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COMPARATIVE STUDY OF MULTISTOREY STEEL SPACE FRAME WITH RIGID AND SEMI RIGID BEAM-COLUMN CONNECTION

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Abstract - *The purpose of this paper is to present the interaction of the steel frames and their joints and to describe and propose an acceptable method of beam column joint design on the basis of FEM software. A key problem in dealing with joints is their classification, the basis of which is described in the Eurocodes, IS code and other available design codes in a variety of different ways. Eurocodes take into account whether the joint is applied within a frame with fixed nodes or within one with sway (semi rigid or hinged) nodes. Engineering methods help us with establishing the load-displacement diagrams of frames by using simple techniques, in a way that local "softening" effects occurring in the vicinity of joints can also be taken into account. Engineering design is an activity of fair complexity, thus it is important to establish direct design methods which, while simple, take into consideration certain complex phenomena such as the stiffness and strength properties of the joints (including beam-to-column joints The results on the basis of large scale experimental tests under both monotonic and cyclic loading had already been computed. So for economic design, to obtain accurate results and due to time constrain, comparative study of rigid and semi rigid connection design is done by Dlubal RFEM software on a multistorey space frame.*

Key words: Semi rigid connection, Moment-rotation relationship, nonlinear analysis, Dlubal RFEM, Steel space frame design

1. INTRODUCTION

One of the basic consideration of structural analysis is that joints are fully rigid or fully hinged. However typical connections in actual structure does not behave either in fully rigid or fully hinged manner. The different types of connections, commonly used, fill the entire flexibility spectrum from flexible connections to rigid connections. To obtain actual behavior of steel structures by applying static and dynamic loads has a prior importance in civil engineering field. The actual behavior of a steel structure can be provided by determining the geometrical, damping, mass and connection properties of the existing structure. In the design stage, supports and beam-to-column connections in steel structures are assumed as fully rigid. However, these connections are not actually fully rigid. The constitution of an almost fully rigid connection is also impractical and economically unjustifiable in most cases. Practically, connections in steel structural frames are semi-rigid, and consequently, the internal forces and bending moment diagrams constructed under the assumption of rigid or pinned joints contain considerable errors. Designs based on these results also lead to an inappropriate sizing of the members. Various design codes for steel buildings such as the AISC Load Resistance Factor Design and the British Standard 5950, Euro code (EN1990+EN1997 CEN), IS code (800-2007) allows accounting for the effects of joint flexibility in the analyses.

For semi rigid connection, several mathematical models were proposed for obtaining moment- rotation values or curves i.e. linear semi rigid connection and nonlinear semi rigid connection model. In linear semi-rigid connection models, the stiffness of connections is assumed to be constant and stiffness matrix of a beam–column member is usually modified by using end-fixity factors. The advantage of these models is simple in formulation and implementation.

So far no remarkable work is done yet for the development of manual procedure of the design of semi rigid connection in steel structure. According to required design procedure multiple iterations may be necessary to reach final design results. Secondly, acceptable deformation and target displacement are not directly considered when sizing the beam column components of a steel structure. This is because the structural strength and structural stiffness need to be satisfied. Hence it is very challenging to design a semi rigid frame to meet the target displacement requirement and attainment of desirable plastic deformation. To resolve this problem, new procedure or methodology on the basis of finite element based design is urgently needed to obtain semi rigid connection design parameters such as moment-rotation values with respect to beam column joint.

There currently exist some of the methods and computer aided programs based on nonlinear inelastic analysis by finite element method of frame structures with rigid and semi rigid connection. All these available computer programs for such advanced analysis has the ability of fine grained modeling, extensive calibration and mesh generation studies that are often impossible for the manual design ability.

1.1 Necessity and Current difficulties:

It appears that there are several interlinked obstacles that prevents todays designer of steel structures from embracing a semi rigid connection philosophy. A general listing of these concerns includes:

1. Utilization classification uncertainties
2. Need for reliable moment –rotation model
3. Efficient analysis methods
4. Serviceability and stability concern
5. Value engineering and professional concern

2. OBJECTIVES

The objective of present project mainly encompasses the comparison between Multistorey steel space frame (MSSF) with rigid and semi rigid connection separately, the objectives are sequentially mentioned below

- To design and analyse MSSF (G+11) with rigid beam-column connection by linear as well as nonlinear analysis.
- To design semi rigid connection for the MSSF designed earlier.
- Calculation of M-▯ parameters after designing semi rigid beam column connection in RF-Frame joint pro module provided by Dlubal RFEM.
- Assigning those M-▯ values with respect to every beam column joint nodes of other similar MSSF to obtain semi rigid behavior of joint.
- Analyse and design MSSF with semi rigid connection by linear as well as nonlinear analysis.
- Comparative study of internal forces such as end moment generated in both cases and to find extent of moment reduction at each beam column joint node.

3. METHODOLOGY

The following comparative study of multistorey steel space frame(MSSF) design with rigid and semi rigid connections separately is performed with respect to the design standards of EN 1990 + EN 1997(National Annex CEN-EU).Design of MSSF by manual calculation is a tedious work as well as time consuming. The accuracy provided by the manual procedure is limited as multiple iterations needs to be done to arrive at design parameters. The design steps which are tedious for manual calculation are mentioned below:

- Linear as well as nonlinear analysis.
- Formation and calculation of 3D solid FE stiffness matrices.
- Formation and calculation of 2D surface FE stiffness matrices.
- Formation and calculation of 1D member FE stiffness matrices.
- Formation of global stiffness matrices.

The analysis and design of the structure is done in FEM software DLUBAL RFEM. The software provides the detailed semi rigid connection design with the classification of connection only as per EN 1990 + EN 1997. Dlubal RFEM software is used for design and analysis because it is more advanced and modified in semi rigid connection design as compared to other software's. It provides RF Frame joint pro connection design modules.

Two models such as steel space frame with rigid and semi rigid beam column connection are considered for the comparative study. The analysis and design of both the cases using rigid and semi rigid connection is mentioned below and analysis and design differences are computed respectively.

Case I

Design of Multistorey steel space frame using rigid connection

3.1 Loading calculation:

Dead load (Self weight): The self-weight of construction works should be classified as a permanent fixed action, as per EN 1990, 1.5.3 and 4.1.1. DL considered as -4.5 KN/M^2 (Z direction).

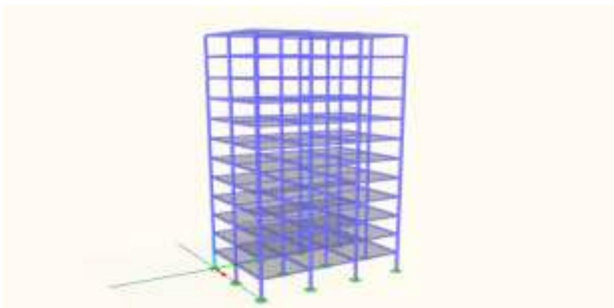
Live load: Live load should be classified as variable free actions, unless otherwise specified in this standard as per EN 1990-1-7

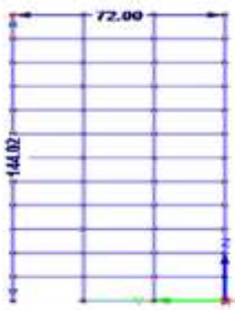
Live load (on Floor) = -4.5 KN/M^2 (Z direction)

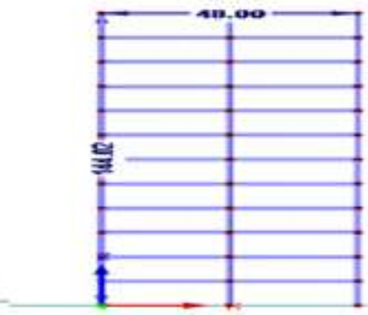
Live load (on roof) = -3 KN/M^2 (Z direction)

Wind load: Wind load (Area load) = 1.65 KN/m^2, calculation as per BS EN 1991-1-4:2005+A1:2010 (Y direction)

3.2 Modeling







3.3 Design considerations and details:

Type of structure: Steel structure (G+11)

C/s of members: Rolled I Section

Beam column connection: Rigid, Semi rigid

Slab: Concrete f_c =4000psi (ACI 318-14, As FE Mesh)

Analysis: Linear and nonlinear (Behavioral comparison of MSSF with rigid and semi rigid connections separately)

Nodal support (Base): Fixed (Fully Rigid)

Dead load and imposed load: Surface loads

Wind load: Area load

3.4 Load combination

Load Case	Load Case Description	EN 1990 + 1997 | CEN Action Category
LC1	Self-weight	Permanent
LC2	Live load	Imposed - Category A: domestic, residential areas
LC3	live load	Imposed - Category H: roofs
LC4	Wind load	Wind

Comb.	Description
CO1	1.35*LC1
CO2	1.35*LC1 + 1.5*LC2
CO3	1.35*LC1 + 1.5*LC2 + 0.9*LC4
CO4	1.35*LC1 + 1.5*LC3
CO5	1.35*LC1 + 1.05*LC2 + 1.5*LC3
CO6	1.35*LC1 + 1.05*LC2 + 1.5*LC3 + 0.9*LC4
CO7	1.35*LC1 + 1.5*LC3 + 0.9*LC4
CO8	1.35*LC1 + 1.5*LC4
CO9	1.35*LC1 + 1.05*LC2 + 1.5*LC4
CO10	LC1
CO11	LC1 + LC2
CO12	LC1 + LC2 + 0.6*LC4
CO13	LC1 + LC3
CO14	LC1 + 0.7*LC2 + LC3
CO15	LC1 + 0.7*LC2 + LC3 + 0.6*LC4

Comb.	Description
CO16	LC1 + LC3 + 0.6*LC4
CO17	LC1 + LC4
CO18	LC1 + 0.7*LC2 + LC4
CO19	LC1
CO20	LC1 + 0.5*LC2
CO21	LC1 + 0.3*LC2 + 0*LC3
CO22	LC1 + 0.2*LC4
CO23	LC1 + 0.3*LC2 + 0.2*LC4
CO24	LC1
CO25	LC1 + 0.3*LC2

3.5 Result combination

Result Combin.	Result Combination Description
RC1	ULS(STR/GEO-2), Approach 2 – Permanent/transient – Eq. 6.10
RC2	SLS - Characteristic
RC3	SLS - Frequent
RC4	SLS - Quasi-permanent

3.6 Designed section details:

Section No.	Cross-Section Description [m]	Location, Member
1	HD 400x382 | ArcelorMittal (EN 10365:2017)	G, 1, 2 floor column
2	IPE O 330 | ArcelorMittal (EN 10365:2017)	All floor beams
3	IPE O 450 | ArcelorMittal (EN 10365:2017)	
4	IPE O 550 | ArcelorMittal (EN 10365:2017)	
5	HD 400x347 | ArcelorMittal (EN 10365:2017)	3, 4, 5 floor column
6	HD 400x314 | ArcelorMittal (EN 10365:2017)	6, 7, 8 floor column
8	HD 400x216 | ArcelorMittal (EN 10365:2017)	9, 10, 11 floor column

Case II

Design of Multistorey steel space frame using Semi rigid connection

For the design of MSSF using semi rigid connection, the model designed for MSSF rigid connection is imported and further connection design is carried out as follows,

3.7 Semi rigid beam-column connection design

DLUBAL RFEM software provides a module i.e. RF Frame joint pro for the connection design as per the geometry of connection. After the analysis and design of connection, software recommends to provide semi rigid connection, if possible.

On the basis of analysis and design carried out, Design details are obtained such as,

- Stiffness class and lateral bracing.
- Negative loading moment and negative ultimate moment i.e. M_D, M_{Rd}
- Limiting stiffness for rigid and pinned behavior i.e. $S_{J\text{-Limit rigid}}$, $S_{J\text{-Limit pinned}}$
- Initial stiffness S_{JIni}, applicable rotational stiffness, effective stiffness coefficient.

Manual calculation is done using above parameters and M- values are obtained.

Data mentioned for bottom 4 storey only,

Storey	Connection location	Ultimate moment (M_{rd}) KN-m	Rotation () Degree
4	Corner	-243.50	0.001565
	Longer edge	-481.71	0.001221
	Shorter edge	-9.06	0.0000432

	Cross	-321.40	0.001925
3	Corner	-321.40	0.001925
	Longer edge	-441.33	0.001562
	Shorter edge	-19.95	0.0001828
	Cross	-356.95	0.002418
2	Corner	-321.40	0.001925
	Longer edge	-441.33	0.001562
	Shorter edge	-19.95	0.0001828
	Cross	-356.95	0.002418
1	Corner	-321.40	0.001925
	Longer edge	-441.33	0.001562
	Shorter edge	-19.95	0.0001828
	Cross	-356.95	0.002418

These M-⍰ values are further required to apply at end nodes of beams-column joint so that it will perform in a semi rigid manner. Same M-⍰ parameters are provided in all directions of the beam column joint as recommended by software.

4. RESULTS AND DISCUSSION:

In following tables, the net moments obtained for the design of multistorey steel space frame are tabulated and comparison between moments for rigid and semi rigid space frame is made separately for beams as well as columns.

For columns, a floor above which cross section of members are varying is considered for the moment results shown below.

Floors considered: 2nd

End moments: Both ends (0, 3.658)

Cases: M_Y max, M_Z max

For beams,

Floors considered: 6th, 12th

End moments: Both ends (0, 7.315)

Cases: M_Y max

4.1 2nd floor columns (Rigid connection):

Member no			**Moments**			**load case**
Column	From bottom (m)		M_T	M_Y	M_Z	
59	0	Max M_Y	-5.151	144.549	-432.9	C03
		Max M_Z	-5.435	73.594	-857.987	C09
	3.658	Max M_Y	4.253	-132.445	325.013	C06
		Max M_Z	5.36	-68.9	598.64	C09
60	0	Max M_Y	-9.209	264.792	-856.991	C03
		Max M_Z	-18.68	252.431	-1491.52	C09
	3.658	Max M_Y	9.906	-183.813	995.284	C08
		Max M_Z	18.907	-245.934	1274.29	C09
61	0	Max M_Y	-9.309	263.663	-833.255	C03
		Max M_Z	-18.7	250.525	-1470.04	C09
	3.658	Max M_Y	9.563	267.175	774.469	C03
		Max M_Z	19.889	-256.015	1253.09	C09
62	0	Max M_Y	0.024	107.128	-81.801	C02

		Max M_Z	-4.111	52.082	-923.037	CO9
	3.658	Max M_Y	0	-112.725	85.9	CO2
		Max M_Z	4.987	-62.28	682.5	CO9

4.2 2nd floor columns (Semi rigid connection):

Member no			**Moments**			**% Moment Reduction**	**load case**
Column	From bottom (m)		M_T	M_Y	M_Z		
59	0	Max M_Y	-1.963	106.06	-224.51		CO3
		Max M_Z	-2.978	101.76	-411.97	52	CO9
	3.658	Max M_Y	2.02	-111.76	237.2		CO3
		Max M_Z	3.058	-106.78	434.74	27.4	CO9
60	0	Max M_Y	-4.657	267.78	-573.37		CO3
		Max M_Z	-6.634	239.83	-901.56	39.5	CO9
	3.658	Max M_Y	4.866	-283.76	599.2		CO3
		Max M_Z	6.931	-254.11	942.86	26	CO9
61	0	Max M_Y	0	266	10.68		CO2
		Max M_Z	-6.49	234.17	-882.59	40	CO9
	3.658	Max M_Y	0	-282	-10.77		CO2
		Max M_Z	6.79	-248.44	923.85	26.25	CO9
62	0	Max M_Y	0	95.79	-81.28		CO2
		Max M_Z	-2.014	69.37	-552.41	40	CO9
	3.658	Max M_Y	0	-101.57	85.45		CO2
		Max M_Z	2.086	-74.32	588.88	13.6	CO9

4.3 6th floor beams (Rigid connection):

Member no			**Moments**			**Load case**
Beam	Location (m)		M_T	M_Y	M_Z	
161	0	Max M_Y	0	25.26	0	CO9
	7.315	Max M_Y	-0.137	-361.352	0	CO9
162	0	Max M_Y	0.139	368.06	0	CO9
	7.315	Max M_Y	-0.133	-358.326	0	CO9
163	0	Max M_Y	0.137	368.035	0	CO9
	7.315	Max M_Y	0	-48.535	0	CO9
164	0	Max M_Y	-0.198	369.6	0	CO8
	7.315	Max M_Y	-0.68	-548.14	0.339	CO9

4.4 6th floor beams (Semi rigid connection):

Note: if allowed value of is negligible then that joint is treated as rigid i.e. no moment reduction will take place at corresponding beam-column members as they shows stress concentration (Not safe), which further results in large moment decrease after formation of plastic hinges at end supports.

Member no	Location (m)		Moments			% Moment reduction	Load case
Beam			M_T	M_Y	M_Z		
161	0	Max M_Y	0	387.94	0	RIGID	CO8
	7.315	Max M_Y	0	-746.62	0	51.60162	CO9
162	0	Max M_Y	0	467.36	0	21.247	CO9
	7.315	Max M_Y	0	-732.09	0	51.05438	CO9
163	0	Max M_Y	0	456.94	0	19.4566	CO8
	7.315	Max M_Y	0	-949.19	0	RIGID	CO9
164	0	Max M_Y	-0.198	369.6	0	26.41775	CO8
	7.315	Max M_Y	-0.68	-548.14	0.339	4.834531	CO9

5. CONCLUSION AND FUTURE SCOPE:

An accurate and efficient procedure has been proposed for evaluating the system reliability of Steel frames with semi rigid connections. The proposed procedure utilizes a refined plastic hinge element and efficiently predict reduction in end moments. It is therefore important to account for the semi rigid behavior of beam to column connections and the assumption of rigid connections commonly used in the analysis and design of steel frames is impractical. The serviceability could be considered as governing limit state when semi rigid connections are considered.

On the basis of results obtained, at ground, 1st, 2nd, 3rd and 4th floor, expected end moment decrease took place due to the provision of semi rigid beam column connection. It proves that all the connections up to 4th floor could be provided as a semi rigid connection. Above 4th floor, connections are not entirely semi rigid. Some connections do possess rigid behavior (rotation value is negligible). So, semi rigid connection provision (wherever permitted) as per results will reflect in economic multistorey steel space frame design.

FEM software based design is most widely used alternative of obtaining the mechanical behavior of connection

- This methodology helps in overcoming the lack of experimental results.
- To understand important local effects which are difficult to measure with sufficient accuracy i.e.to generate extensive parametric study.
- To allow the introduction of deformation and displacement into the model as plasticity, strain hardening, instability effects, contact between plates and prestressing of bolts.
- Mainly the comparative study represents the reduction in the member end moments of steel space frame by the provision of partially reinforced connections in terms of M-θ parameters and expected moment reduction is achieved so that it will reflect in achieving economy in terms of reduced beam column cross section.

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