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# DESIGN OF SEMI-RIGID PLANAR STEEL FRAMES ACCORDING TO TURKISH STEEL DESIGN CODE

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**YARI-RİJİT DÜZLEMSEL ÇELİK ÇERÇEVELERİN TÜRK ÇELİK TASARIM YÖNETMELİĞİNE GÖRE TASARIMI**

**ÖZET**

Bu makale, yarı-rijit birleşimli çelik çerçeveler için bir analiz ve tasarım yöntemi sunmaktadır. Analiz, kiriş-kolon birleşimlerinin lineer olmayan davranışını ve kiriş-kolon elemanlarının P-Δ etkilerini göz önüne almaktadır. Yarı-rijit birleşimlerin modellenmesinde Frye ve Morris polinom modeli kullanılmaktadır. Çerçeve elemanları Türk 'Çelik Yapıların Hesap ve Yapım Kuralları' standardına (TS 648, 1980) göre boyutlandırılmaktadır. Tasarım işlemleri etkileşimlidir ve tasarımcıya bilgisayarla etkileşime girerek, pratik ve ekonomik nedenlerden dolayı, eleman enkesitlerini ve birleşim parametrelerini değiştirme seçeneklerini sunar. Yöntemin etkinliğini göstermek için, değişik birleşim tiplerine sahip iki tasarım örneği sunulmaktadır. Yarı-rijit birleşim modellemesi, rijit birleşim modellemesine göre daha ekonomik çözümler vermektedir. Ayrıca, birleşimlerin rijitliklerinde yapılan değişikliklerin ekonomik çözümler oluşturabileceği ve çerçevelerin yanal ötelenmelerinde ise değişikliklere yol açacağı gösterilmektedir.
**Anahtar Sözcükler:** Yarı-rijit birleşimler, çelik tasarımı, lineer olmayan analiz, ötelenmesi önlenmemiş çerçeveler.

**ABSTRACT**

This article presents an analysis and design method for steel frames with semi-rigid connections. 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 Turkish Building Code for Steel Structures (TS 648, 1980). 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 the 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. It is also shown that changes in the stiffness of the connections may result in economical solutions and alteration in the drifts of the frames.
**Keywords:** Semi-rigid connections, steel design, non-linear analysis, unbraced frames.

## 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

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connections are between the two extreme assumptions and possess some rotational stiffness. Full scale testing requires to explain 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 magnify 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 ones 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 [1-5].

American Institute of Steel Construction (AISC) Load and Resistance Factor Design (LRFD) specification [6] describes two types of steel construction: fully restrained (FR type) and partially restrained (PR type). This specification requires that the connections of the PR type constructions be considered flexible (semi-rigid) and, this flexibility be evaluated by a reasonable analysis or experimental works. On the other hand, Eurocode 3 [7] proposes three types of connection: rigid; semi-rigid and normally pinned or hinged. Giving clear demarcation lines with exact values among these types of connections is the difference of Eurocode 3 from AISC-LRFD. There has not been any information on semi-rigid connections in Turkish Steel Design specifications [8,9].

The aim of the present study is also to consider semi-rigid connections in the design of steel frames according to the specifications of TS 648 [8] and thus to account for the non-linear behaviour due to connection characteristics and P-Δ effects of beam-column members. A polynomial model proposed by Frye and Morris [10] 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.

## 2. CONNECTION MODELLING

A connection rotates through angle $\theta_r$ caused by applied moment M. This is the angle between the beam and the 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 Figure 1 [11]. The geometry and size parameters of six types of connections are shown in Figure 2 [11]. In the present work, a polynomial model offered by Frye and Morris [10] 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 $\kappa$ 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 [12].

## 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 nonlinear

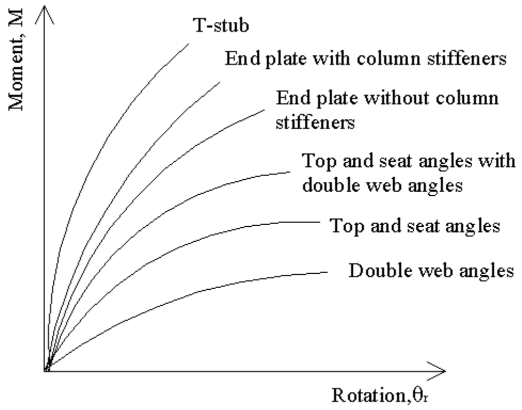


**Figure 1.** Moment- rotation curves of semi-rigid connections [11].

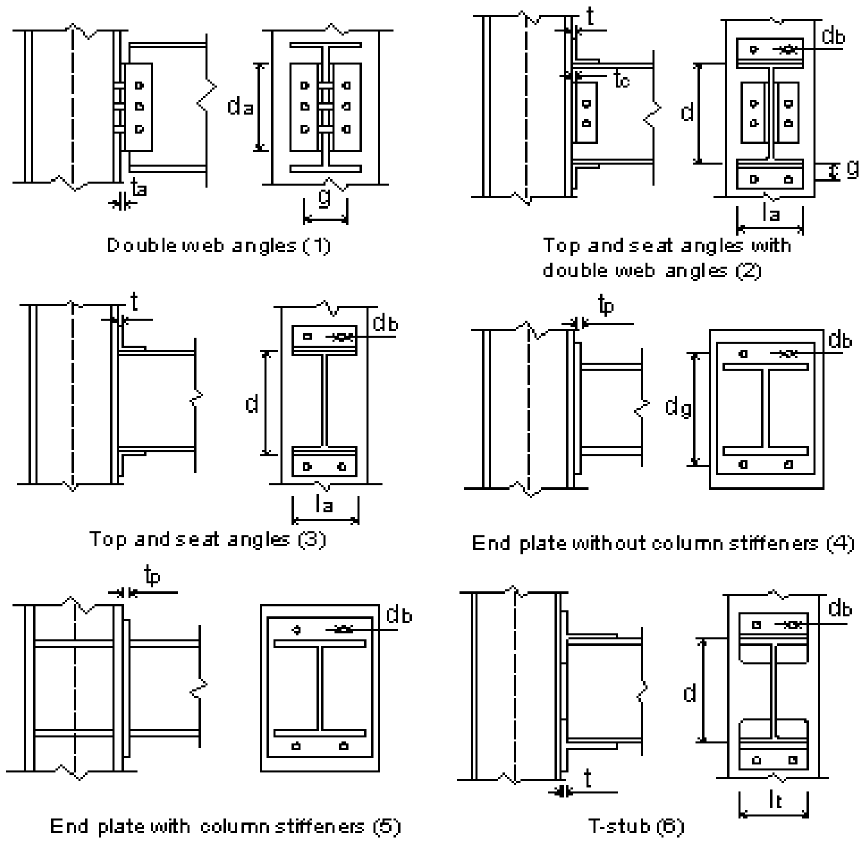


**Figure 2.** Connection types and size parameters (type-nos are given in brackets) [11].

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

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

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, two types of members are adopted for convenience 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.

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

**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 follows:

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

where $[k_E]_i$ is conventional linear-elastic stiffness matrix and $[k_p]_i$ is 'geometrical stiffness matrix' given as [13]

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

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

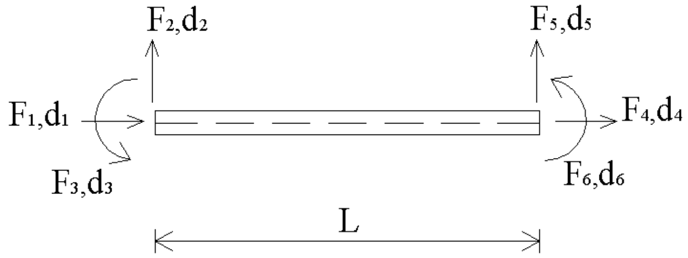


**Figure 3.** A plane-frame member with end forces and displacements.

**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 Figure 4 [14]. $\theta_{rA}$ and $\theta_{rB}$ are the relative spring rotations of both ends and $k_A$ and $k_B$ are the corresponding spring stiffnesses expressed as:

$$k_A = \frac{M_A}{\theta_{rA}} \tag{4}$$

$$k_B = \frac{M_B}{\theta_{rB}} \tag{5}$$

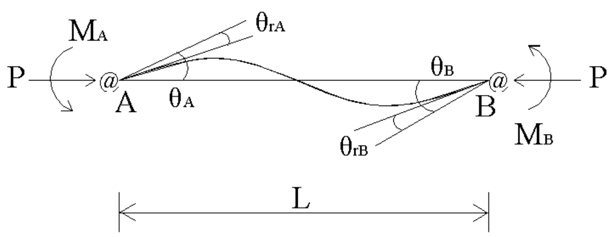


**Figure 4.** Beam member with rotational springs [14].

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

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

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

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

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

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

where

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

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

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

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

Eqs. (7) are converted to the following stiffness matrix of a semi-rigid beam member with 6 degrees of freedom in local coordinates [14].

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

where A is the cross-sectional area of the member. Applying the known steps of the matrix displacement method, this matrix is obtained in global or system coordinates for each member and system stiffness matrix is constituted. The relationships between the 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. Analysis Procedure

The system stiffness matrix is constructed by superimposing the member stiffness matrices which 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:

$$[S]\{\Delta D\} = \{\Delta F\} \tag{10}$$

where $[S]$ is the system stiffness matrix, $\{\Delta F\}$ is the incremental load vector, and $\{\Delta D\}$ is the incremental displacement vector. The incremental equations (10) are iteratively solved by a sequence of linear steps. In the present work an approach called 'secant stiffness method' [13] is used for calculation of connection stiffness due to non-linear character of moment-rotation relationship of semi-rigid connections. The connection secant stiffness, SE, is defined as:

$$SE = \frac{\Delta M}{\Delta \theta_r} \tag{11}$$

where $\Delta M$ 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, system stiffness matrix is formed at the start of each iterative cycle. It requires calculation of the connection secant stiffness at the beginning of each cycle, and change 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 Figure 5. Convergence is obtained when the difference between joint displacements of two consecutive cycles falls below a specified tolerance. In reality, the connections are preloaded by the dead weight of the girders. While an increase in gravity load (due to superimposed dead and live loads) will cause the connections to continue loading, the application of a lateral load (e.g., wind) may cause windward connections to unload. Negative stiffness of connections occurs in this case. To prevent the unloading of connections (e.g., negative stiffness), the vertical and lateral loads are applied to the frame at the same time starting from zero to its final value with small increments in the analysis procedure. Therefore the unloading of connections is not accounted for in this study.

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.

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

1. Divide applied loads into a number 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 system stiffness matrix $[S]$.
4. Solve Eq.(10) for $\{\Delta D\}$ and then determine the incremental member end forces.
5. Obtain the connection secant stiffness by Eq. (11).
6. Update the terms in the member stiffness matrices using the latest connection secant stiffnesses, 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 [8].

*For members subjected to both axial compression and bending stresses*:

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_m \sigma_{bx}}{\left(1 - \frac{\sigma_{eb}}{\sigma'_{ex}}\right)\sigma_{Bx}} \leq 1.0 \tag{12}$$

$$\frac{\sigma_{eb}}{0.6\sigma_a} + \frac{\sigma_{bx}}{\sigma_{Bx}} \leq 1.0 \tag{13}$$

When $\sigma_{eb}/\sigma_{bem} \leq 0.15$, Eq.(14) is permitted in lieu of Eqs.(12) and (13).

$$\frac{\sigma_{eb}}{\sigma_{bem}} + \frac{\sigma_{bx}}{\sigma_{Bx}} \leq 1.0 \tag{14}$$

*For members subjected to both axial tension and bending stresses*:

$$\frac{\sigma_{eç}}{0.6\sigma_a} + \frac{\sigma_{çx}}{\sigma_{çem}} \leq 1.0 \tag{15}$$

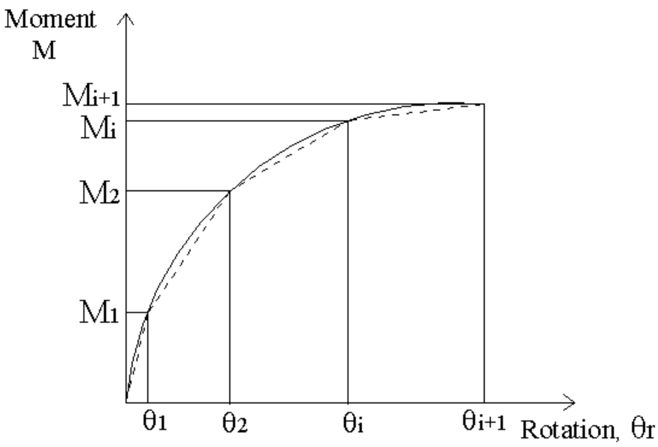


**Figure 5.** Connection secant stiffness through load increments.

In Eqs.(12)-(15) the subscript x, combined with subscripts b, B and e indicates the axis of bending about which a particular stress or design property applies, and $\sigma_{bem}$ is axial compressive stress permitted in the existence of axial force alone, $\sigma_{Bx}$ is compressive bending stress permitted in the existence of bending moment alone, $\sigma'_{ex}$ is Euler stress divided by a factor of safety, $\sigma_{eb}$ is computed axial compressive stress, $\sigma_{bx}$ is computed compressive bending stress at the point under consideration, $C_m$ is a coefficient whose value is taken as 0.85 for compression members in unbraced frames, $\sigma_a$ is the yield stress of steel. In Eq.(15), $\sigma_{eç}$ is the computed axial tensile stress, $\sigma_{çx}$ is the computed bending tensile stress and $\sigma_{çem}$ is the allowable bending stress which is equal to $0.6\sigma_a$. The allowable bending stress is increased by 0.15 in accordance with the specification when produced by wind or earthquake acting in combination with the design dead and live loads. Definitions of the permitted and Euler stresses and other details are given in the specifications [8].

The computed stresses are determined from the 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 compressive stress $\sigma_{bem}$ and Euler stress $\sigma'_{ex}$ 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 [15].

$$\frac{G_A G_B (\pi / K)^2 - 36}{6(G_A + G_B)} = \frac{\pi / K}{\tan(\pi / K)} \tag{16}$$

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

$$G = \frac{\sum I_c / L_c}{\sum I_g / L_g} \tag{17}$$

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

In Eq.(16), 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.(17) is multiplied by the following factors to consider 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 the corresponding end.

## 5. DESIGN PROCEDURE

Interacting with the computer, a design engineer can select member size based on the value of interaction ratio given by Eqs.(12-15) compared with 1. An interaction ratio value greater than 1 implies that the member is insufficient and a larger section should be selected. An interaction ratio 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 designer is convinced of his member size 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 the non-linear behaviour of the semi-rigid connections and the 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.(12)-(15).
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 stiffness to achieve economic designs and control frame drift.
5. Repeat the procedure until adequate design is reached.

## 6. DESIGN EXAMPLES

A computer program has been developed in the present study, which is the implementation of the design procedure. Two design examples are presented to demonstrate the application of the design algorithm. As the considered beam-to-column connections are set up at right angle, rectangular frames with reasonable superimposed dead and live loads in conjunction with lateral loads (earthquake or wind load) are selected as design examples. 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 steel with a modulus of elasticity of 205940 MPa and yield stress of 235.4 MPa. Material density is 77000 N/m$^3$. European wide flange beams (i.e., HE sections) [16] are used in the designs of the frames. The types of semi-rigid connections used in the designs are the same as the ones given in Figure 2.

**6.1. Three-storey, Single-bay Frame**

The dimensions, loading and numbering of members of the 3-storey single-bay frame are shown in Figure 6. The connection size parameters which remain fixed during design process are given in Table 2 depending on the connection types. The results of the final designs for six types of semi-rigid connections and also rigid connection are given in Table 3 in terms of the weights and the drifts (the sway of the top storey).

The results of the designs in Table 3 show that the weights of the semi-rigid frames decrease by 7-10% depending on connection types compared to the rigid frame weights. As regards the frame drifts, the drifts of semi-rigid frames increase by 38-91% compared to the drift of rigid frame. The final design sections and their maximum interaction ratio values for type 2 connection (top and seat angles with double web angles) are presented in Table 4.

For the same design, absolute maximum end-moments of the members are given in Table 5 while the positive span-moments of the beam members are presented in Table 6. The results of Table 5 and Table 6 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 beams and in columns in general when compared to those of rigid frame.

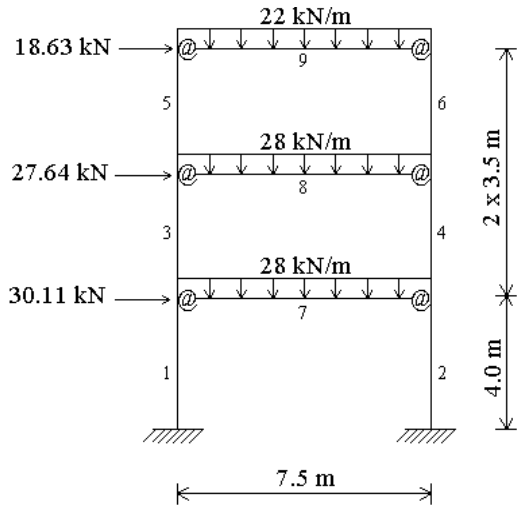


**Figure 6.** Three-storey, single-bay frame.

**Table 2.** The fixed connection size parameters for three-storey, single-bay frame.

| Connection types | Connection size parameters (cm) |
|---|---|
| 1 | $t_a$=2.0 g=14.0 |
| 2 | t=2.0 $t_c$=2.0 g=6.5 |
| 3 | t=2.0 $d_b$=2.8 |
| 4 | $t_p$=2.0 $d_b$=2.8 |
| 5 | $t_p$=2.0 $d_b$=2.8 |
| 6 | t=2.0 $d_b$=2.5 |

**Table 3.** Final design results of 3-storey, single-bay frame

| Semi-rigid connection type | Weight (N) | | Drift (cm) | |
|---|---|---|---|---|
| | Semi-rigid connection | Rigid connection | Semi-rigid connection | Rigid connection |
| 1 | 30862 | 33264 | 3.89 | 2.40 |
| 2 | 29881 | | 3.32 | |
| 3 | 30440 | | 4.59 | |
| 4 | 30979 | | 3.89 | |
| 5 | 30440 | | 4.28 | |
| 6 | 30048 | | 3.38 | |

**Table 4.** Final design sections and the maximum interaction ratio values for 3-storey, single-bay frame

| Member no. | Semi-rigid connection | | Rigid connection | |
|---|---|---|---|---|
| | Section | The ratio | Section | The ratio |
| 1,2 | HE 320AA | 0.993 | HE 340AA | 0.923 |
| 3,4 | HE 260AA | 1.000 | HE 300AA | 0.870 |
| 5,6 | HE 240AA | 0.961 | HE 280AA | 0.994 |
| 7 | HE 360AA | 0.866 | HE 400AA | 0.868 |
| 8 | HE 340AA | 0.871 | HE 340AA | 0.988 |
| 9 | HE 300AA | 0.857 | HE 300AA | 0.776 |

**Table 5.** Absolute maximum end-moments in 3-storey single-bay frame

| Member no. | Semi-rigid connection moment (kN-m) | Rigid connection moment (kN-m) |
|---|---|---|
| 1 | 86.75 | 70.00 |
| 2 | 112.09 | 119.69 |
| 3 | 18.16 | 20.11 |
| 4 | 68.56 | 98.28 |
| 5 | 23.45 | 60.45 |
| 6 | 62.85 | 100.38 |
| 7 | 136.03 | 202.35 |
| 8 | 112.36 | 170.82 |
| 9 | 62.85 | 100.38 |

**Table 6.** Span moments in the beams of 3-storey, single-bay frame

| Member no. | Semi-rigid connection moment (kN-m) | Rigid connection moment (kN-m) |
|---|---|---|
| 7 | 153.97 | 97.44 |
| 8 | 140.67 | 82.09 |
| 9 | 109.19 | 71.94 |

To examine the effect of the connection stiffness on the design of frames, the same frame with connection type 2 is designed with various connection size parameters and the results are presented in Table 7.

The results of Table 7 indicate that, the softening of the connection results in an increase in the frame drift, but a decrease in the frame weight.

**Table 7.** The effect of connection stiffness on the design of 3-storey, single-bay frame

| Connection size parameters (cm) | Weight (N) | Drift (cm) |
|---|---|---|
| $t=t_c=2.0$ $g=5.5$ | 29871 | 3.20 |
| $t=t_c=1.8$ $g=6.0$ | 29028 | 3.41 |
| $t=t_c=1.6$ $g=6.5$ | 27586 | 3.81 |

**6.2. Four-storey, Three-bay Frame**

Figure 7 shows configuration, dimensions, loading, and numbering of members. The results of the final designs for six types of semi-rigid connections together with rigid connection are presented in Table 8 in terms of the weights and the drifts.

The results in Table 8 indicate that the weights of semi-rigid frames decrease by 2-9% compared to the weights of rigid frames. On the other hand, the drifts of semi-rigid frames increase by 39-65% depending upon connection types compared to the rigid frame drifts.

The final design sections and their maximum interaction ratio values for type 2 connection are given in Table 9. The same fixed connection size parameters as the ones for the first design example are used in this example.

**Table 8.** Final design results of 4-storey, 3-bay frame.

| Semi-rigid connection Type | Weight (N) | | Drift (cm) | |
|---|---|---|---|---|
| | Semi-rigid connection | Rigid connection | Semi-rigid connection | Rigid connection |
| 1 | 99675 | 101332 | 6.95 | 4.21 |
| 2 | 92310 | | 5.91 | |
| 3 | 99675 | | 6.80 | |
| 4 | 98445 | | 6.36 | |
| 5 | 99675 | | 6.55 | |
| 6 | 92594 | | 5.85 | |

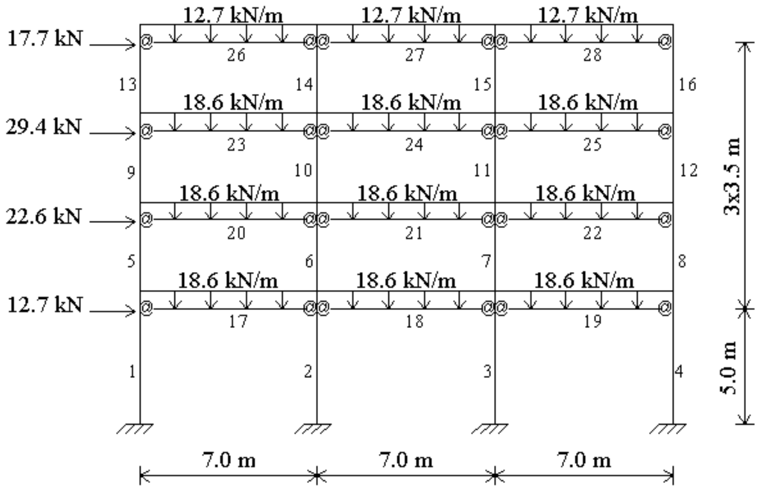


**Figure 7.** Four-storey, three-bay frame.

**Table 9.** Final design sections and interaction ratio values for 4-storey, 3-bay frame.

| Member no. | Semi-rigid connection | | Rigid connection | |
|---|---|---|---|---|
| | Section | The ratio | Section | The ratio |
| 1-4 | HE 500AA | 0.894 | HE 400AA | 0.988 |
| 5-12 | HE 340AA | 0.910 | HE 300AA | 0.890 |
| 13-16 | HE 200AA | 0.818 | HE 220AA | 0.895 |
| 17-25 | HE 280AA | 0.953 | HE 340AA | 0.898 |
| 26-28 | HE 240AA | 0.943 | HE 240AA | 0.954 |

For the same design, absolute maximum end-moments of the members are given in Table 10 and, span-moments of the beams are given in Table 11. For the semi-rigid frame, it is found from the results of Table 10 and Table 11 that overall maximum moments decrease in beams while increasing in some columns, as compared to those of rigid frames. The frame is also designed for three different connection stiffness values and the results are presented in Table 12. It is found from the results of Table 12 that, reducing the connection stiffness causes an increase in the drift and a decrease in the weight of frame.

**Table 10.** Absolute maximum end-moments in 4-storey 3-bay frame

| Member no. | Semi-rigid connection moment (kN-m) | Rigid connection moment (kN-m) |
|---|---|---|
| 1 | 162.99 | 115.64 |
| 2 | 188.47 | 154.58 |
| 3 | 187.79 | 152.47 |
| 4 | 184.48 | 152.65 |
| 5 | 32.59 | 19.74 |
| 6 | 74.21 | 78.90 |
| 7 | 74.29 | 76.52 |
| 8 | 68.49 | 79.87 |
| 9 | 8.74 | 11.71 |
| 10 | 67.80 | 62.58 |
| 11 | 69.10 | 58.82 |
| 12 | 64.34 | 74.97 |
| 13 | 63.31 | 17.06 |
| 14 | 25.42 | 25.24 |
| 15 | 20.59 | 18.64 |
| 16 | 36.08 | 44.35 |
| 17 | 98.69 | 158.53 |
| 18 | 97.34 | 149.63 |
| 19 | 98.40 | 151.60 |
| 20 | 100.74 | 145.59 |
| 21 | 99.66 | 137.36 |
| 22 | 101.72 | 138.43 |
| 23 | 89.14 | 122.45 |
| 24 | 86.70 | 114.59 |
| 25 | 88.20 | 108.69 |
| 26 | 52.36 | 70.08 |
| 27 | 50.33 | 67.09 |
| 28 | 36.08 | 48.44 |

**Table 11.** Span moments in the beams of 4-storey, 3-bay frame

| Member no. | Semi-rigid connection moment (kN-m) | Rigid connection moment (kN-m) |
|---|---|---|
| 17 | 83.60 | 62.04 |
| 18 | 79.61 | 49.53 |
| 19 | 80.01 | 50.61 |
| 20 | 83.17 | 58.27 |
| 21 | 79.41 | 46.00 |
| 22 | 79.95 | 47.30 |
| 23 | 77.99 | 52.79 |
| 24 | 73.11 | 40.82 |
| 25 | 73.41 | 43.41 |
| 26 | 50.35 | 36.76 |
| 27 | 39.91 | 24.81 |
| 28 | 45.19 | 31.71 |

**Table 12.** The effect of connection stiffness on the design of 4-storey, 3-bay frame

| Connection size parameters (cm) | Weight (N) | Drift (cm) |
|---|---|---|
| $t=t_c=2.0$ $g=5.5$ | 96154 | 5.19 |
| $t=t_c=1.8$ $g=6.0$ | 90868 | 6.01 |
| $t=t_c=1.6$ $g=6.5$ | 90702 | 6.40 |

**7. SUMMARY AND CONCLUSIONS**

A combined analysis and design procedure is presented for the design of steel frames with semi-rigid connections accounting for the 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 creates lighter frames. The reason for this conclusion is that the overall maximum moments in beams decrease by comparison with the moments of rigid frames and thus smaller sections are assigned to them. However, the moments increase in some columns while decreasing in others when compared to those of rigid frames. It is noticed from Table 9 that some column sections are larger and the others are smaller than those of the rigid frame. The overall semi-rigid frame weight decreases consequently.

The semi-rigid connections cause a large increase in the frame drift. Trying various connection stiffness values, the drift can be controlled and economic frames can be obtained. The softening of connections results in considerable increase in frame drift. An economic frame system can be attained by controlling the frame drift with the stiffness of the connections.

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