11202	13.42
t_p =1.746	12172	13.97
t_p =1.588	12258	15.74
t_p =1.270	12293	19.00
t_p =0.792	12809	26.23

Table 12 -Design results of ten-storey, single-bay frame for rigid column bases

Semi-rigid connection types	Weight (kg)	Top storey sway (cm)
1	11379	12.62
2	11225	12.43
3	12711	16.96
4	12024	12.32
5	12028	12.68
6	11717	12.25

7 Discussion and conclusions

A combined analysis and design procedure is presented for the design of steel frames with semi-rigid connections and semi-rigid column bases accounting non-linear behaviour of frames. Computer-based analysis and design procedure is interactive and iterative in nature. Design examples are included to demonstrate the influence of connection flexibility and geometric non-linearity on the design of steel frames.

It is observed from the results of design examples that semi-rigid connection modelling may create lighter frames providing that appropriate connection size parameters are selected. The reduction in weight is calculated as 7.6% at most in the examples considered. The end moments of a beam with semi-rigid connections decrease while its span moment increases in comparison with those of the beam with rigid connections.

Connection stiffness plays important role in the design of semi-rigid frames. The semi-rigid connections cause a large increase in frame sway over the rigid connections. This increase is found to be up to 86% in the examples presented. Trying various connection stiffness values, the sway can be controlled and economic frames can be obtained. Reducing of connection stiffness results in increase in frame weight and sway. The reason for the increase in weights is that an increase in frame displacements magnifies column and beam end moments and thus larger sections are assigned to the members. The softening of connections results in considerable increase in frame sway. Examining the results of Table 11, it is found that the fifth trial for reducing the stiffness results in an increase in the sway by 95% over the first trial. An economic frame system can be attained by controlling frame sway with the stiffness of the connections.

In semi-rigid frames, semi-rigid column bases create lighter frames but they increase the sways when compared to the rigid base.

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