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Design and analysis of non-linear space frames with semi-rigid connections

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Abstract. Semi-rigid connections are the actual behavior of beam-to-column connections in steel frames. However, the behavior of semi-rigid connections is not taken into account for the simplicity in the conventional analysis and design of steel frames. A computer-based analysis and design has been studied for the three-dimensional steel frames with semi-rigid connections. The nonlinear analysis which includes the effects of the flexibility of connections is used for this study. It is designed according to the buckling and combined stress constraints under the present loading after the joint deformations and the member end forces of the space frame are determined by the stiffness matrix method. The semi-rigid connection type is limited to the top and bottom angles with a double web angle connection. The Frye-Morris polynomial model is used to describe the non-linear behavior of semi-rigid connections. Various design examples are presented to demonstrate the efficiency of the method. The results of design and analysis of unbraced semi-rigid frames are compared to the results of unbraced rigid frames under the same design requirements.

Keywords: semi-rigid connection; stiffness method; space frames; nonlinear analysis; structural analysis; structural design

1. Introduction

Beam-to-column connections play an important role in behavior of steel frames. Steel frames are traditionally analyzed and designed assuming that beam-to-column connections are ideally pinned or fully rigid connections. Despite the simplification of the analysis and design process, there are differences between idealised behavior and actual behavior. Experimental investigations (Shi *et al*. 2007, Girão Coelho and Bijlaard 2007) have shown that the actual behavior that falls between these two idealised models has been classified as semi-rigid steel connections. Neglecting the actual behavior of the connection in the analysis may lead to unrealistic predictions of the response and reliability of steel frames. As identified by Ngo-Huu *et al*. (2012), the flexibility of connections affects the strength and displacement response of steel frames. Some researchers (Hadianfard and Razani 2003, Zlatkov *et al*. 2011) argue, the semi-rigid behavior of the connections must be properly considered by design and calculation of all engineering structures if more reliable results are desired. EN 1993-1-8 has classified a joint as nominally pinned, rigid or

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semi-rigid in elastic global analysis. Several researches have shown that the performance of semi-rigid connection provides benefits such as size and weight reductions (e.g., Dhillon and O'Malley 1999, Hayalioğlu and Değertekin 2005, Cabrero and Bayo 2005, Girão Coelho 2013).

The behavior of semi-rigid connections has been a non-linear structure and is not resolved easily. To model the behavior of connections, the moment-rotation curve must be used. The rotational deformation is customarily expressed as a function of the moment in the connection. When a moment M is applied to a connection, it rotates by θ_r (Chen and Lui 1991). A way to obtain the curve is full-scale experimental tests (Chen and Lui 1991). Many tests on semi-rigid connections were conducted by some researchers (Abidellah *et al*. 2012, Girão Coelho *et al*. 2009). Using the data of the tests, analytical expressions (e.g., Frye and Morris 1975, Richard and Abbott 1975) have been developed to obtain the moment-rotation curve if test data are not available for any connection details. In addition, prediction methods (Weynand *et al*. 1995, Huber and Tschemmernegg 1998, Simoes da Silva *et al*. 2000, Simoes da Silva and Girão Coelho 2001) for the behaviour of joints present the approach based on mechanical models by using a combination of rigid and flexible components.

Despite the growing amount of published research (Simoes 1995, Doğan and Saka 2011, Kameshki and Saka 2003, Değertekin and Hayalioğlu 2004) about the analysis and design of the planar steel frameworks accounting for the behavior of semi-rigid connections, studies (Nguyen and Kim 2013, Aydın *et al*. 2007, Kim and Choi 2001, Kaveh and Moez 2006, 2008) on the analysis of the space steel frameworks are limited. Some researchers developed the several methods for analysis of semi-rigid space frames. For example, a number of studies (Ngo-Huu *et al*. 2012, Nguyen and Kim 2013) have presented the numerical procedures based on the beam-column method.

Considering the theoretical gap about design of semi-rigid space frames in the literature, the purpose of this paper is to develop a computer-oriented nonlinear analysis and design method for unbraced space steel frames with semi-rigid connections by using a specific computer program (MRVSSF), prepared in the MATLAB language. Taking into consideration the non-linear behavior of beam-to-column connections, the analysis of three dimensional (3D) steel frames with

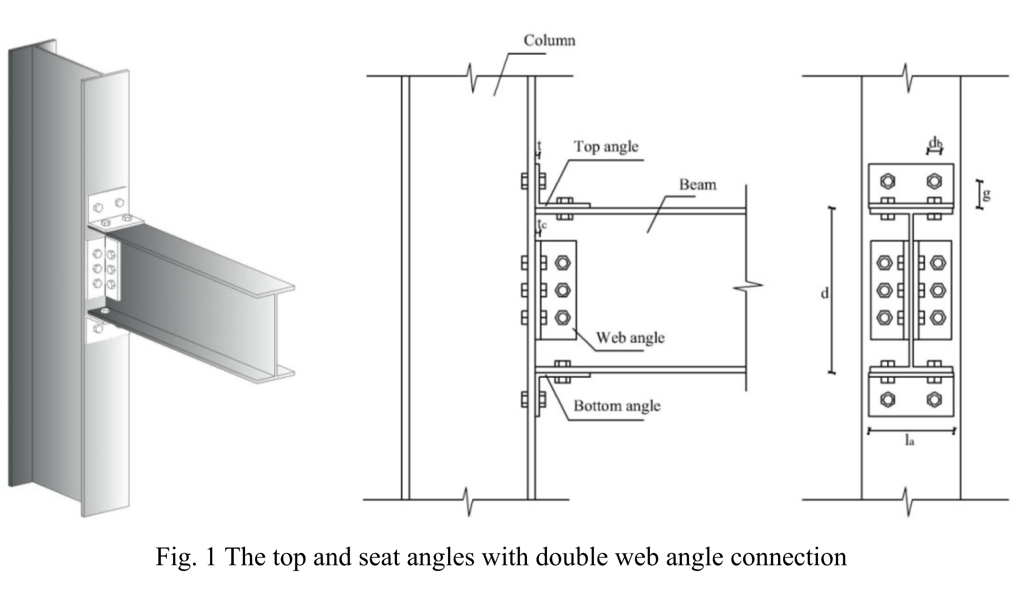


Fig. 1 The top and seat angles with double web angle connection

semi-rigid and rigid connections was obtained with the stiffness matrix method. The semi-rigid behavior of in the directions of both *xy* and *yz* of the connections is considered in the analysis of space frames by using the moment-rotation curve. MRVSSF program gives this chance to the designer to change member cross-section or connection type, by interacting with computer.

2. Semi-rigid connection modelling

While columns do not have any internal flexible connections, the beams possess semi-rigid end connections (Değertekin and Hayalioğlu 2004). If the ends of the beam are not rigidity but flexibly connected to columns, the effect of connection flexibility can calculate adaptation of the relation between the ends moment and the ends rotation. In frame analysis, the effect is modelled by attaching rotational springs with stiffness moduli K_A and K_B.
The stiffness module k_A and k_B of the flexible connections are determined by considering non-linear connection behavior. The relationship between the end-moments (M_A and M_B) and end-rotations (θ_{rA} and θ_{rB}) of a beam can be written, respectively in the ends A and B, as follows

$$k_A = \frac{M_A}{\theta_{rA}} \qquad k_B = \frac{M_B}{\theta_{rB}} \tag{1}$$

During the present study, top and seat angles were used with double web angle in the connection. The type of connections is non-linear over the entire range of loading. Top and seat angle connections with double web angles are consisting of two angles connecting the beam flanges to the column, and two angles connecting the beam web to the column. The geometry of this connection is given in Fig. 1.

The connections show the nonlinear behavior over the entire range of loading because of geometrical properties. The behavior is represented by the moment-rotation curve of connections. The rotational deformation is customarily expressed as a function of the moment in the connection. The stiffness modules are determined to take into account the non-linear curves.

The researches have made full-scale experiments in order to obtain real moment-rotation characteristics of connections. Several moment-rotation relationships have been derived from the experimental studies for modelling steel frames with semi-rigid connections. In the article, a polynomial model offered by Frye and Morris (1975) is used. Because its application is easy, and it presents the M-θ_r characteristics reasonably well. Frye and Morris presented an odd-power polynomial to present the relationship

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \tag{2}$$

where K is the standardisation parameter, C_1, C_2, C_3 are the curve-fitting constants and the values of these constants are given in the refer (Frye and Morris 1975).

The rotational stiffness k_A and k_B of springs at the ends of member are obtained calculating the rotation value corresponding to the moment obtained with Eq. (2).

Most connections exhibit the nonlinear behavior almost from the beginning of the loading. So the connection flexibility requires a process of iterative solution in frame analysis. If the connections are not loaded yet, the stiffness modules have almost a linear slope that is equal to the initial slope of the M-θ_r curve.

The connection stiffness in the loading is obtained as the rotation for a moment value. If the connection is not loaded, the connection flexibility is the initial stiffness. In the paper, it is used the formulation of the initial stiffness (R_{ki}) proposed by Frye and Morris (1975).

3. Analysis of semi-rigid connections with stiffness method

The first step in the structural analyses is to develop the analytical model of a real structure and it is necessary to point out that structural response predicted from the analysis is valid only to the extent that the model represents the real structure (Kassimali 1999). When the analytical model was created, the behavior of joints as rigid, hinged or semi-rigid is taken into account.

The stiffness method is an efficient way to solve complex determinant or indeterminant structures. Members of space frames may be arranged in various directions and connected as rigid, hinged or semi-rigid. The members of a space frame may be subjected to bending moments about both principal axes, shears in both principal directions, torsional moments, and axial moments. A free joint of a space frame has six degrees of freedom-the translations in the x, y and z directions and the rotations about the x, y, and z axes. Therefore, a member of a space frame has 12 degrees of freedom as shown in Fig. 2.

$[k]$ represents the 12 × 12 member stiffness matrix in member local coordinates and given as

$$[k] = \begin{matrix} [k]_{11} & [k]_{12} \\ [k]_{21} & [k]_{22} \end{matrix} \tag{3}$$

where (Tezcan 1963)

$$[k]_{11} = \begin{bmatrix} H & 0 & 0 & 0 & 0 & -G_i \\ 0 & S & 0 & 0 & 0 & 0 \\ 0 & 0 & D & C_i & 0 & 0 \\ 0 & 0 & C_i & A_i & 0 & 0 \\ 0 & 0 & 0 & 0 & T & 0 \\ -G_i & 0 & 0 & 0 & 0 & E_i \end{bmatrix} \qquad [k]_{12} = \begin{bmatrix} -H & 0 & 0 & 0 & 0 & -G_j \\ 0 & -S & 0 & 0 & 0 & 0 \\ 0 & 0 & -D & C_j & 0 & 0 \\ 0 & 0 & -C_i & B & 0 & 0 \\ 0 & 0 & 0 & 0 & -T & 0 \\ G_i & 0 & 0 & 0 & 0 & F \end{bmatrix}$$

$$[k]_{22} = \begin{bmatrix} H & 0 & 0 & 0 & 0 & -G_j \\ 0 & S & 0 & 0 & 0 & 0 \\ 0 & 0 & D & -C_j & 0 & 0 \\ 0 & 0 & -C_j & A_j & 0 & 0 \\ 0 & 0 & 0 & 0 & T & 0 \\ G_j & 0 & 0 & 0 & 0 & E_j \end{bmatrix} \qquad [k]_{21} = [k]_{12}^T \tag{4}$$

where (Tezcan 1963)

$$\begin{aligned} &A_{i,j} = \alpha_{i,j}\frac{EI_x}{L} \quad B = b\frac{EI_x}{L} \quad E_{i,j} = e_{i,j}\frac{EI_z}{L} \quad F = f\frac{EI_z}{L} \quad S = \frac{EA}{L} \\ &C_{i,j} = \frac{A_{i,j}+B}{L} \quad D = \frac{C_i + C_j}{L} \quad G_{i,j} = \frac{E_{i,j}+F}{L} \quad H = \frac{G_i + G_j}{L} \quad T = \frac{GJ}{L} \end{aligned} \tag{5}$$

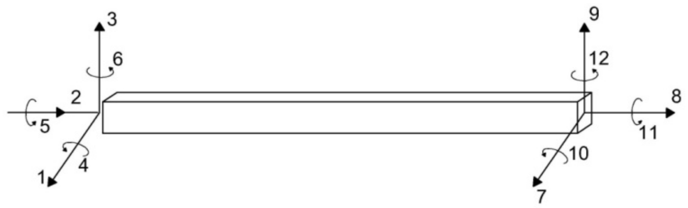


Fig. 2 A space frame member with end forces

where E and G are the elasticity and shear modulus of material; A, L, J are the area, the length and the torsional constant of the member respectively; Ix and Iz are the moment of inertia with respect to x, z axes; $a_{i,j}$, $e_{i,j}$, b, f are stiffness constants and 4, 4, 2, 2 for rigid connections, respectively. For members with semi-rigid end connections, the constants must be calculated to take into account semi-rigid behavior. This study, using the literature (Dhillon and O'Malley 1999), is a revised solution of three dimensional frame members with semi-rigid connections

$$a_{i,j} = \frac{1}{R_X}\left(4 + \frac{12EI_X}{Lk_{AX,BX}}\right) \quad e_{i,j} = \frac{1}{R_Z}\left(4 + \frac{12EI_Z}{Lk_{AZ,BZ}}\right) \quad b = \frac{2}{R_X} \quad f = \frac{2}{R_Z} \tag{6}$$

$$\text{where:} \quad R_{X,Z} = \left(1 + \frac{4EI_{X,Z}}{Lk_{AX,AZ}}\right)\left(1 + \frac{4EI_{X,Z}}{Lk_{BX,BZ}}\right) + \left(\frac{EI_{X,Z}}{L}\right)^2\left(\frac{4}{k_{AX,AZ}k_{BX,BZ}}\right) \tag{7}$$

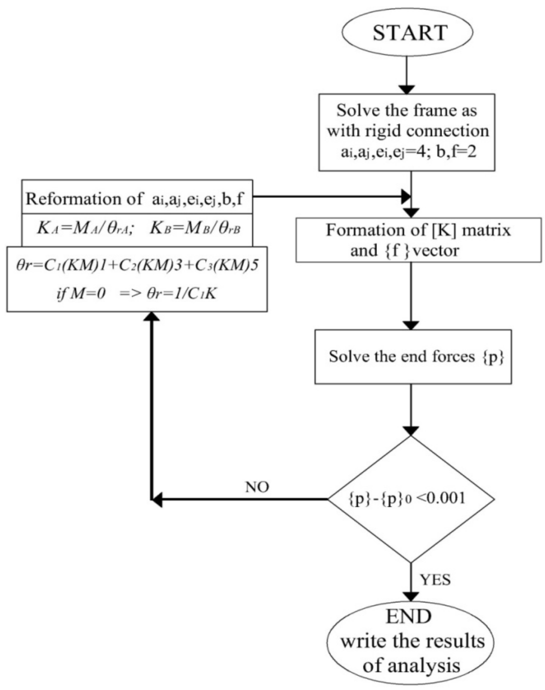


Fig. 3 Analysis procedure

in which k_{Am} and k_{Bm}, are the stiffness modules of the flexible connections at the ends of the member, dimensional spring coefficients and show moments in response to one radian.

{*f*} fixed-end reactions of members are obtained by considering the following fixed-end moments

$$f_4 = \frac{(\alpha_i \Delta_i - b\Delta_j)}{L} \quad f_6 = \frac{(e_i \Delta_i - f\Delta_j)}{L} \quad f_{10} = \frac{(b\Delta_i - \alpha_j \Delta_j)}{L} \quad f_{12} = \frac{(f\Delta_i - e_j \Delta_j)}{L} \tag{8}$$

in which Δ_i and Δ_j, are formulas of the fixed-end moment at ends of members.

Analysis of the space frames with semi-rigid connection is summarized in Fig. 3.

4. Design of semi-rigid connection

4.1 Section classification

The program MRVSSF designs according to the Turkish Building Code for Steel Structures (TS-EN648 1980) by selecting suitable sections from a standard set of steel sections (European-IPE sections). However, connection parameters are not standard. Suitable connection parameters such as the diameter of bolts, angle thickness, and connection depth are calculated for each steel section. These parameters are present in the section table.

4.2 Design considerations

In the present study, it is designed according to the combined stress constraints as specified in the TS-EN648 (1980).

The combined stress constraints taken from TS-EN648 (1980) are expressed in the following equations. For members subjected to both axial compression and bending stresses

$$\text{for} \quad \frac{\sigma_{eb}}{\sigma_{bem}} \le 0.15 \quad \rightarrow \quad \frac{\sigma_{eb}}{\sigma_a} + \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{\sigma_{by}}{\sigma_{By}} \le 1 \tag{9}$$

$$\begin{gathered} \text{else} \quad \frac{\sigma_{eb}}{\sigma_{bem}} + \frac{C_{mx}\sigma_{bx}}{\left(1 - \dfrac{\sigma_{eb}}{\sigma'_{ex}}\right)\sigma_{Bx}} + \frac{C_{my}\sigma_{by}}{\left(1 - \dfrac{\sigma_{eb}}{\sigma'_{ey}}\right)\sigma_{By}} \le 1 \\ \frac{\sigma_{eb}}{0.6\sigma_a} + \frac{\sigma_{bx}}{\sigma_{Bx}} + \frac{\sigma_{by}}{\sigma_{By}} \le 1 \end{gathered} \tag{10}$$

where the subscript x and y combined with subscripts b, B and e indicates the axis of bending about which a particular stress or design property applies, and σ_{bem} is axial compressive stress permitted in the existence of axial force alone, σ_{Bx} and σ_{By} are compressive bending stress permitted in the existence of the bending moment alone, σ'_{ex} and σ'_{ey} are Euler stress divided by a factor of safety, σ_{eb} is computed axial compressive stress, σ_{bx} and σ_{by} are computed compressive bending stress at the point under consideration, σ_a is the yield stress of steel. C_m is a coefficient and defined as follows (Gaylord *et al*. 1992)

For rotationally restrained members with no transverse loads between the supports

$$C_m = 0.6 - 0.4\frac{M_1}{M_2} \geq 0.4 \tag{11}$$

For members with transverse loads between the supports

$$C_m = 0.85 \quad \text{(if member ends are rotationally restrained)} \tag{12}$$

For members subjected to both axial tension and bending stresses

$$\frac{\sigma_{es}}{0.6\sigma_a} + \frac{\sigma_{s,x}}{\sigma_{sem}} + \frac{\sigma_{sy}}{\sigma_{sem}} \leq 1 \tag{13}$$

where σ_{es} is the computed axial tensile stress, σ_{sx} and σ_{sy} are the computed bending tensile stress and σ_{sem} is the acceptable bending stress which is equal to $0.6\sigma_a$. Other details are given in the specifications (TS-EN648 1980).

4.3 Effective column-lenght factor

Effective column factor (K-factor) of columns must be determined for the stability design of the columns in frames with rigid and semi-rigid. The end conditions of columns depend on the stiffness of the beams and girders framing into the column, and the rigidity of the beam-column connections. The effective length factor K for the columns in a frame is determined from the

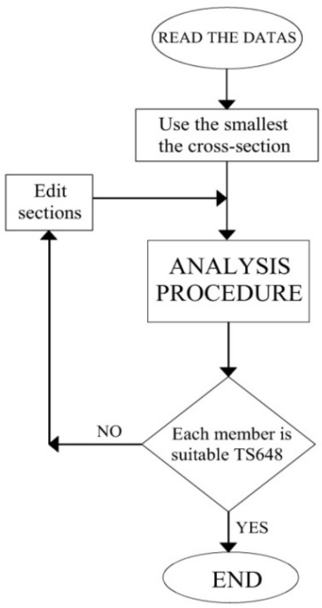


Fig. 4 Design procedure

interaction equation (Kishi *et al*. 1987). To account for end conditions of flexibly connected, the beam/girder stiffness *Ig*/*Lg* in the equation is multiplied by following factor

$$R_k = \frac{1}{\left(1 + \frac{6EI}{L_g k_{A,B}}\right)} \tag{14}$$

where $k_{A,B}$ is spring stiffness of corresponding end.

4.4 Design procedure

Design of the space frames with semi-rigid connections is summarized in Fig. 4.

5. Design examples

Various space steel frames with semi-rigid connections are investigated using program MRVSSF to demonstrate the application of the design algorithm. The designs of unbraced semi-rigid frames are compared to the designs of unbraced rigid frames under the same design requirements. The type of semi rigid connection which is at the top and seat angle with the double web angle is considered in the all examples. The moment-rotation relationship has been developed for the type of connection using the Frye-Morris Polynomial Modelling equation. The members are designed according to the combined stress constraints as specified by the TS-EN648 (1980).

5.1 Three-storey, *two-bay plane frame*

This example is taken from Değertekin and Hayalioğlu (2004) to demonstrate the validity of program MRVSSF. Toward this end, Değertekin performed a comparison using the results of the analysis procedure. The material is A36 steel with a modulus of elasticity of 210 GPa and yield

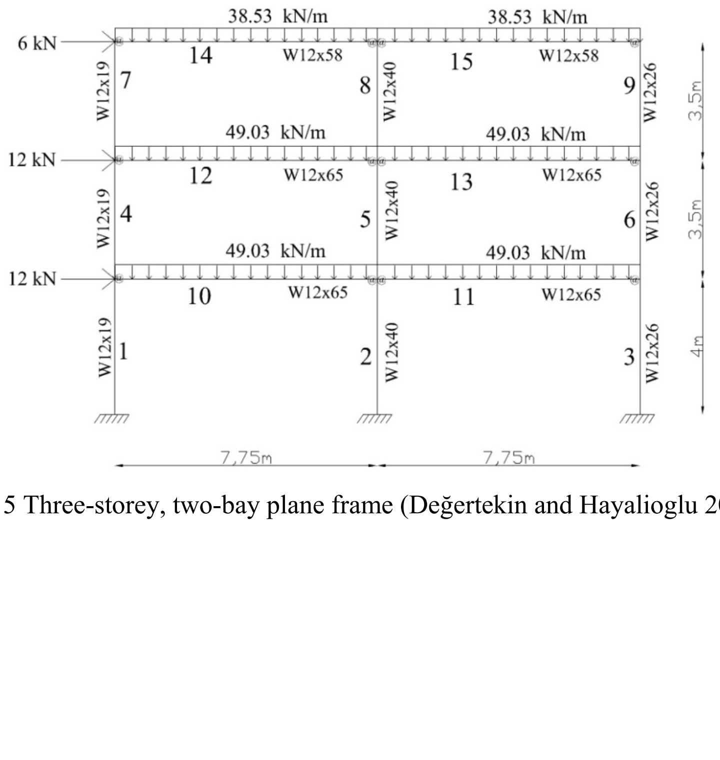


Fig. 5 Three-storey, two-bay plane frame (Değertekin and Hayalioglu 2004)

Table 1 Absolute maximum end-moments in three-storey two-bay plane frame for semi-rigid connection and rigid connection

Member No.	Rigid Connection Moment (kN-m)			Semi-rigid Connection Moment (kN-m)		
	Değertekin (2004)	MRVSSF	Differences	Değertekin (2004)	MRVSSF	Differences
1	60.40	60.44	0.1%	11.08	24.00	53.8%
2	27.72	25.82	6.9%	27.70	29.83	7.1%
3	88.55	88.77	0.2%	42.28	56.40	25%
4	106.94	106.73	0.2%	32.99	55.37	40.1%
5	16.30	16.37	0.4%	17.16	18.20	5.7%
6	118.63	118.48	0.1%	47.35	71.56	33.8%
7	131.48	139.22	5.9%	39.31	87.73	55.1%
8	6.80	6.81	0.1%	4.96	5.40	8.1%
9	139.28	131.40	6.0%	50.10	72.04	30.4%
10	288.15	288.20	0.0%	130.04	147.58	11.9%
11	253.79	253.66	0.1%	92.64	127.96	27.6%
12	267.66	267.71	0.0%	116.38	143.11	18.7%
13	247.02	246.98	0.0%	98.22	132.25	25.7%
14	219.83	219.88	0.0%	92.76	119.77	22.6%
15	213.02	213.06	0.0%	87.80	120.30	27.0%

stress of 235 MPa. Connection size parameters are t = 2.54 cm, t_c = 2.54 cm, g = 11.43 cm. The configuration, sections, dimensions, loading and numbering of members are shown in Fig. 5.

The maximum end-moments of the final analysis for semi-rigid connection and rigid connection that are compared with Değertekin and Hayalioglu (2004) are given in Table 1. The results of Table 1 show that the absolute maximum moments decrease in the frame with semi-rigid connections when compared to those of rigid frame. While MRVSSF program results show a close comparison with Değertekin and Hayalioglu (2004) for rigid connection, some differences is observed in the results for semi-rigid connection. The differences may be caused for the reasons such as difference between the analysis procedures (Değertekin uses load increments methods), the analysis is nonlinear and a solution depends on iteration, initial stiffness of both of analysis procedures is different. As a result, good agreement is observed between the results obtained by Değertekin's and MRVSSF's analysis.

5.2 One-storey, 8-member space frame

The material is grade A36 steel with a modulus of elasticity of 210 GPa and yield stress of 235 MPa. European sections (IPE sections) are used as steel sections in all other design examples considered in the present study. The value of connection size parameters such as the diameter of bolts, angle thickness, connection depth, etc. is calculated depending upon the standard steel section adopted for the beam during the design process.

63-member frame that is applied 10 kN/m constant loads through beams is selected to demonstrate the analysis application the program MRVSSF. Fig. 6 shows the frame configuration, sections, loading, dimensions and numbering of members.

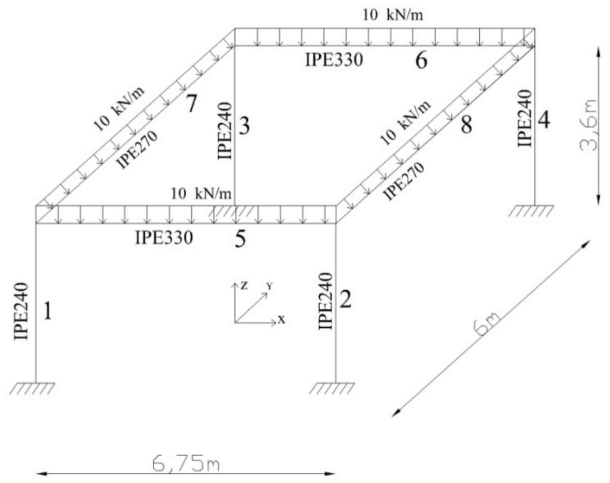


Fig. 6 One-storey, 8-member space frame

The absolute end forces of the final analysis for semi-rigid connection and rigid connection are compared in Table 2. The results of Table 2 show that the absolute end forces generally decrease in the frame with semi-rigid connections when compared to those of rigid frame. In the frame with semi-rigid connection, the shear forces and moments in the beams of the XY plane and the small torsion moments in the members are emerged because semi-rigid behavior affects the internal force distribution.

Table 2 Internal end forces of the members in one-storey, 8-member space frame for semi-rigid connection and rigid connection

Member No	Member End Forces					
	V1 (kN)	P (kN)	V3 (kN)	M1 (kNcm)	T (kNcm)	M3 (kNcm)
1i [a]	7.337	128.194	21.735	2717.941	0.000	-879.773
1i [b]	6.203	129.083	19.884	2281.710	0.098	-680.773
1j [a]	-7.337	-128.194	-21.735	5106.804	0.000	-1761.521
1j [b]	-6.203	-129.083	-19.884	4876.490	-0.098	-1552.462
2i [a]	-7.337	128.194	21.735	2717.941	0.000	879.773
2i [b]	-6.076	128.108	19.026	2329.412	0.069	776.384
2j [a]	7.337	-128.194	-21.735	5106.804	0.000	1761.521
2j [b]	6.076	-128.108	-19.026	4519.854	-0.069	1411.008
3i [a]	7.337	126.806	-18.735	-2106.424	0.000	-879.773
3i [b]	5.531	126.141	-16.741	-2060.025	0.070	-651.589
3j [a]	-7.337	-126.806	18.735	-4638.321	0.000	-1761.521
3j [b]	-5.531	-126.141	16.741	-3966.785	-0.070	-1339.676
4i [a]	-7.337	126.806	-18.735	-2106.424	0.000	879.773
4i [b]	-5.659	126.667	-16.168	-1870.528	0.070	690.443

Table 2 Continued

Member No	Member End Forces					
	V1 (kN)	P (kN)	V3 (kN)	M1 (kNcm)	T (kNcm)	M3 (kNcm)
4j [a]	7.337	-126.806	18.735	-4638.321	0.000	1761.521
4j [b]	5.659	-126.667	16.168	-3950.128	-0.070	1346.665
5i [a]	0.000	21.735	68.194	5106.804	0.000	0.000
5i [b]	-0.064	19.812	68.848	4876.429	0.182	21.522
5j [a]	0.000	-18.735	66.806	-4638.321	0.000	0.000
5j [b]	0.064	-16.812	66.152	-3966.752	-0.182	21.484
6i [a]	0.000	21.735	68.194	5106.804	0.000	0.000
6i [b]	-0.064	19.097	68.344	4519.915	-0.134	21.484
6j [a]	0.000	-18.735	66.806	-4638.321	0.000	0.000
6j [b]	0.064	-16.097	66.656	-3950.161	0.134	21.466
7i [a]	0.000	7.337	60.000	1761.521	0.000	0.000
7i [b]	0.071	6.140	60.236	1552.280	-0.061	-21.423
7j [a]	0.000	-7.337	60.000	-1761.521	0.000	0.000
7j [b]	-0.071	-6.140	59.764	-1410.874	0.061	-21.415
8i [a]	0.000	7.337	60.000	1761.521	0.000	0.000
8i [b]	0.071	5.595	59.988	1339.858	0.034	-21.414
8j [a]	0.000	-7.337	60.000	-1761.521	0.000	0.000
8j [b]	-0.071	-5.595	60.012	-1346.798	-0.034	-21.396

[a] contains the results of rigid frame
[b] contains the results of the semi-rigid frame

5.3 Three-storey, 63-member space frame

63-member frame that is applied 11 kN/m constant loads through beams in the direction of gravity is selected demonstrate the design application the program MRVSSF. The configuration and numbering of frame is shown in Fig. 7. The lengths of the columns and beams are 3.2 m and 5 m, respectively. The analysis results of the random members are given as example to show the differences between semi-rigid and rigid connections.

The absolute end-moment results of the random selected member final designs for semi-rigid and rigid connections are given in Table 3. The end moments in XZ plane generally decrease (16.9% in total) in beams and increase (15.4% in total) in columns, in XY plane increase all members when compared to frame with rigid connection.

The end-rotation results of the selected member final designs for semi-rigid and rigid connections are given in Table 4. The end-rotation in XZ plane generally increase (40.6% in total) for all members when compared to frame with rigid connection. In XY plane, while the end-rotation of members is zero for rigidity connection, the values of the member end-rotation are obtained for semi-rigid connections.

The results of the selected member final design for the both of semi-rigid connections and rigid connections are given in Table 5. The results of all member design show that 8.5% ligther frame

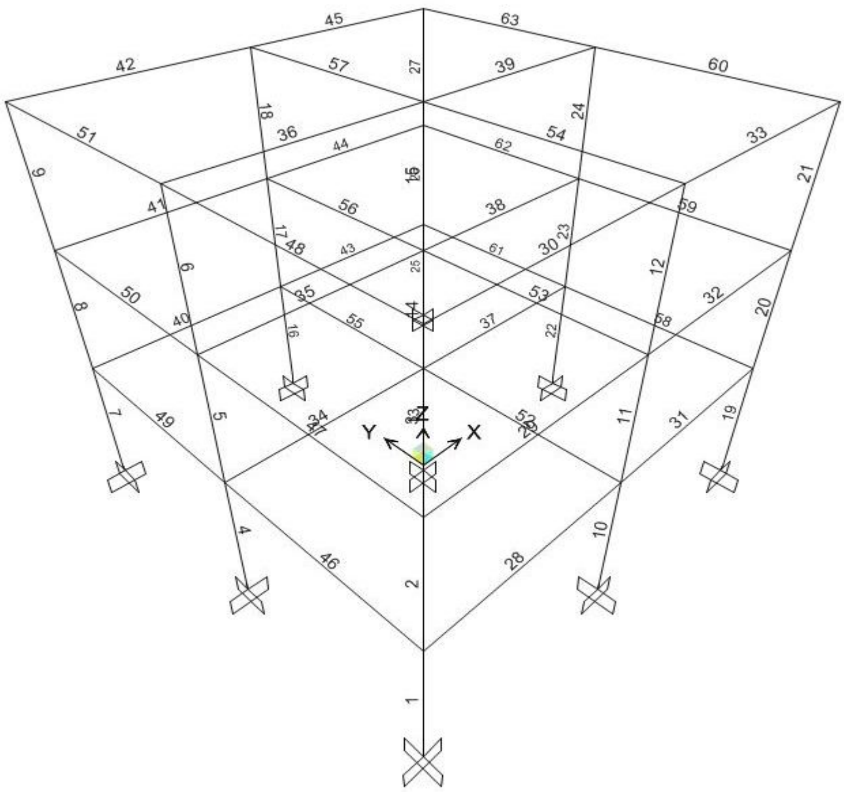


Fig. 7 Three-storey, 63-member space frame

Table 3 Absolute end-moment of random selected members in three-storey 63-member space frame for semi-rigid connection and rigid connection

Member No.	Moment (kNcm)							
	Rigid connection				Semi-rigid connection			
	XZ Plane		XY Plane		XZ Plane		XY Plane	
	i point	*j* point	*i* point	*j* point	*i* point	*j* point	*i* point	*j* point
1	394.15	788.93	-80.98	-162.00	430.45	960.40	-199.02	-248.90
2	496.68	592.31	-88.40	-90.32	597.68	770.50	-157.75	-183.32
4	523.28	1045.50	0.00	0.00	373.23	923.47	16.42	17.26
5	787.11	845.19	0.00	0.00	933.76	1005.72	3.31	5.04
6	682.37	919.56	0.00	0.00	637.62	918.82	28.13	28.57
19	-394.15	-788.93	-80.98	-62.00	-489.50	-880.50	-114.76	-244.85
20	-496.68	-592.31	-88.40	-90.32	-601.97	-701.07	-156.09	-183.05
21	-523.86	-595.81	-63.68	-64.93	-566.63	-728.77	-100.02	-103.64
32	2822.05	-1116.35	0.05	0.04	2052.82	-1267.80	2.62	1.54
42	595.92	-3035.75	0.08	0.04	619.05	-2329.65	4.37	4.35
45	3035.75	-595.92	-0.04	-0.08	2325.86	-654.68	0.58	1.69

Table 3 Continued

Member No.	Moment (kNcm)							
	Rigid connection				Semi-rigid connection			
	XZ Plane		XY Plane		XZ Plane		XY Plane	
	i point	*j* point	*i* point	*j* point	*i* point	*j* point	*i* point	*j* point
46	250.48	-3274.24	-0.02	-0.03	406.70	-2459.47	-0.06	-0.04
49	3274.24	-250.48	0.03	0.02	2438.90	-373.38	0.14	0.07
60	64.95	-3321.57	-0.08	-0.12	103.65	-2557.61	-3.97	-4.34
63	3321.57	-64.95	0.12	0.08	2540.27	-92.00	-1.06	-1.69

Table 4 Absolute end-rotation of the random selected members in three-storey 63-member space frame for semi-rigid connection and rigid connection

Member No.	Rotation (rad*1000)							
	Rigid connection				Semi-rigid connection			
	XZ Plane		XY Plane		XZ Plane		XY Plane	
	i point	*j* point	*i* point	*j* point	*i* point	*j* point	*i* point	*j* point
1	0.00	-2.29	0.00	0.00	0.00	-2.47	0.00	-0.01
2	-2.29	-2.44	0.00	0.00	-2.47	-3.91	-0.01	0.05
4	0.00	0.00	0.00	0.00	0.00	0.01	0.00	-0.01
5	0.00	0.00	0.00	0.00	0.01	0.04	-0.01	0.03
6	0.00	0.00	0.00	0.00	0.04	0.07	0.03	0.03
19	0.00	-2.29	0.00	0.00	0.00	-2.48	0.00	0.07
20	-2.29	-2.44	0.00	0.00	-2.48	-3.99	0.01	-0.02
21	-2.44	-2.58	0.00	0.00	-3.99	-4.42	-0.02	0.06
32	-2.28	-2.44	0.00	0.00	-3.83	-3.99	0.02	-0.02
42	2.58	2.48	0.00	0.00	3.48	3.43	-0.03	-0.03
45	2.48	2.58	0.00	0.00	3.43	3.47	-0.03	0.02
46	-2.29	0.00	0.00	0.00	-2.47	0.01	-0.01	-0.01
49	0.00	2.29	0.00	0.00	0.01	2.91	-0.01	-0.01
60	-2.58	0.00	0.00	0.00	-4.42	0.11	0.01	0.05
63	0.00	2.58	0.00	0.00	0.11	3.47	0.00	0.02

with semi-rigid connection are obtained when compared to rigidly connected frame. While the weight of frame with semi-rigid connection is 81.37 kN, that of rigid frame is 88.64 kN.

5.4 Other frames

The configurations, loadings, weights and lightness ratio of the various frames with semi-rigid connection comparative with rigidly connected frames are shown in Table 6. The constant

Table 5 Final design sections of the random selected members in three-storey 63-member space frame for semi-rigid connection and rigid connection

Member No.	Final sections	
	Rigid connection	Semi-rigid connection
1	IPE240	IPE270
2	IPE180	IPE200
4	IPE300	IEP300
5	IPE240	IPE270
6	IPE200	IPE200
19	IPE240	IPE270
20	IPE180	IPE200
21	IPE160	IPE160
32	IPE270	IPE270
42	IPE270	IPE270
45	IPE270	IPE240
46	IPE270	IPE270
49	IPE270	IPE240
60	IPE270	IPE240
63	IPE270	IPE240

Table 6 Final design results of various frames for semi-rigid connection and rigid connection

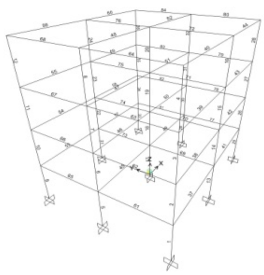
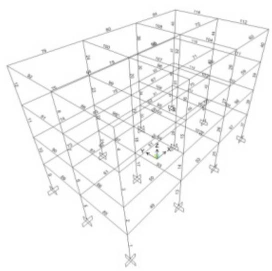
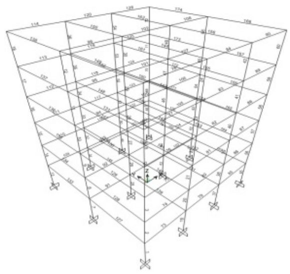
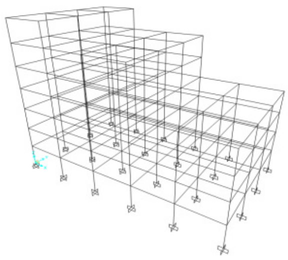
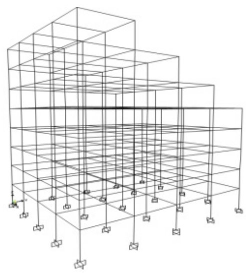
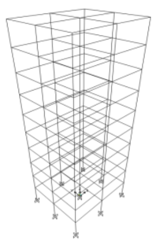
Frames		Load (kN/m)	Weigth (kN)		Weight reduction ratio
			Rigid connection	Semi-rigid connection	
4-storey 84-member		8.8	130.69	123.03	-5.86%
4-storey 116-member		8.8	190.34	175.26	-7.93%

Table 6 Continued

Frames		Load (kN/m)	Weigth (kN)		Weight reduction ratio
			Rigid connection	Semi-rigid connection	
6-storey 174-member		4.5	197.93	196.78	-0.58%
7-storey 346-member		4.5	500.62	491.09	-1.90%
8-storey 436-member		4.5	425.94	429.71	+0.89%
10-storey 210-member		4.5	284.62	291.76	+2.50%

loadings through the beams are applied in the direction of gravity. Generally, the frames with semi-rigid connections are lighter than the frames with rigid connections, but as the height of the building increases, economy of frames decrease. Especially, in frames higher than 8-storey, the weight of the frames with semi-rigid connection is greater than rigidly connected frame.

The design results of semi-rigid frames that is under same loading and has same number of story show that beam/column ratio increases, weight reduction ratio decreases according to the rigidity frame. The design results of semi-rigid frames that is under same loading and has about same beam/column ratio show that the number of story increases, economy decreases according to the rigidity frame.

6. Conclusions

A combined analysis and design procedure is presented for the design of space steel frames with semi-rigid connections accounting for the non-linear behavior of frames subjected to TS-EN648 (1980) specifications. An example was taken from literature (Değertekin and Hayalioglu 2004) was compared with the results of the literature (Değertekin and Hayalioglu 2004) to demonstrate the validity of the analysis procedure and various examples were undertaken to show the effect of connection flexibility on the space frame design.

In XZ plane of the space frame with semi-rigid connections, generally the end-moments of the beams decrease while the end-moments of columns increase when the results compared with the rigid connection modelling. The end-rotations of members of the semi-rigid frames generally increase according to rigid frames.

In XY plane of the space frame with rigid connections, there are no values of the moments and shear forces of the members due to no lateral loading. But the values are emerged in semi-rigid frames due to the force distribution.

While the sections of columns increase, the sections of beams decrease in the design results of frames with semi-rigid connection. So, generally the beams of frames provide the economy. As the story numbers of frame increase, increase in the sections of columns is more than decrease the sections of beams. For high-rise buildings, the weight of the frames with semi-rigid connection is greater than rigidly connected frame.

The semi-rigid connection design always could not result in more economical solutions. But this result indicates the importance of realistic connection modeling in the optimum design of steel frames. Therefore, neglecting the real behavior of the connection in the analysis may lead to unrealistic predictions of the response and reliability of steel frames.

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