# ANALYSIS AND DESIGN OF STEEL FRAMES WITH SEMI-RIGID STEEL CONNECTIONS AS PER IS 800:2007

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## ABSTRACT

**Globally, Structural steel design is based on the "Limit State Design Method". This method has proved economical for various structures constructed in the past. The philosophy of the limit state design method (LSM) represents a significant advancement over traditional design philosophies for steel structures. The Limit state design method has presented a new era for the safe and economic construction of steel structures. The new standard IS 800:2007 for the design of steel structures has provided an opportunity for modern design philosophy, design specifications, and provisions as per Limit State Method of Design in our country. For the design of steel structures, steel connections are important elements for controlling behaviour of the structure. It is essential that connectors develop full or a little higher strength compared to the members being joined for achieving an economical design.**

**Steel structures are analysed by considering ideally pinned or rigid end conditions for structural members like beam, column or bracings universally. At the end of members different types of steel connections like single web angle connections, double web angle connections, unstiffened seat angle, header plate connections etc. are designed for steel structures. These connections are provided for pinned end conditions where only shear force is resisted by the connections. End plate connections, Top and seat with web angle connections, T-Stub connections are provided and designed for rigid end conditions where shear force and bending moments both are to be resisted. These connections are not providing ideally pinned or ideally rigid end conditions in actual condition. Behaviour of the connections is in between two ideal conditions discussed over here. Connections in actual conditions provide some stiffness and behave as a semi rigid connections. Stiffness in connections is dependent on various parameters such as length of connectors, diameters of bolts, thickness of connectors etc. Such behaviour of connections is important to understand for safe and economical design of steel structures.**

**A analysis and design method has been employed for steel frames with semi-rigid connections using IS 800:2007 LIMIT STATE DESIGN METHOD in this paper. Analysis takes into account non-linear behaviour of beam-to-column connections. Frye and Morris polynomial model is used for modeling of semi-rigid steel connections as suggested in IS 800: 2007. The analysis and design of members in two storey one bay frame has been done by considering ideally rigid and ideally pinned end conditions using STAAD Pro.2006. Connections have been designed after evaluating end forces in members. Secant stiffness has been calculated for connection as per Frey and Morris polynomial modeling. The secant stiffness values have been incorporated in analysis instead of assumptions of ideally rigid or ideally pinned end conditions in the frame. Design of members has been conducted using the Indian standards provisions. The design process has been repeated for selecting member cross-sections and connection parameters. Economical solutions, have been evaluated using software. Design methodology for a frame with different types of connections is presented to demonstrate the efficiency of the method.**

**A comparison of design parameters has been made for different types of semi-rigid connections. Comparison is made for design parameters like end span bending moment, mid-span bending moment, shear force, axial force, weight of column, weight of beams, weight of frames, time period and top storey displacements. It has been concluded from the results that semi-rigid connection modeling provides more economical solutions compared to rigid or**

**pinned connection assumptions. It has been further observed that a change in stiffness of the connections is resulting in to economical solutions as well as variation in magnitude of sway for the frame.**

***Index Terms***— Limit State Design, steel structures, Semi-rigid Connections, Connection modeling, Secant stiffness

## 1. INTRODUCTION

Steel structures are made up of different elements like beams, columns, bracings, flooring and roofing systems. These elements are properly connected to form a composite unit known as steel structures. Beam-to-column connections are an integral element of a steel frame, and their behavior affects its overall performance of structure under loads. In common engineering practice, it is usually assumed that connections are either rigid or pinned. In reality, steel connections are not providing ideal rigid or pinned end conditions. Connections provide some flexibility in case of ideal rigid connections and gives rigidity in case of ideal pinned end conditions. The behaviour of connections which falls between two extreme end conditions like ideal pinned and rigid has been classified as semi-rigid steel connections.

IS 800 (Limit State Design) is a new era of safe and economical design for steel construction. The revised standard has enhanced the confidence of designers, engineers, contractors, technical institutions, professional bodies and industries for economical construction in steel [1].

Different types of semi-rigid connections incorporated in IS 800 [1] are as follows:

- Single web angle connections.
- Double web angle connections.
- Top and seat with web angel connections.
- Top and seat without web angel connections.
- End plate with column stiffener connections.
- End plate without column stiffener connections.
- T-Stub connectors.
- Header plate connections.

Most connections that connects beam to column using angles, plates, welds, and bolts are deformable and exhibit a nonlinear behavior between conditions fully fixed and perfectly pinned. It is more reliable to consider all connections as semi rigid, with rigid and pinned conditions being ideal cases. Connection flexibility affects both force distribution and deformation in beams and columns of the frame, and must be accounted for in a structural analysis [2].

In engineering practice, it is important to know in which cases connections are to be assumed as rigid, semi-rigid or flexible. For practical purposes, connections can be regarded as rigid and frame can designed as a rigid frame if the following limit is satisfied, Gerstle and ackroud [3].

$$EI_g / (R_k L) < 0.05 \quad \ldots (1)$$

In this equation, $EI_g$ is flexural rigidity of beam, L is length of beam, and $R_k$ is connection stiffness. Classification or large database on Moment-Rotation characteristics of various connections has been given in Euro Code 3 [4]. Kishi et al. have shown that single web angle connections, single plate connections, Double web angle connections, header plate connections should be classified as flexible connections and Top and Seat angle connections and flush end plate connections can be classified as semi rigid connections. Hasan et.al. have shown that extended end plate connections possessing initial stiffness ≥ 105.05 kNm/rad behave as rigid connections. All welded connections and extended end plate connections can be considered as rigid for design purposes. Arul Jayachandran et al. suggest that if joint stiffness is less than 0.001 kNm/rad, it may be assumed as hinged and if it is more than 1000 kNm it may be assumed as fixed [3].

To establish guidelines for design of semi rigid frames, it is necessary to know the m-θr behaviour of actual beam to column connections and to formulate appropriate m-$\theta_r$ model for use in analysis and design of semi-rigid frames. Though finite element studies have been used to study the connections behaviour Krishnamurthy et al. at present more used approach to describe m-θr curve is to curve-fit experimental data with simple expressions. Several analytical models have been developed to present flexibility of connections using available experimental data like linear, bilinear, piece-wise linear, cubic B-spline (jones at al. polynomial (Frye & Morris), exponential function and power function(Kishi & chen) [3].

A connection rotates through angle θr 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 modeling 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 [3] .

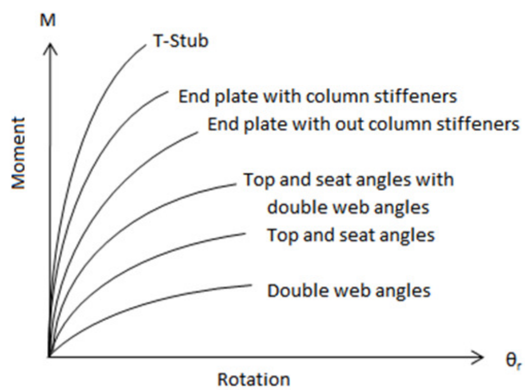



**Fig 1 Moment-rotation curves of semi-rigid connections [5]**

This paper aims to study the behaviour of steel frames by incorporating various types of semi rigid steel connections at the beam to column joints. In this paper analysis and design results like bending moments, shear forces, axial forces of members and top storey displacements, time period and weight of frames for ideal pinned and rigid connections have been compared. Such comparison has been worked out for semi rigid connections like Single web angle connection, Double web angel connections, Top and seat angel connections, End plate with column stiffener, T-Stub connectors and Header plate connections.

## 2. MODELLING OF SEMI RIGID CONNECTIONS

IS 800 [1] recommends Frye-Morris polynomial model to develop moment rotation's relationship curvature for modeling of semi-rigid steel connection. The Model is based on polynomial equation as follows:

$$\theta_r = C_1(KM)^1 + C_2(KM)^3 + C_3(KM)^5 \qquad \ldots 2$$

where K is a standardization parameter and is dependent upon connection type and geometry. $C_1$, $C_2$, $C_3$ are curve fitting constants. Fig 1 shows moment rotation curve of different types of connections. Curve fitting constant $C_1$, $C_2$, $C_3$ and standardization constant K for each type of connection have been given in table 1. Size parameters for various types of connections used in this paper have been shown in Fig. 2 [1].

Stiffness of different types of connections recommended by IS : 800 has been evaluated from the equations of standardized constants K as shown in Table 1. For different types of connections size parameters have been given in Table 1 for preliminary analysis using a bilinear Moment-Rotation relationship. The values have been evaluated based on secant stiffness at a rotation of 0.01radian. Typical dimension of connecting elements and other components have been evaluated.

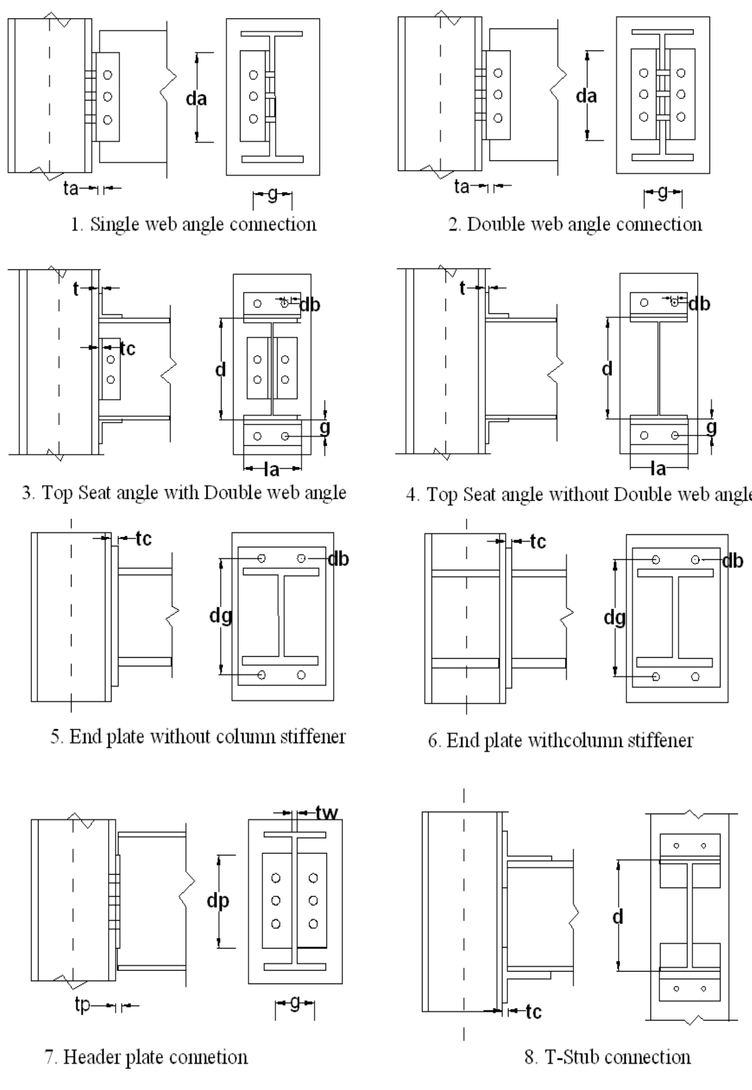



**Fig 2 Size parameters for various types of connections [1]**

Size parameters considered in modeling of connections influencing behaviour of connections have been presented in fig 2.

**Table 1 Connection constants in Frye-Morris Model [1]**

| No | Connection type | Curve fitting Constants | Standardization constants |
|---|---|---|---|
| 1 | Single web angle connection | $C_1=1.91 \times 10^4$<br>$C_2=1.3 \times 10^{11}$<br>$C_3=2.7 \times 10^{17}$ | $K= d_a^{-2.4} t_c^{-1.81} g^{0.15}$ |
| 2 | Header plate connection | $C_1=3.87$<br>$C_2=2.71 \times 10^5$<br>$C_3=6.06 \times 10^{11}$ | $K=d_p^{-2.3} t_p^{-1.6} g^{1.63} t_w^{-0.5}$ |
| 3 | Double web angle connection | $C_1=1.64 \times 10^3$<br>$C_2=1.03 \times 10^{14}$<br>$C_3=8.18 \times 10^{25}$ | $K= d_a^{-2.4} t_c^{-1.81} g^{0.15}$ |
| 4 | Top and seat angle connection | $C_1=1.63 \times 10^3$<br>$C_2=7.25 \times 10^{14}$<br>$C_3=3.31 \times 10^{23}$ | $K=d^{-1.5} t_a^{-0.5} l_a^{-0.7} d_b^{-1.1}$ |
| 5 | End plate connection with column stiffeners | $C_1=2.60 \times 10^2$<br>$C_2=5.37 \times 10^{11}$<br>$C_3=1.31 \times 10^{22}$ | $K=d_g^{-2.4} t_p^{-0.6}$ |
| 6 | T-Stub connection | $C_1=4.05 \times 10^2$<br>$C_2=4.45 \times 10^{13}$<br>$C_3=-2.03 \times 10^{23}$ | $K=d^{-1.5} t_a^{-0.5} l_t^{-0.7} d_b^{-1.1}$ |

Where, Nomenclature used in Table 1 is given as follows.
$d$ = depth of beam,
$d_a$ = depth of angle in mm,
$d_b$ = diameter of bolt in mm,
$d_g$ = center to center of outermost bolt of end plate connection, in mm
$g$ = guage distance of bolt line,
$t_a$ = thickness of top angle in mm
$t_c$ = thickness of web angle in mm,
$t_f$ = thickness of flange T-stub connector in mm
$t_w$ = thickness of web of the beam in connection in mm
$t_p$ = thickness of end plate and header plate in mm
$l_a$ = length of angle in mm
$l_t$ = length of T-stub connector in mm

## 3. ANALYSIS & DESIGN OF A FRAME CONSIDERING IDEAL RIGID OR PINNED END CONDITIONS

Analysis and design of a typical frame of a steel structure shown in fig 3(a) is conducted using STAAD Pro. 2006. The typical two-storey one-bay frame with dimensions, loading and numbering of members and nodes is shown in Fig.3(b). The connections are design as per IS 800 [1] and size parameters like length of angle, depth of plate, thickness of plate, diameter of bolts, gauge distance of bolts etc. have been evaluated from connection details for design of member end forces.

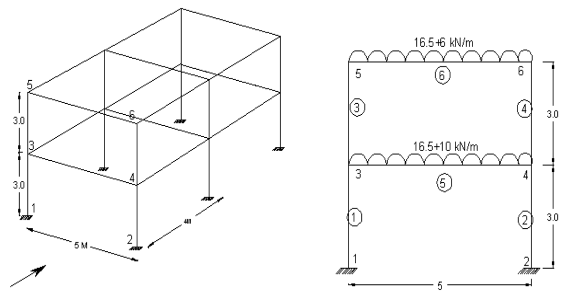



**Fig 3 (a). Typical Steel structure (b). Typical Steel Frame**

*3.1 Analysis of a frame:*

Details of loading considered for analysis of frame have been as follows.
Slab thickness is 125 mm,
Floor finish is 1.0 kN/m$^2$,
Live load = 2.5 kN/m$^2$ on typical flooring,
= 1.5 kN/m$^2$ on roof of frame.
Center to center spacing of frame is 4 m.
Dead Load of slab =3.125 kN/m$^2$
Dead Load of Floor finish = 1.0 kN/m$^2$
Total Dead Load = 4.125 kN/m$^2$
Total Dead Load per meter length of beam =16.5 kN/m
Live Load in typical frame per meter length of beam =10 kN/m
Live Load in typical frame per meter length of beam =6 kN/m

*Load Cases considered in analysis and design are as follows:*

1. Dead Load + Live Load
2. Earthquake Load

Analysis of frame is conducted by considering ideal pinned and rigid end conditions for beams of the frame. From frame analysis, governing forces like bending moment and shear forces at ends of beams have been evaluated for design of steel connections. Forces as shown in table 1 are maximum end forces at ends of beams for particular rigid or pinned end conditions. Connections are designed for this end forces.

**Table2: Shear force and Bending Moment summary at end of beam for connection design**

| End Condition | Bending Moment (kNm) | Shear Force (kN) |
|---|---|---|
| Rigid Frame | 187.14 | 74.85 |
| Pinned Frame | 0 | 66.19 |

3.2 *Design of connections*

Connections are designed for shear forces and bending moment values as shown in table 2 using IS 800 [1] provisions. Different types of shear bolted connections like single web angel connections, double web angel connections, top and seat angel connections and header plate connections are to be designed for 66.19 kN shear force. Moment resisting connections like End plate with column stiffener, T-Stub connections are to be designed for 74.85 kN Shear force and 187.14 kNm bending moment.

*3.2.1 Single angle web connection*

Shear force= 66.19 kN,

$f_y = 250$ N/mm$^2$, $\gamma_{m0} = 1.1$,

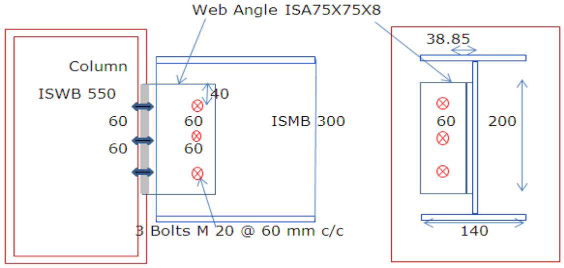



Fig 4 Details of single angle web connections

Evaluated size parameters from the connections design are shown in fig 4.
Depth of angle =200 mm, Thickness of angel = 8mm, Guage distance =39 mm

*3.2.2 Header plate connection*

Shear force = 66.19 kN

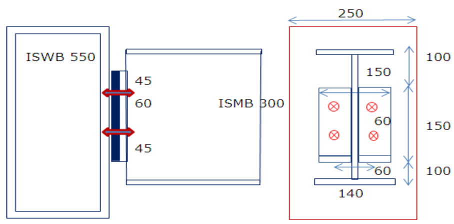



Fig 5 Details of Header plate connections

Size parameters from the connections design is as shown in fig 5.
Length of plate=150mm, width of plate=150mm, 4 no of m-20 bolts has been provided.

*3.2.3 Double angle web connection*

Following details are accomplished while designing double angle web connection taking shear force = 66.19 kN.

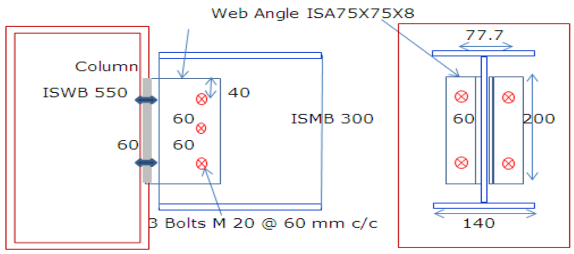



Fig 6 Details of double web angle connections

Size of elements has been shown in fig 6. Depth of angle =200 mm,
Thickness of angel = 8mm, guage distance =77.7 mm has been evaluated.

*3.2.4 Top and Seat angel connection*

Shear force = 66.19 kN.

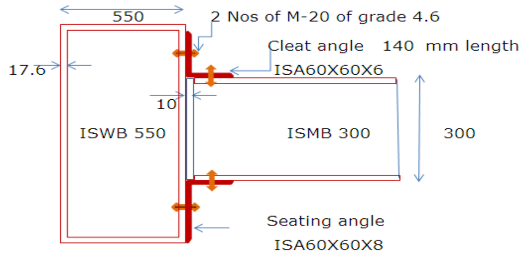



Fig 7 Details of top and seat angle connections

For safe design of connection required parameters has been shown in fig 7. Cleat angle and seat angel is 60X60X8mm, Length Cleat angel =140mm and 8 No of M-20 bolts.

*3.2.5 End plate connection*

Design of end plate connection for shear force = 74.85 kN and Bending Moment =187.14 kNm.

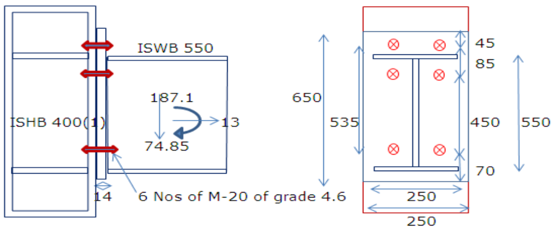



Fig 8 Details of end plate connections

Safe end plate connection details are shown in fig 8. Required thickness of plate =14mm and distance between two outer bolts is taken as 535mm.

*3.2.6 T-Stub connection*

T-Stub connection is design for shear force=74.85kN and bending moments =187.14kNm.

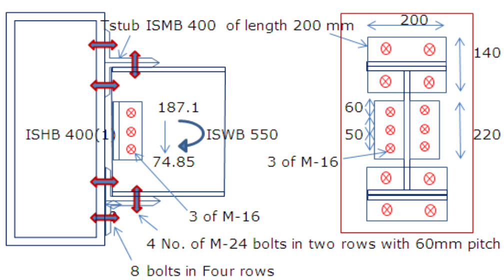



Fig 9 Details of T-Stub connections

In T–Stub connection size parameters considered in connection modelling has been shown in fig 9. Depth of beam = 550 mm, thickness of flange of beam =16 mm, length of T-Stub = 200mm and Diameter of bolts is taken as 24mm.

In problem of a typical frame as shown in fig 3(b) connections at node 3, 4, 5 and 6 have been designed by IS 800 [1]. For these connections secant stiffness has been evaluated using polynomial equation 2. Secant stiffness have been calculated from size parameters for all six types of semirigid steel connections. Moment rotation relationship curve is drawn based on polynomial Equation 2. From moment rotation curves, slope at 0.01 radian rotation has been selected. This slope at 0.01 radian gives the secant stiffness for a typical connection.

*3.3 Moment Rotation Curvature*

Several attempts have been made in past to study moment-rotation relationship. Extensive experiments have been conducted on commonly used connections, and a large collection of test data is reported (Kishi and Chen). Several mathematical models have been proposed to fit moment-rotation curves from experimental data. These models vary widely in their complexity ranging from the simplest linear model to polynomial, exponential, and power models [2].

Here, moment rotation relationship is developed using Frey-Morris polynomial modeling equation 2 for all connections which have been designed. Rotation values for given moments at the connections have been calculated. K is the standardized constant, formula of k is given in table 2. K depends on size parameters of the connections given in equations from table 1 [1]. Evaluated size parameters for connections which have been designed are as shown in Table 3.

*For a typical header plate connections, From Equation 2,*

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

Where, Curve fits from table 1 are

$C_1$=3.87, $C_2$=271000, $C_3$=6.06E+11

$$K = d_p^{-2.3}\, t_p^{-1.6}\, g^{1.6} t_w^{-0.5} \qquad \ldots 3$$

$K$= 1.418X10$^{-4,}$ from table 3 $d_a$=150, $t_p$=6, $g$=60, $t_w$=7.7

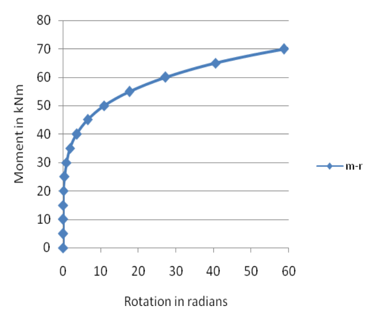



**Fig 10 Moment-rotation curves of Header plate connections**

The slope calculated at 0.01 radian rotation = 1001, which is the secant stiffness of header plate connections.
Secant stiffness is evaluated for different types of connections in similar manner are as given in table 3.

**Table 3 Secant Stiffness calculated considering semi rigid steel connections**

| No | Type of connection | Dimensions in (mm) | Secant stiff. (kNm/rad) |
|---|---|---|---|
| 1 | Single angle web connection | $d_a$=200, $t_a$=8, $g$=38.85 | 434 |
| 2 | Header plate connection | $d_p$=150, $t_p$=6, $g$=60, $t_w$=7.7 | 1001 |
| 3 | Double angle web connection | $d_a$=200, $t_a$=8, $g$=77.7 | 1890 |
| 4 | Top and seat angle connection without double web angle connection | $d_b$=300, $t_a$=6, $l_a$=140, $d_b$=20 | 2668 |
| 5 | End Plate with column stiffeners connection | $d_g$=535, $t_p$=14, | 7362 |
| 6 | T-Stub Connections | $d$=550, $t_f$=16, $l_t$=200, $d_g$=24 | 385854 |

## 4. ANALYSIS & DESIGN OF THE FRAME CONSIDERING SEMI RIGID END CONDITIONS

Analysis and design of the typical frame for the steel structure has been carried out again using STAAD Pro. 2006. Semi rigid end conditions have been introduced at ends of the beam this time. Secant stiffness has been computed for semi rigid connections.

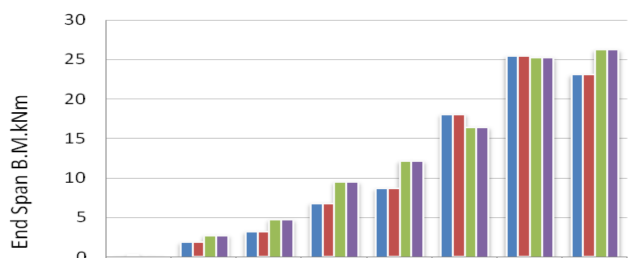
The comparison has been made for different parameters like

- End span bending moment.
- Mid span bending moment.
- Shear force.
- Axial forces in member.
- Nodal displacements of frame.
- Weight of the columns.
- Weight of beams.
- Total weight of frame.
- Top storey displacements.
- Time period of frames.

The comparison has been made for ideal pinned and rigid end conditions of steel frame with different types of semi rigid steel connections.

Variation of above parameters has been drawn from analysis results of frame using STAAD Pro.2006.

As can be observed from fig 3(b), members 1, 2, 3 and 4 are the columns of the typical frame and member 5 and 6 are the beams for the frame.




| | Pinned | 1 | 2 | 3 | 4 | 5 | 6 | Rigid |
|---|---|---|---|---|---|---|---|---|
| Column1 | 0.02 | 1.82 | 3.18 | 6.78 | 8.66 | 18.04 | 25.40 | 23.06 |
| Column2 | 0.26 | 1.82 | 3.18 | 6.78 | 8.66 | 18.04 | 25.40 | 23.06 |
| Column3 | 0.00 | 2.70 | 4.67 | 9.47 | 12.11 | 16.39 | 25.24 | 26.20 |
| Column4 | 0.00 | 2.70 | 4.67 | 9.47 | 12.11 | 16.39 | 25.24 | 26.20 |

Fig 11 Column End span B.M. (DL+LL)

For vertical load cases like DL+LL variation of end span bending moments in the column 1, 2, 3 and 4 has been presented fig 11. Increased in bending moment has been observed for different type of connections with increase in stiffness of frames. Increase in rigidity at the beam end conditions has resulted in to increase in stiffness of frame as a whole.

Variation of end span bending moment of columns for earthquake load cases has been given in fig 12. Decrease in bending moment has been seen with increase in rigidity of the frame. This indicates that columns are designed for governing bending moment for pinned end conditions for earthquake load cases. Column design is uneconomical for pinned end conditions where the earth quake load is governing.

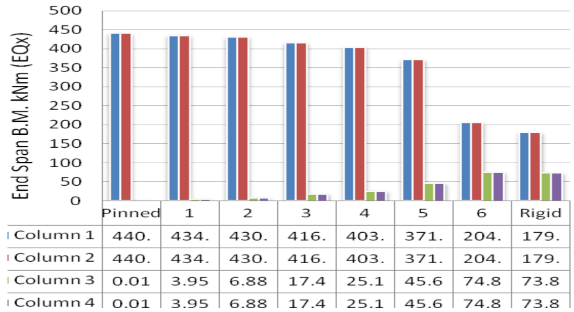



Fig 12 Column End span B.M. (EQx)

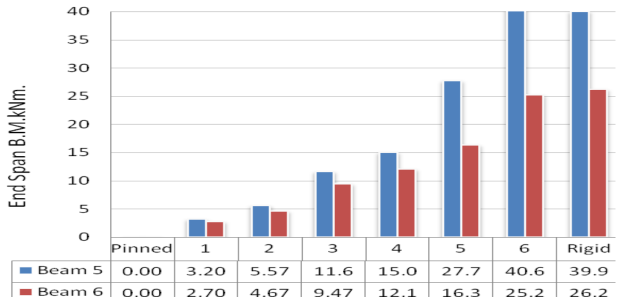



Fig 13 Beam end span bending moment in kNm (DL+LL)

Variation of end span bending moments for DL+LL load case is as shown in fig 13. Increase in bending moments is noted with an increase in rigidity. Maximum value is 40.6kNm is given in fig 13.

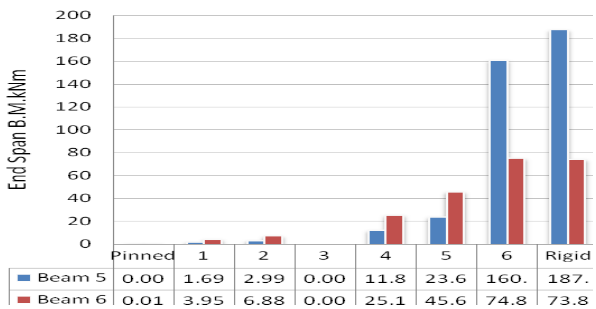



Fig 14 Beam end span bending moment in kNm (EQx)

For earthquake load case, end span bending moment value is 187.14 kNm as shown in fig 14. For this governing moment a connection has been designed.

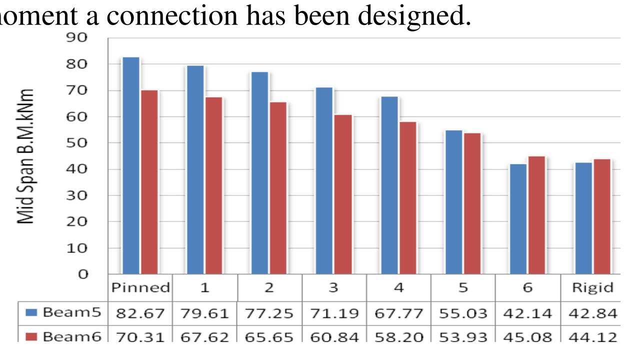
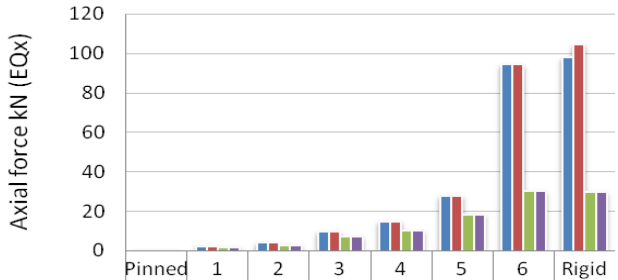



Fig 15 Mid span bending moments (DL+LL)

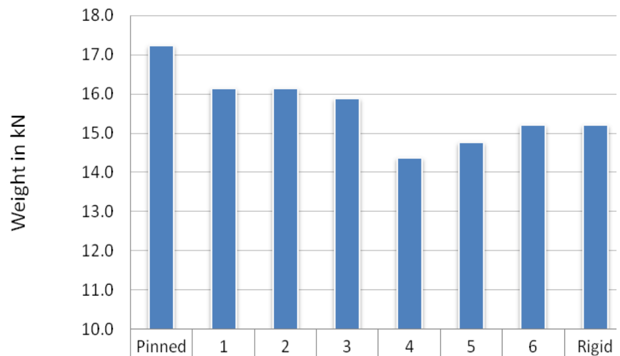
Decrease has been observed in mid span bending moment for beam with increase in connection rigidity as shown in fig 15.




| | Pinned | 1 | 2 | 3 | 4 | 5 | 6 | Rigid |
|---|---|---|---|---|---|---|---|---|
| Column1 | 0.05 | 2.26 | 3.94 | 9.53 | 14.7 | 27.6 | 94.1 | 97.6 |
| Column2 | 0.05 | 2.26 | 3.94 | 9.53 | 14.7 | 27.6 | 94.1 | 104. |
| Colum3 | 0.00 | 1.58 | 2.75 | 6.96 | 10.0 | 18.2 | 29.9 | 29.5 |
| Column4 | 0.00 | 1.58 | 2.75 | 6.96 | 10.0 | 18.2 | 29.9 | 29.5 |

Fig 16 Column axial forces (EQx)

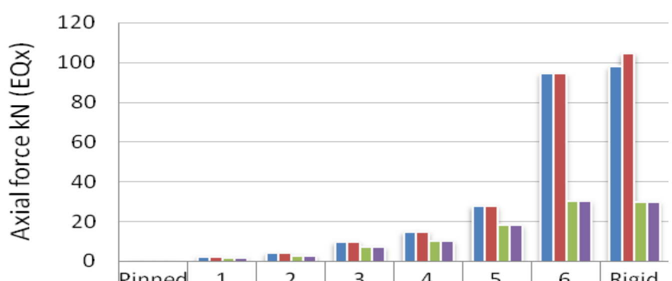
Column Axial forces remain constant in DL+LL load case. Increase in axial force has been noted with increase in connection rigidity for earthquake load case as shown in fig 16.




| | Pinned | 1 | 2 | 3 | 4 | 5 | 6 | Rigid |
|---|---|---|---|---|---|---|---|---|
| Total Weight | 17.23 | 16.14 | 16.14 | 15.88 | 14.38 | 14.78 | 15.22 | 15.22 |

Fig 17 Column Shear forces (DL+LL)

Column shear force increases with rigidity for DL+LL load cases and remain constant for earthquake load cases as shown in fig 17.




| | Pinned | 1 | 2 | 3 | 4 | 5 | 6 | Rigid |
|---|---|---|---|---|---|---|---|---|
| Series1 | 0.05 | 2.26 | 3.94 | 9.53 | 14.7 | 27.6 | 94.1 | 97.6 |
| Series2 | 0.05 | 2.26 | 3.94 | 9.53 | 14.7 | 27.6 | 94.1 | 104. |
| Series3 | 0.00 | 1.58 | 2.75 | 6.96 | 10.0 | 18.2 | 29.9 | 29.5 |
| Series4 | 0.00 | 1.58 | 2.75 | 6.96 | 10.0 | 18.2 | 29.9 | 29.5 |

Fig 18 Beam Shear forces (EQx)

Increase in shear forces has been observed with increase in rigidity for earthquake load cases as shown in fig 18. Shear force vary from 0 kN for pinned end condition to 74.8kN for rigid end conditions. Moment resisting connections are designed for 74.8kN shear force.

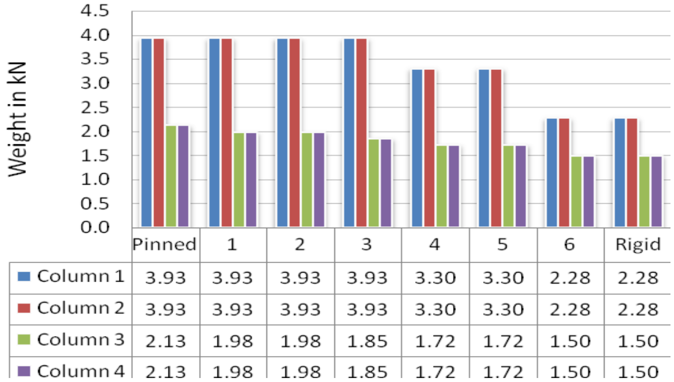



Fig 19 Column weight in kN

Column weight decrease with increase in rigidity of the end conditions for beam. Design of column is carried out for governing bending moments. Governing value of bending moment has been noted in fig 12 is 440kNm significantly higher in pinned end condition than the value of value of DL+LL case which is 26.20 kNm as observed in fig 11.

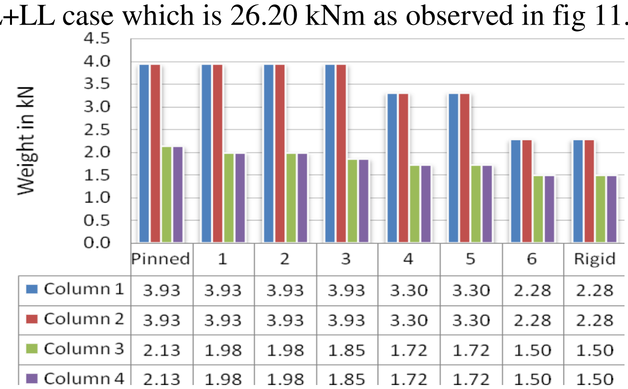



Fig 20 Beam weight in kN

Increase in beam weight is observed for both pinned and rigid end conditions. Weight decreases for semi rigid end conditions as shown in fig 20.

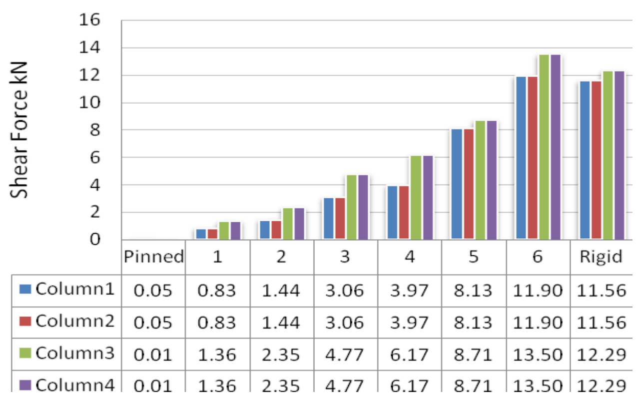



Fig 21 Total weight of frame in kN

Total weight of frame has been varies as observed in fig 21. For different types of connections weight varies from 14.38kN for top and a seat angle connection to maximum of 16.14kN for single web angle connections. Weight of rigid frame is of 15.22kN instead of 17.23kN for pinned end conditions.

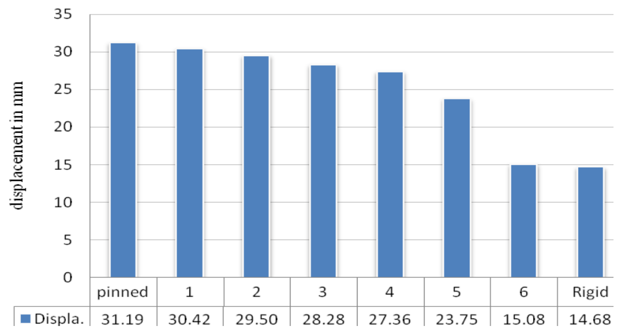



Fig 22 Top storey displacement of frame in mm

Top storey displacements of flexible frame are higher for frame with pinned end condition frame as can be observed from fig 22.

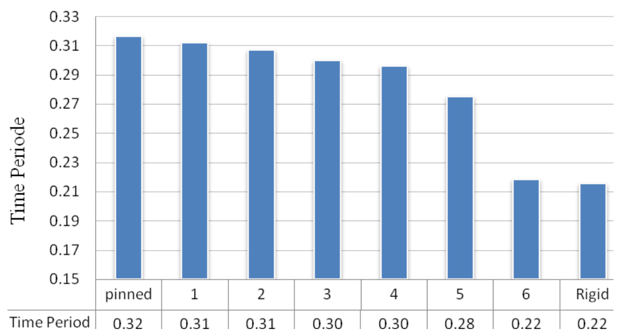



Fig 23 Time period of frame in second

Time period varies for different types of connections has been observed from fig 23 for different semirigid end condition of frame. Results obtained from analysis of different types of semi rigid steel frame have been given in fig 23. It has been observed that time period increases with decrease in rigidity of the frame.

## 5. CONCLUDING REMEARKS

Analysis and design has been conducted for the steel frame with semi rigid connections accounting non-linear behaviour of the frame. Software based analysis and design procedure of repetitive nature has been carried out for safe and economical sections. Design examples have been included to demonstrate influence of flexibility on the connection of steel frames.

- From the results of the design example it has been observed that end span moments in beam are increase enhances with increase in rigidity of end conditions of beam for resistance against vertical loads.
- It can be seen from results of column shows that end span moments in column increase with increase in rigidity of end conditions of beam for vertical load cases like DL+LL.
- For horizontal load, end span moments increase in beam with increase in connection rigidity. Reduction has been observed in column with increase in rigidity of end conditions for the beam.
- Mid span moments in beam and column has been reduce with increase in rigidity of end conditions of beam for vertical load cases.
- From comparison of column axial force it can be noted that axial force increased with increase in rigidity for horizontal load cases. Axial force in column remains constant for vertical load cases.

- Shear force in columns increase with increase in rigidity for vertical load cases. For beams, enhancement has been observed with increase in rigidity for horizontal load cases.
- Column weight decreases with increase in rigidity of  the frame.
- Weight of beam decreases with provision of semi rigid connection.
- It can be seen from design example that weight of frame varies for different types of semi rigid connections.
- Time period and top storey displacement increase with increase in flexibility of semi rigid connection types used for the frame.

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