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Pushover analysis of gabled frames with semi-rigid connections

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Abstract. The nonlinear static analysis of structure, which is under the effect of lateral loads and provides the capacity curve of the structure, is defined as a push-over analysis. Ordinarily, by using base shear and the lateral displacement of target point, the capacity curve is obtained. The speed and ease of results interpretation in this method is more than that of the NRHA responses. In this study, the nonlinear static analysis is applied on the semi-rigid steel gabled frames. It should be noted that the members of this structure are analyzed as a prismatic beam-column element in two states of semi-rigid connections and supports. The gabled frame is modeled in the OpenSees software and analyzed based on the displacement control at the target point. The lateral displacement results, calculated in the top level of columns, are reported. Furthermore, responses of the structure are obtained for various support conditions and the rigidity of nodal connections. Ultimately, the effect of semi-rigid connections and supports on the capacity and the performance point of the structure are presented in separated graphs.

Keywords: nonlinear static analysis; gabled steel frame; beam-column element; semi-rigid connection; OpenSees software

1. Introduction

In recent years, the construction of steel gabled frame takes place constantly as industry factories. The heavy machines such as crane and some other encasements, which are supported by the main frame, cause the increasing of the effective mass of structure. Therefore, the dynamic analysis of gabled frame is really necessary. It should be added that as the height and the span of gabled frame structures increase, the stiffness becomes smaller and the basic natural period generally reaches to 0.5 second, sometimes even up to 1 second.

The nonlinear static analysis is a simple way for estimating resisting capacity in plastic limit. This technique can also be utilized to determine weak regions of the structure. A pattern of the lateral load distributed in structure height is defined in this method. The lateral load increases uniformly based on displacement control in the maximum level of the structure. Afterwards, the capacity curve of the structure is established according to reports of the maximum level displacement and base shear in consecutive stages. With interference of the capacity curve and

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seismic response spectrum, the performance point of the structure can be calculated.

Several researches have been carried out in the field of nonlinear static analysis of steel frames. The development and implementation of this method date back to recent decades. Among them, references (Saiidi and Sozen 1981, Fajfar and Gaspersic 1996, Bracci *et al*. 1997) can be mentioned. It was concluded that the nonlinear static analysis of a concrete frame would lead to conservative responses in comparison with nonlinear dynamic analysis by (Pachenari *et al*. 2013). Tsai and Lin examined the earthquake-resistant RC buildings and understood that the capacity of the structure showed to be capable of predicting the progressive collapse (Tsai and Lin 2008). In 2001, Mwafy and Elnashai employed the nonlinear static analysis against the incremental dynamic collapsed analysis for RC structures. Also, the actual and artificial seismic spectra for a twelve floors structure were utilized in their study (Mwafy and Elnashai 2001).

One of assumptions in dynamic analysis of frames is the consideration of rigid and hinge connections. These connections include support and nodal connections. Although, the application of this method simplifies the analysis, it doesn't reveal the true behavior of the structure. In the past, many researches have been done about semi-rigid frame analysis. A numerical solution was presented for linear and non-linear free vibration analysis of frames with semi-rigid connections by Chan and Ho. They modeled connections with rotational springs and introduced stiffness matrix (Chan and Ho 1994). In 1989, the influence of semi-rigid connections with non-linear behavior on static and dynamic responses of spatial frames is examined by Shi and Atluri. They analyzed semi-rigid frames by using geometric stiffness matrix of the semi-rigid member (Shi and Atluri 1989). In 1998, Rodrigues *et al*. investigated the behavior of steel frames, having nodal semi-rigid connections, with and without lateral brace. They employed FEM in the analysis frame (Rodrigues *et al*. 1998). Also, the seismic analysis of the braced steel frame with semi-rigid connections was investigated by (Fu *et al*. 1998).

In recent decade, seismic analysis researches have been performed about semi-rigid steel frames. Nguyen and Kim based on beam-column theory, introduced an effective solution for nonlinear elastic analysis of three dimensional steel structures with semi-rigid connections. The effect of nonlinear geometric behavior was highlighted with stability function and geometric stiffness matrix. They utilized Newton-Raphson method and Newmark integration for solving nonlinear dynamic equation (Nguyen and Kim 2013). In 2002, the influence of the flexibility and damping of connections on steel frame dynamic responses was examined by Sekulovic (Sekulovic *et al*. 2002) They worked on nonlinear impact of nodal connection behavior of frame members. Finally, they found out a numerical pattern considering second-order nonlinear effects and the behavior of connections (Sekulovic *et al*. 2002). The numerical modeling and experimental study were performed to investigate the semi-rigid connection of corner joints in gabled frames with tapered members by (Wang *et al*. 2011). Another research was done on the design of a light-weight steel structure with three span gabled frame by (Du *et al*. 2012).

In this research, nonlinear pushover analysis is used for determining the capacity curve of the one-bay gabled steel frame. Members of this structure are modeled in a prismatic shape. Moreover, the effect of various support conditions is assessed. For investigating the flexibility of nodal and support connections effect, the rotational springs are used. For analysis of this frame, OpenSees software (version 2.4) is employed. The conclusions are separately reported for various states of support and semi-rigid connections. It should be noted that the performance point of the structure is obtained via the graphical method of ATC-40 and with stages of trial and effort for the maximum level of frame columns. The inherent damping of the structure is estimated 5 percent. Ultimately, the displacement of the target point and the base shear of the structure are published for the performance point of the structure.

2. Modeling of gabled frame

In recent years, it is more common to use gabled steel frame rather than truss in large bays. The current study investigates the behavior of one kind of these symmetrical frames. For a detailed examination of seismic behavior of this frame, its supports and connections are modeled in the state of semi-rigid. Members of this frame are assumed to be prismatic with equal moment of inertia. The general model of the frame is illustrated in Fig. 1. The span of the frame in all cases is supposed to be 12.0 m. The rigidity of connections in the frame is shown by symbol *R*. The frame columns height is equal to 5.0 m. Also, the height coefficient of the slope roof, which is defined by letter *n*, is assumed to be 0.4. Furthermore, Table 1 reports properties of geometric cross-sections of structure members.

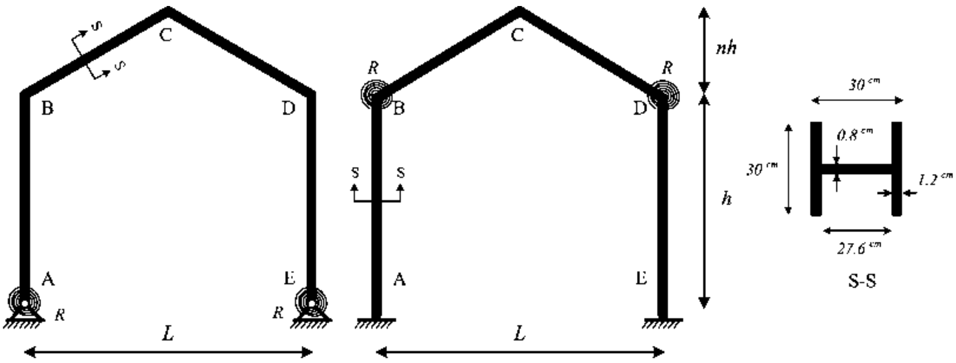

Fig. 1 The model of steel gabled frame with uniform I-section

Table 1 The cross-section properties of gabled frame members

Cross-section properties	h (cm)	n	d (cm)	h_w (cm)	t_f (cm)	t_w (cm)	b (cm)	A (cm^2)	I (cm^4)
	500	0.4	30	27.6	1.2	0.8	30	94.08	16340.19

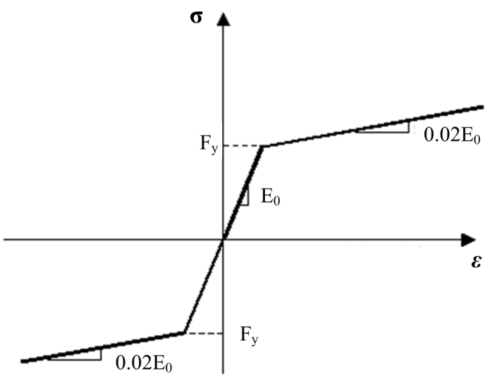

Fig. 2 The stress-strain curve of material Steel01

The analysis of gabled frame is performed by using nonlinear static analysis in OpenSees software and seismic nonlinear spectra in SeismoSpect software, which is used to achieve displacement-acceleration inelastic response spectrum (demand spectrum). Making model in the software is performed in two-dimensional case and each node has three degree of freedom. Effects of *P*-Δ are ignored. Materials, which are used in structure members, are supposed to be bilinear axial steel. It should be added, the yield stress and the module of elasticity are equal to 2,400 Kgf/cm^2 and 2.05×10^6 Kgf/cm^2, respectively. The stress-strain behavior of materials is shown in Fig. 2. Members of this frame are modeled as a nonlinear beam-column element. The modified Newton-Raphson algorithm , which is used for solving nonlinear equation, is employed in this analysis (Foundation ©2006).

3. Ground motions selection

The appropriate selection of information from earthquake records impacts on the frames dynamic analysis. In order to achieve demand spectrum, the specifications of five suitable earthquakes are considered. The record of these earthquakes is shown in Table 2. Besides, by utilizing SeismoSpect software, the mean spectrum can be calculated (LTD 2002). The demand

Table 2 Characteristics of ground motions (Regents of the University California 2000)

Earthquake	Year	Magnitude (*M*)	Station	CD* (km)	PGV (cm/s)	PGA (g)
Kobe	1995	6.9	0 KJMA	0.6	38.3	0.821
Imperial_Valley	1979	6.5	5165 El Centro	5.3	20.7	0.707
Northridge	1994	6.7	90055 Simi Valley	14.6	40.9	0.877
Loma Prieta	1989	6.9	14 WAHO	16.9	38.0	0.638
Landers	1992	7.3	24 Lucerne	1.1	31.9	0.785

*CD: Closest distance to fault rupture

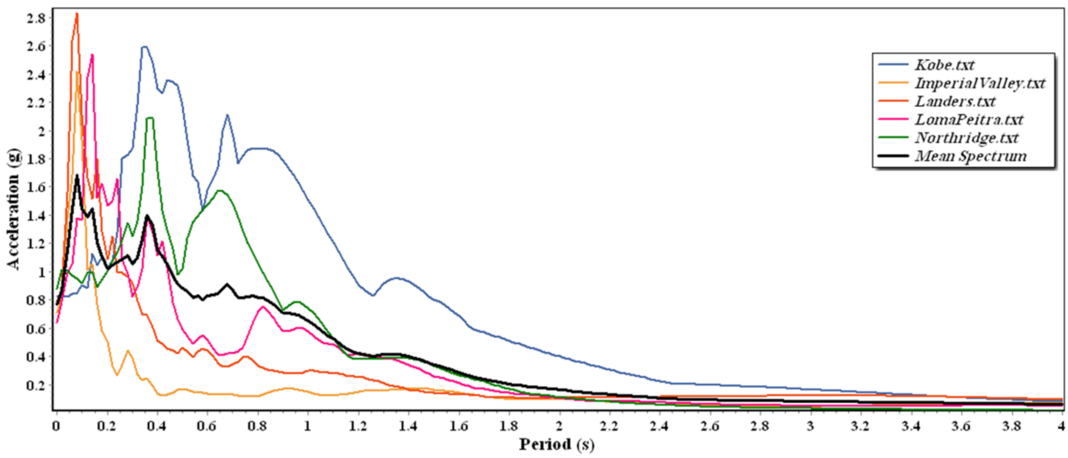

Fig. 3 Ground motion spectra and mean spectrum

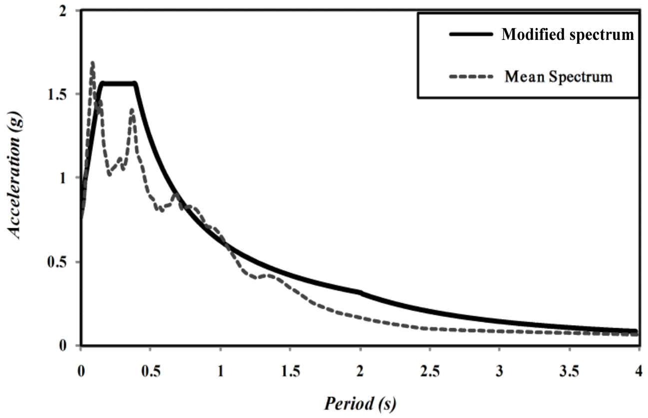

Fig. 4 Comparing the mean spectrum with modified spectrum

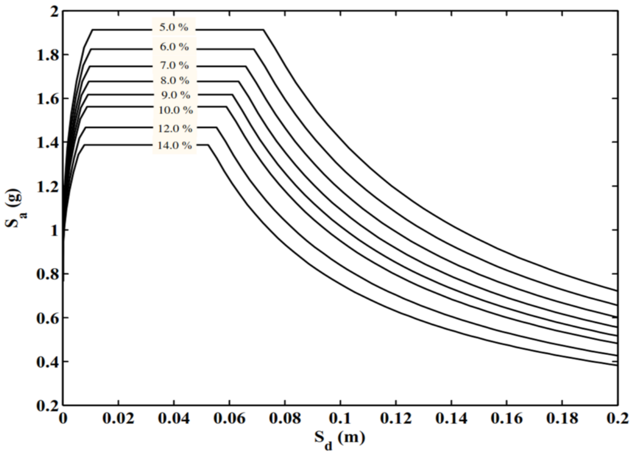

Fig. 5 The modified spectrum for various damping percentages

spectra of earthquake records in Table 2 and the mean spectrum are shown in Fig. 3.

The vertical and horizontal axes of the diagram show the spectral acceleration and period, respectively. Fig. 4 shows the modified mean spectrum.

By using Eq. (1), the standard design spectrum changes from spectral acceleration-period to spectral acceleration- spectral displacement. Fig. 5 illustrates modified spectrum for with various damping percentages

$$S_d = S_a \frac{T^2}{4\pi^2} \tag{1}$$

4. The nonlinear static analysis method

In this paper, in order to determine the performance point of the structure, the demand spectrum technique of ACT-40 standard is used. In this technique, the capacity curve of structure is compared with demand spectrum. In the method of the capacity spectrum, the maximum displacement response of a SDOF nonlinear structure is supposed to be equal to that of SDOF elastic structure with an effective period. In this research, the gabled frame structure is modeled with an equivalent mass. Moreover, the values of the modal participation factor and the first mode shape are assumed to be one because the total mass of frame is considered at the top level of columns. In this technique, the performance point is calculated according to Fig. 6. It is necessary to depict the demand and capacity curve in spectral acceleration contrasted with spectral displacement, called ADRS. In order to achieve it, some calculations are noted in the reference (ATC-40 1996). These calculations are summarized in Eq. (2).

$$S_a = \frac{\frac{V}{W}}{\alpha_1}, \qquad S_d = \frac{\Delta_{roof}}{\Gamma_1 \phi_{roof}} \tag{2}$$

Where α_1 is the modal mass factor for the first mode and Γ_1 is the modal participation factor for the first mode. The value of them is equal to one. The mode shape is also introduced by φ_{roof}. This factor is also considered one. The structure base shear and the sum of live and dead loads are shown by parameters of *V* and *W*, respectively.

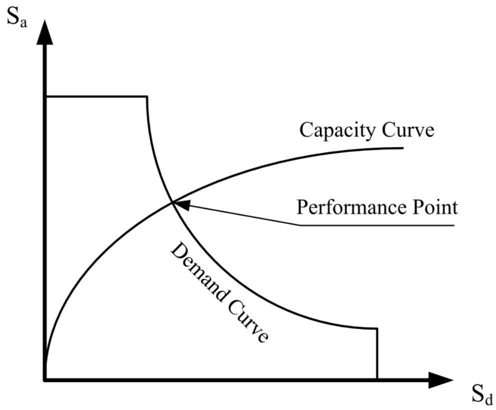

Fig. 6 The performance point in ATC-40 capacity spectrum method

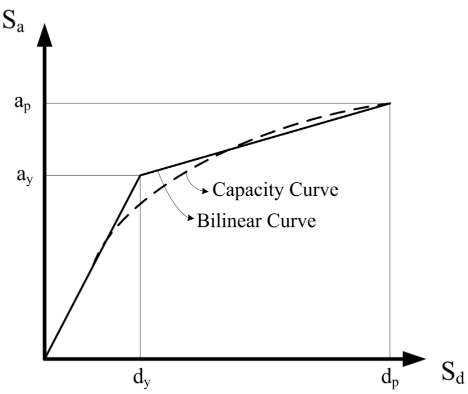

Fig. 7 The capacity and equivalent bilinear curves according to ATC-40 rule (ATC-40 1996)

One of important subjects in capacity spectrum technique is estimating effective damping of structure in the nonlinear limit and preparing reduced demand spectrum. Each point on capacity curve has equivalent damping and period. To determine all these specifications, the capacity curve is approximated by a bilinear curve. To access bilinear curve according to Fig. 7, the following rules are to be considered:

(1) The bottom area of the actual capacity curve should be equal with that of the bilinear curve.
(2) The crossing of tangent line in elastic limit with initial slope, defining as the effective stiffness, and the capacity curve of the structure is a point which has the value of $0.6Vy$.

To decrease demand spectrum, the nonlinear damping of structure should be estimated. This damping includes visco-elastic damping and hysteresis damping. Visco-elastic damping is an inherent damping of the structure which supposed to be 5.0 percent. Hysteresis damping is related to hysteresis internal area which is calculated by depicting base shear versus structure displacement. Accordingly, ATC-40 rule suggested the effective damping calculation as follow equation, by the use of bilinear capacity curve, for determining reduced demand spectrum.

$$\xi_{eff} = \frac{63.7(a_y d_p - a_p d_y)}{a_p d_p}\kappa + 5 \tag{3}$$

5. Connections behavior pattern

It is common that the behavior of a connection is defined by its movement-curvature. In this study, in order to model of semi-rigid connections, a linear pattern is used. The simplest connection pattern is the linear pattern which is shown in Eq. (4).

$$M = R.\theta \tag{4}$$

The connection stiffness (R) could be introduced by the stiffness of the initial connection or tendinous stiffness. In this research, the dimensionless coefficient r is defined as flexibility ratio of connection and support. The range of r is from zero to one. The relation between flexibility factor and connection stiffness is shown in Eq. (5) (Monforton and Wu 1963).

$$r = \frac{1}{1 + \frac{3EI}{RL}} \qquad R = \frac{3EI}{L}\left(\frac{r}{1-r}\right) \tag{5}$$

Where the connection stiffness is R and the flexibility factor is r. In this research, only the initial stiffness of connections is considered. It should be noted, if r tends to zero, the aforementioned connection is hinge and if r tends to one, the connection is fixed. In this study, the elastic module (E) is equal to 205 GPa. The height of columns is considered 5.0 m. Specifications of the column cross-section are discussed in Table 1. The connection stiffness versus their flexibility factor (r) is reported in Table 3.

Table 3 The connections stiffness versus flexibility factor (r)

r	*R* (N.m)
0	0
0.2	5024611
0.4	13398963
0.6	30147666
0.8	80393776
1.0	∞

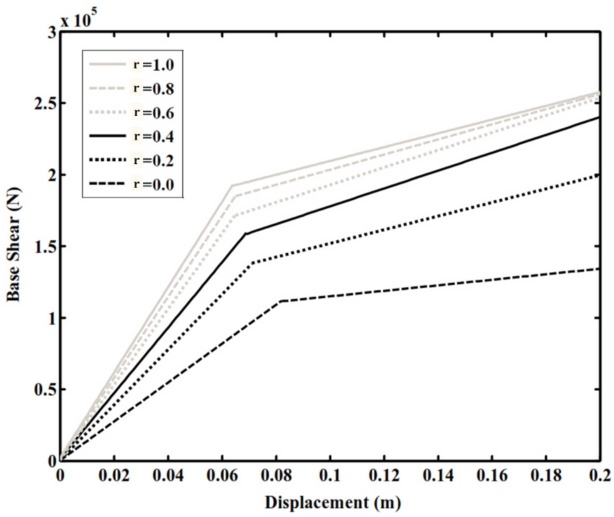

Fig. 8 Equivalent bilinear capacity curves of gabled frame with various connections flexibility factor

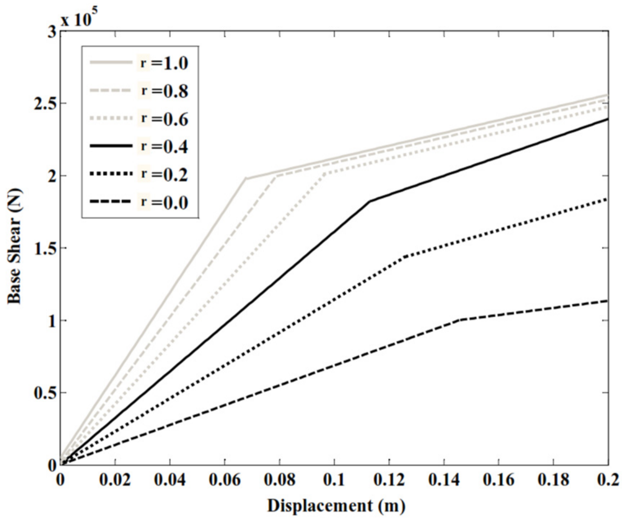

Fig. 9 Equivalent bilinear capacity curves of gabled frame with various supports flexibility factor

6. Parametric study

By utilizing outputs of OpenSees software, the real capacity curve of gabled frame structure is obtained. Moreover, with the application of the bilinear solution, the equivalent bilinear capacity curve is depicted. The horizontal axis of diagrams expresses the displacement of the top level of columns and vertical axis reports the total base shear of frame. The bilinear capacity curves for various factors of the connections and support flexibility are shown in Figs. 8-9.

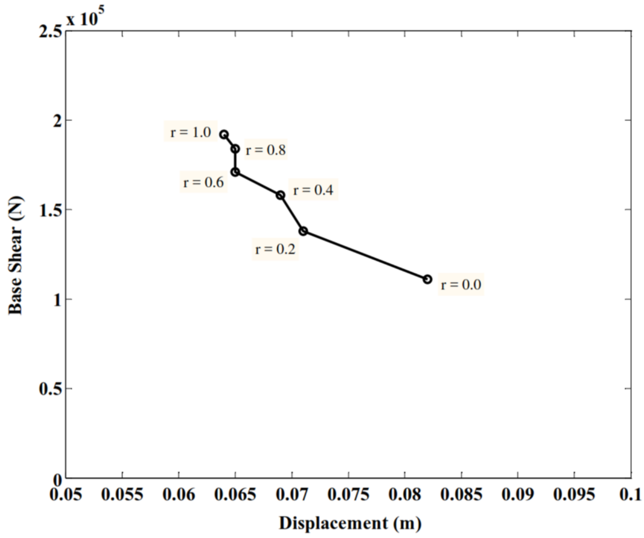

Fig. 10 Elastic performance curves of gabled frame with various connections flexibility factor

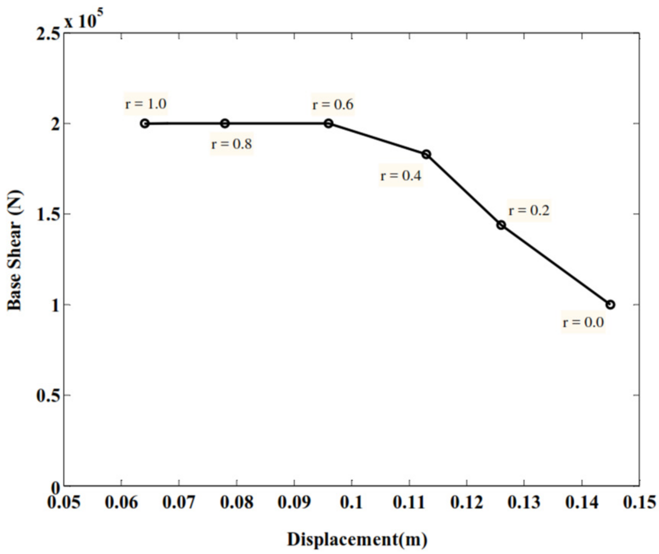

Fig. 11 Elastic performance curves of gabled frame with various supports flexibility factor

To determine the initial deformation of structure, the yielding point of the structure capacity curve should be calculated. The elastic behavior of structure describes a point which transforms linear limit to nonlinear limit, and is illustrated in Figs. 10-11. Noted, the flexibility factor of connections and supports are shown by index *r*, on diagram. The structure yielding point with semi-rigid connections and fixed supports is reported in Fig. 10. In addition, the structure yielding point with rigid connections and semi-rigid supports is shown in Fig. 11. With the application of these points and structure performance points, it is possible to estimate the structure ductility factor in different states of connection and support flexibility.

By applying the ATC-40 rule and trial and effort method, performance points of gabled frame is calculated and reported in Table 4. The final step of trial and effort method is presented in this table. The effective damping, spectral acceleration, target point displacement and the total base shear are reported. In this case, the supports are considered to be fixed.

Similarly, performance points of structure with the effect of semi-rigid support are shown in Table 5. In this case, the nodal connections are assumed to be rigid.

In order to compare the capacity curve of gabled frame in two cases of semi-rigid connections and supports, the diagram of total base shear versus displacement for top level of columns is illustrated in Fig. 12. The horizontal axis of this graph is displacement in the unit of meter and the vertical axis is the total base shear in the unit of kN. It is worth to mention that in Fig. 12, the flexibility factor of the support is shown by r_s and the flexibility factor of the connection is defined by r_c. Based on Fig. 12, the base shear and displacement values of points, which have been

Table 4 Performance points of gabled frame with semi-rigid connections

Capacity curve properties						Performance point			
r	d_y (m)	d_p (m)	a_y (g)	a_p (g)	m (kg)	S_a (g)	S_d (m)	V (kN)	ξ (%)
0.0	0.082	0.125	0.754	0.822	15000	0.781	0.114	115	11
0.2	0.071	0.105	0.937	1.046	15000	1.026	0.099	151	9
0.4	0.069	0.093	1.073	1.175	15000	1.175	0.093	173	8
0.6	0.065	0.087	1.162	1.257	15000	1.257	0.087	185	8
0.8	0.065	0.085	1.250	1.325	15000	1.318	0.082	194	8
1.0	0.064	0.080	1.325	1.372	15000	1.372	0.08	203	8

Table 5 Performance points of gabled frame with semi-rigid supports

Capacity curve properties						Performance point			
r	d_y (m)	d_p (m)	a_y (g)	a_p (g)	m (kg)	S_a (g)	S_d (m)	V (kN)	ξ (%)
0.0	0.146	0.168	0.679	0.720	15000	0.720	0.168	106	7
0.2	0.126	0.139	0.992	1.52	15000	0.998	0.131	147	6
0.4	0.113	0.115	1.236	1.25	15000	1.25	0.115	184	5
0.6	0.097	0.102	1.325	1.379	15000	1.325	0.097	195	6
0.8	0.079	0.086	1.325	1.379	15000	1.379	0.086	203	7
1.0	0.064	0.080	1.325	1.379	15000	1.379	0.080	203	8

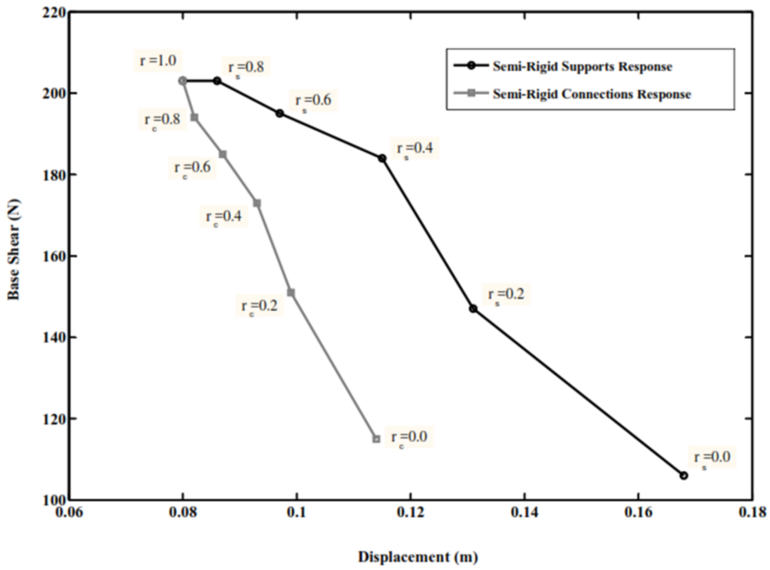

Fig. 12 Performance points of gabled frame with semi-rigid supports and connections

illustrated with r = 1.0, are the same. Conclusively, the accuracy and correctness of gabled frame modeling with semi-rigid connections is approved.

6. Conclusions

The development of using gabled frame in large span and the existence of the lumped mass cause the seismic analysis of this frame controls safety and optimum designs. In this study, nonlinear static analysis (Pushover) is applied to gabled steel frame considering effects of semi-rigid connections and supports. By utilizing the outputs of OpenSees software and analytical responses, the following results can be obtained:

- Based on the capacity curve, using fixed supports instead of simple supports leads to decrease the displacement of top level of column and increase the total base shear. Although, using fixed support in gabled frame increases the total base shear and reduces the ductility of the structure.
- According to the elastic performance curves, by growing the nodal flexibility factor of the structure with rigid supports, the elastic displacement in target point decreases and the corresponding base shear increases. This process for the case of rigid connections with semi-rigid supports is true, except that the effect of semi-rigid supports on the elastic displacement of the target point is more than that of semi-rigid connections.
- In the case of semi-rigid supports, in the range of fixed coefficient (r), which is more than 0.6, it is observed that by decreasing elastic displacement of the target point, great change in the total base shear does not occur. Whereas in the structures with rigid supports and semi-rigid connections in the range of the flexibility factor, which is more than 0.6, it is observed that by increasing of the total base shear, the elastic displacement of the target

point does not change.

- Based on the performance point in two different states of semi-rigid connections and supports, it can be concluded that the support flexibility coefficient has more impact on the final displacement of target point than the connection flexibility coefficient. Although, the range of the total base shear is the same for both states.

References

ATC-40 (1996), Seismic Evaluation and Retrofit of Concrete Buildings; Applied Technology Council, Redwood City, CA, USA.

Bracci, J.M., Kunnath, S.K. and Reinhorn, A.M. (1997), “Seismic Performance and Retrofit Evaluation of Reinforced Concrete Structures”, *J. Struct. Eng. ASCE*, **123**(1), 3-10.

California, Regents of the University (2000), http://peer.berkeley.edu/smcat

Chan, S.L. and Ho, G.W.M. (1994), “Nonlinear Vibration Analysis of Steel Frames with Semirigid Connections”, *J. Struct. Eng.*, **120**(4), 1075-1087.

Du, W., Sun, Z., Zhang, H. and Yu, F. (2012), “Design and Analysis of a Light-Weight Steel Gabled Frames Structure”, *Appl. Mech. Mater.*, **238**(1), 572-575.

Fajfar, P. and Gaspersic, P. (1996), “The N2 Method for the Seismic Damage Analysis of RC Buildings”, *Earthq. Eng. Struct. Dyn.*, **25**(1), 31-46.

Foundation, N.S. (©2006), opensees-support @ berkeley.edu.

Fu, Z., Ohi, K., Takanashi, K. and Lin, X. (1998), “Seismic behavior of steel frames with semi-rigid connections and braces”, *J. Constr. Steel Res.*, **46**(1-3), 440-441.

LTD, SeismoSoft (2002), SeismoSpect, Pavia, Italy.

Monforton, G.R. and Wu, T.S. (1963), "Matrix Analysis of Semi-Rigid Connected Steel Frames ", *J. Struct. Div.* (*ASCE*), **89**(6), 13-42.

Mwafy, A.M. and Elnashai, A.S. (2001), “Static pushover versus dynamic collapse analysis of RC buildings”, *J. Eng. Struct.*, **23**(5), 407-424.

Nguyen, P.C. and Kim, S.E. (2013), “Nonlinear elastic dynamic analysis of space steel frames with semi-rigid connections”, *J. Constr. Steel Res.*, **84**(1), 72-81.

Pachenari, A., Keramati, A. and Pachenari, Z. (2013), “Investigation of progressive collapse in intermediate RC frame structures”, *Struct. Des. Tall Special Build.*, **22**(2), 116-125.

Rodrigues, F.C., Saldanha, A.C. and Pfei, M.S. (1998), “Non-linear analysis of steel plane frames with semirigid connections”, *J. Constr. Steel Res.*, **46**(1-3), 94-97.

Saiidi, M. and Sozen, M.A. (1981), “Simple nonlinear seismic analysis of R/C structures”, *J. Struct. Div. ASCE*, **107**(5), 937-951.

Sekulovic, M., Salatic, R. and Nefovska, M. (2002), “Dynamic analysis of steel frames with flexible connections”, *J. Comput. Struct.*, **80**(11), 935-955.

Shi, G. and Atluri, S.N. (1989), “Static and dynamic analysis of space frames with nonlinear flexible connections”, *Int. J. Numer. Methods Eng.*, **28**(11), 2635-2650.

Tsai, M. and Lin, B. (2008), “Investigation of progressive collapse resistance and inelastic response for an earthquake-resistant RC building subjected to column failure”, *J. Eng. Struct.*, **30**(12), 3619-3628.

Wang, Y., Liu, Y.J. and Xu, Y.F. (2011), “Stiffness analysis on semi-rigid joints in gabled frames”, *Adv. Mater. Res.*, **243-249**(1), 120-123.

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