A Parametric Study on the Influence of Semi-Rigid Connection Nonlinearity on Steel Special Moment Frames

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Abstract

Challenges exist in modeling an accurate nonlinear behavior of a steel structure, where one of these is the modeling of semi-rigid behavior of connections. A detailed finite element model would take into account the complex interaction between all surfaces due to contact, friction and bolt pretension besides the material and geometric nonlinearity effects. All these nonlinearities could be simply lumped as a moment-rotation type model at the connection region. Such a methodology is followed in this paper, and the main aim is to study the semi-rigid connection’s lumped nonlinear behavior on the overall structural response of low-rise, mid-rise and high-rise steel special moment resisting frames. In linear elastic analysis of steel structures, it is not possible to estimate the energy absorption capacity of members and the structure. Such estimation would be possible in nonlinear analysis, and insertion of the semi-rigid behavior of connections into the analysis would give even more accurate and realistic results in this regards. After the investigation of moment-rotation characteristics of semi-rigid steel connections, they are included in the nonlinear structural analysis of steel special moment frames (SMFs). Three, nine and fifteen story SMFs are analyzed and designed as rigid frames, then the behaviors of the frames are reanalyzed considering nonlinearity in semi-rigid connections. A parametric study is conducted through changing span lengths and connections’ rigidities and plastic moment capacities. Changes in the ductility and overstrength reduction factors of SMFs obtained from pushover curves are compared between the rigid and the semi-rigid modeling alternatives.

Keywords: *Semi-rigid connection; steel special moment frame; nonlinear structural analysis; pushover analysis*

1 Introduction

According to the general practice of structural engineering, the design of steel structures is carried out by assuming the connections as hinged or fully rigid. However, the real behavior of the connection is more complex and can be represented more accurately by using semi-rigid connections that form a third class of connections based on their moment-rotation characteristics. In semi-rigid framed structures, there are non-negligible rotations and moment resistance at the beam to column connection regions, and furthermore, not only the linear but also the nonlinear behavior of the connections may have a significant effect on the structural analysis results.

Increasing attention has been given to nonlinear modeling of especially steel structures, and research in this area has been growing as dependent on the development of computer technology. Modeling of steel's nonlinear behavior has gained significant attention due to the fact that steel is an isotropic material, and has definite elastic properties, ductile behavior and ability to dissipate energy efficiently. Both member level and system level nonlinear analyses can be conducted, and several such studies are available in literature. The following two research studies by Maison and Kasai (2000) and Kim and Choi (2005) are taken into account in the parametric study undertaken in this paper. The study carried out by Kim and Choi (2005) focused on the overstrength, ductility and the response modification factors of special braced frames and ordinary concentric braced frames. The study tried to compare the suggested response modification factors or force reduction factors by the codes with the results of a group of parametric structural analysis results. In another study carried out by Maison and Kasai (2000), low-rise and mid-rise steel moment frames with semi-rigid connections were analyzed under the same earthquake loadings. Firstly, the buildings were designed with fully restrained (FR) connections and then they were redesigned with partially restrained (PR) connections. Natural periods and total girder/column weights were compared. Total base shear versus roof drift ratios were also evaluated.

The study in this paper focuses on the effects of semi-rigid connections on a subgroup of steel moment frames as defined by ANSI/AISC 360-10 Specification for Structural Steel Buildings. This study tries to identify the effects of semi-rigid connections on the steel special moment resisting frame (SMF) systems of different properties by following the analysis methodologies described by above cited references. Firstly, rigid SMF buildings are designed according to the relevant codes. Then, semi-rigid connections are assigned to these rigidly designed frame connection regions. By carrying out nonlinear static pushover analyses, the rigid and semi-rigid frame behaviors and responses are assessed and various response measures such as ductility and overstrength reduction factors obtained from both models are compared.

2 Steel Special Moment Frame Analysis and Design Case Studies

In the scope of the study, three main groups of steel special moment frames (SMFs) were selected to be analyzed and designed. They are grouped as 3, 9 and 15-story steel SMFs in order to cover low-rise, mid-rise and high-rise buildings. Then, a second main parameter was added to the geometries of these 3 main groups of frames to study the effects of bay lengths of frames. Figure 1, which is common for all frame groups, shows top view of the buildings, i.e. all selected frames are the members of this top view of building that is surrounded by a steel SMF at its outer perimeter and has gravity beams on each floor level to carry out vertical loads of frames. The selected buildings are all office type buildings. As indicated previously, by considering the bay length parameter, there exits 6 main groups of frames, which have the following general characteristics:

- 3-story, 3-bay SMF with L_{bay}=6m
- 3-story, 3-bay SMF with L_{bay}=10m
- 9-story, 3-bay SMF with L_{bay}=6m
- 9-story, 3-bay SMF with L_{bay}=10m
- 15-story, 3-bay SMF with L_{bay}=6m
- 15-story, 3-bay SMF with L_{bay}=10m

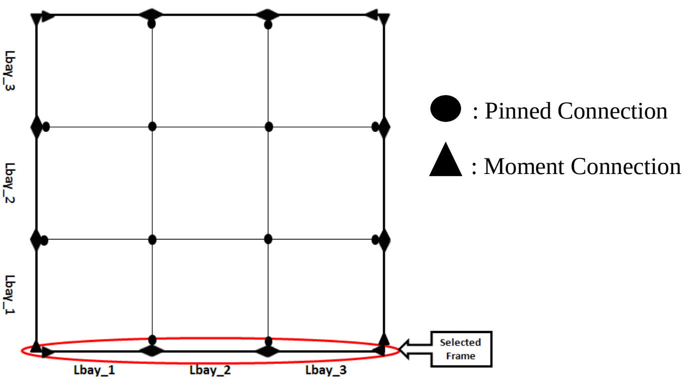

Figure 1. 3, 9 and 15-story 3-D frame top view.

The design of these above cited frames was carried out by utilizing ANSI/AISC 360-10 Specification for Structural Steel Buildings, ANSI/AISC 341-10 Seismic Provisions for Structural Steel Buildings and Specification for Structures to be Built in Disaster Areas (Turkish Earthquake Code 2007, i.e. TEC), TS498 Design Loads for Buildings and ASCE/SEI 7-05 Minimum Design Loads for Buildings and Other Structures. Also, detailing of the steel members was done by using Load and Resistance Factor Design (LRFD) method and since current steel detailing code of Turkey is not suitable for the application LRFD, AISC 360-10 was selected for detailing of the members according to LRFD. In detailing for seismic effects, AISC 341-10 was preferred. For detailing of steel frame members, AISC 360-10 and AISC 341-10 were utilized. In the determination of the earthquake loads and other loads (dead, live and wind), TEC and TS498 were used respectively with the proposed load combinations of ASCE/SEI 7-05. As for the serviceability control of the design, limitations of ASCE/SEI 7-05 were considered once again.

In the structural analysis stage, SAP2000 was utilized. To include the second-order effects, P-Δ option of SAP 2000 was considered. All the load effects from SAP 2000 were considered in the design calculations directly as including the P-Δ effect for all the members. In the design stage, for each three main group of frames (3, 9 and 15-story frames), two different design alternatives were considered as L_{bay}=6m and L_{bay}=10m with L_{col}=3.6 m. Other than story numbers, bay widths and story heights, all other parameters (e.g. response modification factor, earthquake zone, soil class, etc.) were kept constant for all the frames. Also, steel grade used for all frames was selected as S355 with F_y=355 MPa and F_u=510 MPa. All the frames were assumed to be located in seismic zone 1 of Turkey and building heights for 3, 9 and 15-story buildings are 10.8 m, 32.4 m and 54 m respectively. Then, it was concluded Equivalent Lateral Load Method can be applied to 3-story and 9-story frames since their heights are smaller than 40 m, which is the limit of TEC. As for 15-story building, Mode Superposition Method was utilized to determine the earthquake forces. All the loads other than earthquake loads were calculated by referencing TS 498. Wind loads are used as the following by referencing TS 498. The load information is listed as the following:

Dead Loads	: 4.00 kN / m^2 (of floor area)	Live Loads	: 2.00 kN / m^2 (of floor area)
HVAC + Ceiling	: 0.50 kN / m^2 (of floor area)	Exterior Walls	: 5.04 kN / m (of beam area)
Floor Cover	: 0.50 kN / m^2 (of floor area)	Interior Walls	: 3.24 kN / m (of beam area)

Load combinations advised by ASCE SEI/7-05 for LRFD were used in the design. Basic combinations of ASCE SEI/7-05 are the following:

1. 1.4 (D+F)
2. 1.2 (D+F+T)+1.6 (L+H)+0.5 (L_r or S or R)
3. 1.2 D+1.6 (L_r or S or R)+(L or 0.8W)
4. 1.2 D+1.6 W+L+0.5 (L_r or S or R)
5. 1.2 D+1.0 E+L+0.2 S
6. 0.9 D+1.6 W+1.6 H
7. 0.9D+1.0 E+1.6 H

3 Pushover Analyses of Rigid and Semi-Rigid Steel Special Moment Frames

In the modeling of yielding characteristics of column and beam members, default hinge elements of the SAP2000 were utilized. SAP2000 offers also user-defined hinge property definition. In the scope of the study, all the columns and beams' yielding behavior was modeled via default hinges of SAP2000. The default hinge property of SAP2000 for steel beams uses definitions and limits, which are presented in Table 5-6 of FEMA356. Link Element modeling in SAP2000 was used to model semi-rigid connections. Link Elements were defined as different elements like beams and columns. For all the frame cases being analyzed, Link Element length was selected as 15 cm for both beam lengths (L_{bay}=6m and L_{bay}=10 m). As for the loading of the Link Elements, the same vertical loads used in the design of the beams were applied to Link Elements.

A wide range of semi-rigid connections was attempted to be included in the analysis in order to determine the envelope of total effects of all possible semi-rigid connections on the designed frames. For this purpose, four different semi-rigid connection types were defined with changing parameters based on properties of the beam, which the connection is linked, as 'flexible-weak' (FW), 'flexible-strong' (FS), 'stiff-weak' (SW) and 'stiff-strong' (SS) connection. Although these connections have different properties, they have the same generic connection model as shown in Figure 2. On the graph, M_{yc} is the yield moment capacity of the connection, M_{pc} is plastic moment capacity of the connection and Mu is the ultimate moment capacity of the connection. For all these four connection types, M_{yc} was assumed to be equal to 67% of M_{pc} and M_u was assumed to be 80% of M_{pc}.

Their corresponding rotation values were determined based on the definition for SMFs in AISC Seismic Provisions for Structural Steel Buildings that the connection shall be capable of accommodating a story drift angle of at least 0.04 rad. Also, it is advised by Chen (2000) that a connection in SMF is ductile if $\theta_u \geq 0.04$ rad. Thus, for all types of the connections θ_u was assumed to be 0.04 rad. θ_{pc} was calculated by taking reference from θ_u (0.04 rad). θ_{pc} was taken as 80% of θ_u for all the connection models. Other parameters change according to the connection model.

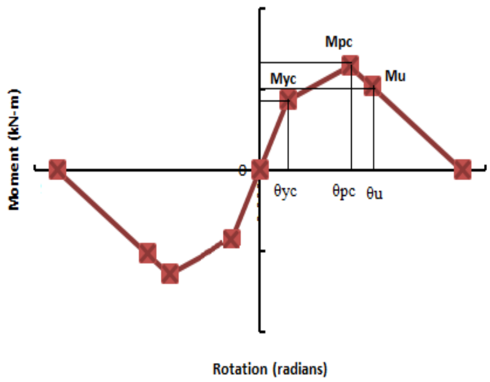

Figure 2. SR connection generic M-θ model

In addition to the rigid connection that has infinite rotational stiffness and infinite strength, there are four connection categories that constitute semi-rigid behavior, which are flexible-weak, flexible-strong, stiff-weak, stiff-strong. These are specified depending on the initial stiffness ($K_{initial}$) of the connection with respect to the beam flexural rigidity (EI_{beam}/L_{beam}) with α factor, i.e. $K_{initial} = \alpha \times EI_{beam}/L_{beam}$, and the plastic moment capacity of the connection M_{pc} with respect to the plastic moment capacity of the beam M_{pb} with a factor β, i.e. $M_{pc} = \beta \times M_{pb}$. Flexible and stiff terms are associated with $\alpha = 7$ and $\alpha = 15$, respectively. Weak and strong properties of the connections are associated with $\beta=0.75$ and $\beta=1.50$, respectively. In each case, there is only one parameter left to be determined, that is θ_{yc}, and this can be calculated by dividing M_{yc} to $K_{initial}$.

Figure 3 shows the Pushover Curves of rigid, flexible-weak connection and flexible-strong connection cases for 3 story frames with L_{bay}=6m in comparison with the rigid case. For all the groups of frames, same pushover curves were obtained in a similar manner. Detailed presentation of all analyses results are available in the thesis of Metin (2013).

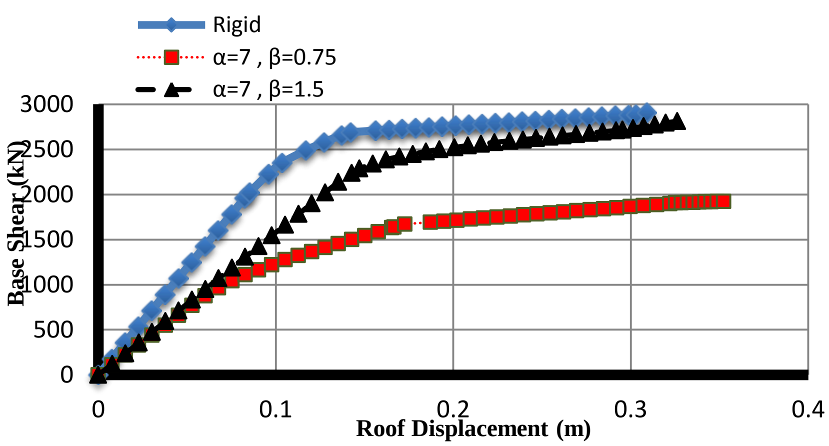

Figure 3. Pushover Curves of 3-Story Frame under Code Lateral Load Pattern (L_{bay}=6m)

4 Evaluations of Overstrength and Ductility Reduction Factors

In the scope of the study, 6 main groups of frames were designed with rigid connections as outlined above. Also, all relevant nonlinear modeling assumptions of these frames were also presented. Considering the fact that rigid connection and four different connection categories are taken into account, and furthermore, three different lateral load patterns are used, i.e. code pattern, uniform pattern and elastic first mode pattern, all cases sum up to 90 pushover analyses. The resulting pushover curves were tried to be idealized by the method advised in FEMA356. As seen from Figure 4, FEMA356 offers a bilinearization method for Force-Displacement Curves based on post-yield slope.

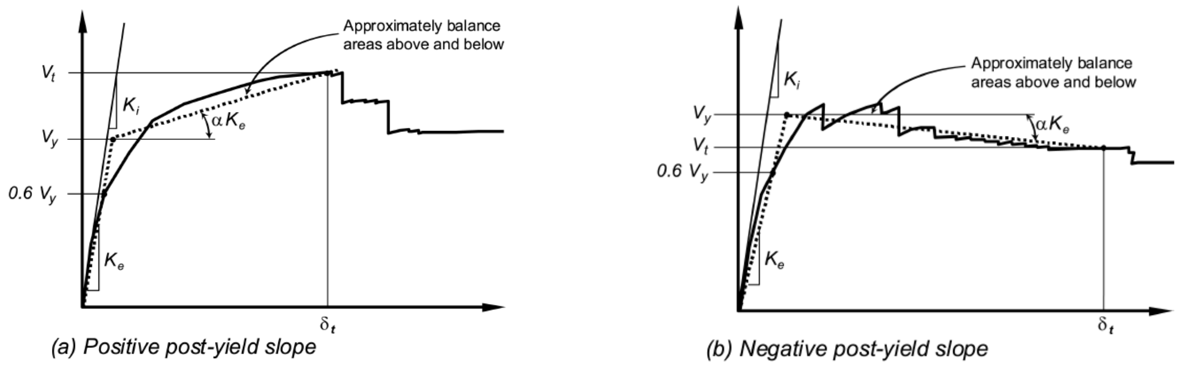

Figure 4. Idealized force-displacement curves.

In this bilinearization method, initial slope of the curve is named as K_e and post-yield slope is characterized by factor α. In this method, line segments are used to idealize the curve approximately balancing the areas above and below the real pushover curve. The effective lateral stiffness is taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength (V_y). Post-yield slope (α) is determined by the line segment which passes through actual curve at the calculated target displacement. Target displacement was determined in the pushover curves based on the advice of FEMA356 for Collapse Prevention (CP) and Life Safety (LS) Structural Performance Levels for steel moment frames. FEMA356 defines 2.5% inter story drift ratio (IDR) for LS level and 5% inter story drift ratio (IDR) for CP Level. The following scale was used for CP level determination:

- IDR@CP Level was assumed as 3% for low-rise frames
- IDR@CP Level was assumed as 5% for mid-rise and high-rise frames

Based on the above explanations, R_o and R_μ were calculated by using the following equations:

$$R_o = \frac{V_y}{V_d} \quad (1)$$

where V_y: Effective yield strength and V_d: Design base shear

Based on the design natural periods of structures and considering semi-rigid connections decrease the rigidity of the system, all the frame cases were assumed to have enough flexibility to be able to assume the following equality based on the studies of Miranda (1993) and Miranda& Bertero (1994) and Newmark & Hall. (1982)

$$R_\mu = \mu \quad (2)$$

where μ: Ductility ratio

Figure 5 shows R_o and R_μ values for the low-rise frames 6 m bay length under the Code Lateral Loading Pattern, which were calculated from the corresponding pushover curves. For all other groups of the frames, same methodology was used to calculate R_o and R_μ values, and results are available in Metin (2013).

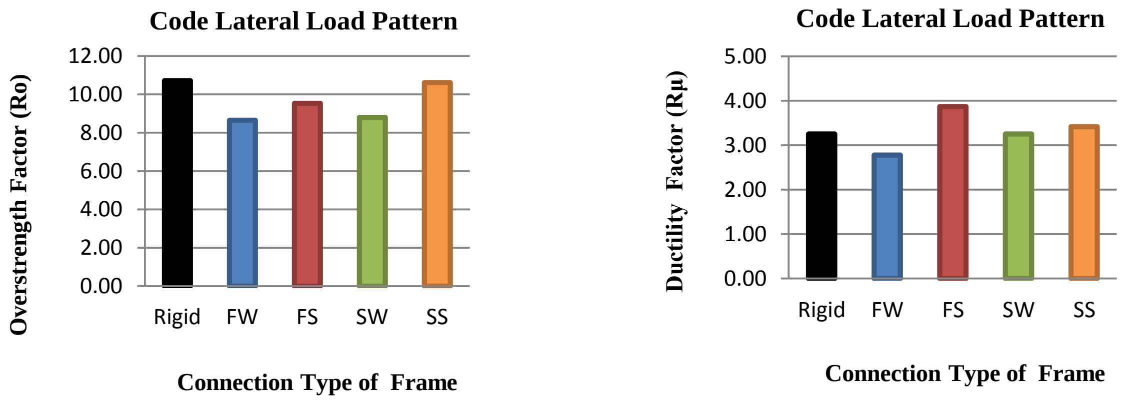

Figure 5. R_o and R_μ values for 3-Story Frame under Code Lateral Load Pattern (L_{bay}=6m)

5 Conclusion

The results of the study demonstrate that semi-rigid connection modeling has a significant effect on the steel special moment frames. This emphasizes that to obtain accurate results and more reliable response of the steel special moment frames, semi-rigid connections should be treated with great care. Also, semi-rigid connections could eliminate overdesign problem in case of designing steel special moment frames as evaluated based on rigid frames' having higher R_o values than semi-rigid frames in all the analysis cases.

Although semi-rigid connections contribute to the steel special moment frames' ductility, results showed that ductility of the rigid frames were higher in all the cases. This indicates an important conclusion that to increase semi-rigid frames' ductility, plastic hinges in the semi-rigid connections should not be formed earlier than plastic hinges in beams. Thus, timing of the plasticity formation in semi-rigid connections in comparison with plasticity formation in beams would have a major impact on the steel special moment frames' ductility response.

The results of pushover loadings and resultant behavior parameters (R_o and R_μ) indicate the behavior in some cases can be sensitive to the lateral loading type. Selection of a proper lateral loading type has a significant effect on the response evaluation of steel special moment frames.

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