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# Pushover analysis of prefabricated structures with various partially fixity rates

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**Abstract.** Prefabricated structures are constructed by bolted connections of separated members. The design and analysis of these structures are generally performed by defining fully hinges for the connection of separated members at the joint of junction. In practice, these connections are not fully hinged. Therefore, the assumption of semi-rigid connections (restrained or partially fixity) instead of fully hinge connections is a more realistic approach for bolted connections used in the prefabricated elements. The aim of this study is to investigate the effects of semi-rigid connections on seismic performance of prefabricated structures. Nonlinear static analysis (pushover analysis) of a selected RC prefabricated structure is performed with SAP2000 structural analysis program by considering various partially fixity percentages for bolted connections. The target values of roof displacements obtained from the analyses according to ATC-40, FEMA-356, FEMA-440, and TEC-2007 codes are compared each other. The numerical results are given in tables and figures comparatively and discussed. The results show that the effects of semi-rigid connections should be considered in design and analysis of the prefabricated structures.

**Keywords:** prefabricated structures; semi-rigid connections; nonlinear static pushover analysis

## 1. Introduction

Prefabricated structures have been widely used for industrial areas since they are more economical in construction and they provide large areas for production. In recent years, many damages in the prefabricated structures have occurred due to past earthquakes because of their weakness in construction. Especially, significant economic losses occurred after 1989 Loma Prieta and 1994 Northridge earthquakes in United States of America; 1995 Kobe earthquake in Japan; 1992 Erzincan, 1999 Marmara and Duzce, 2003 Bingol and 2011 Van earthquakes in Turkey (Çavdar and Bayraktar 2014).

Three types of structural damages have been frequently observed in the precast buildings: flexural hinges at the base of columns, roof girders rotated off their supports due to stripping bolts, and pounding of the precast elements at the roof level (Posada and Wood 2002, Arslan *et al*. 2006). For this reason, seismic safety of prefabricated structures becomes a common subject of investigation in worldwide.

Prefabricated structures have neither fully hinged connections nor fully rigid connections. The semi-rigid connections have not been considered in design and analysis of prefabricated structures and truss systems. For this reason, this type of structures was heavily damaged in past earthquakes. The damages reported are due to the weak and insufficient connections (Doğan *et al*. 2010, Nguyen and Kim 2014).

There have been a few procedures for seismic performance evaluation of structures in the literature. The most important ones of these procedures are defined in the following codes: The seismic evaluation and retrofit of concrete buildings (ATC-40 1996), pre-standard and commentary for the seismic rehabilitation of building (FEMA-356 2000), improvement of nonlinear static seismic analysis procedures (FEMA-440 2005) and seismic rehabilitation of existing buildings (ASCE-41 2007). In Turkey, performance based evaluation of existing buildings has been given in Turkish Earthquake Code (TEC-2007 2007). The procedures contain linear and nonlinear elastic methods for assessment of seismic performance of structures. The seismic performance of structures is determined by nonlinear static (pushover analysis) and nonlinear time history analyses. Nonlinear static analysis has been widely used for seismic design and assessment of structures (Tehranizadeh *et al*. 2016, Sarkar *et al*. 2016). The analysis is performed under monotonically increasing lateral equivalent earthquake forces with specified height-wise distribution until a target displacement is reached (Chintanapakdee and Chopra 2003, Goel 2011). Although nonlinear time history analysis is the most reliable among all the analysis methodologies, nonlinear static analysis has been more frequently used in the worldwide due to its simplicity in application compared to the nonlinear time history analysis.

Many studies have been performed about performance evaluation of prefabricated structures (Arslan *et al*. 2006,

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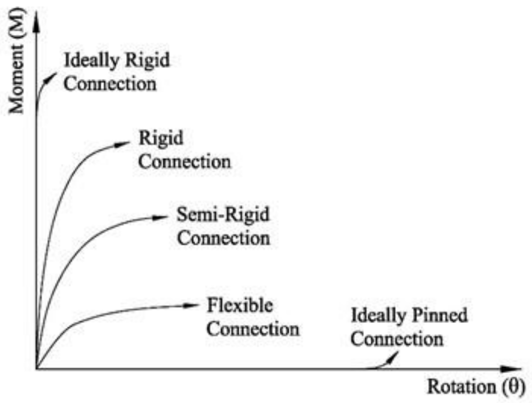



Fig. 1 Connection types for structural elements

Magliulo *et al*. 2008, Popa and Cotofana 2009, Senel and Kayhan 2010, Bozdağ and Düzgün 2010, Seckin *et al*. 2015, Tuan Ngo and Mendis 2016). In these studies, connections of prefabricated structures were considered as fully hinged. However, this assumption does not give realistic results due to bolted connections in the prefabricated structures. It can be seen from the literature review that a few works on performance evaluation of prefabricated structures with semi-rigid connections have been studied (Kartal *et al*. 2010, Deyanova *et al*. 2014). Therefore, the aim of this study is to investigate the effects of semi-rigid connections on seismic performance of prefabricated structures. Nonlinear static analysis of a selected RC prefabricated structure is performed with SAP2000 structural analysis program (SAP2000 2015) by considering various partially fixity percentages for bolted connections. Columns of the prefabricated structure are modeled as a nonlinear frame element. The nonlinear behavior of the columns is taken into account by assuming the plastic hinges at the lower end of columns in which plastic deformations are concentrated. The target displacements and the pushover curves obtained from the pushover analyses according to ATC-40, FEMA-440, FEMA-356, and TEC-2007 codes are compared each other.

## 2. Semi-rigid connections

The connections of structural elements are, in general, assumed to be ideally rigid or ideally pinned (Kartal 2004) as shown in Fig. 1. However, flexibility of the connections may change according to their moment-rotation curves (Fig. 1). The moment-rotation curves are usually derived from experimental data, e.g., various $M$-$\theta$ model types developed by Chen and Lui (1991).

The connection flexibility at the ends of structural elements can be represented by rotational springs. The stiffness terms of the rotational springs can be obtained by using the Young's modulus ($E$), the moment of inertia ($I$), and the length ($L$) of the related beam element. This approach is very effective and useful for introducing the connection flexibility (Kartal *et al*. 2010). The stiffness matrix of the beam element with rotational springs at both ends in the local coordinate system can be written in the following (McGuire *et al*. 1999)

$$[k] = \frac{EI}{L^3}\begin{bmatrix} 12\theta_1 & 6L\theta_2 & -12\theta_1 & 6L\theta_3 \\ 6L\theta_2 & 4L^2\theta_4 & -6L\theta_2 & 2L^2\theta_5 \\ -12\theta_1 & -6L\theta_2 & 12\theta_1 & -6L\theta_3 \\ 6L\theta_3 & 2L^2\theta_5 & -6L\theta_3 & 4L^2\theta_6 \end{bmatrix} \tag{1}$$

The coefficients of $\theta_1$, $\theta_2$, $\theta_3$, $\theta_4$, $\theta_5$ and $\theta_6$ in Eq. (1) are defined as follows

$$\theta_1 = \frac{\alpha_i + \alpha_j + \alpha_i\alpha_j}{4(3+\alpha_j)+\alpha_i(4+\alpha_j)} \tag{2a}$$

$$\theta_2 = \frac{\alpha_i(2+\alpha_j)}{4(3+\alpha_j)+\alpha_i(4+\alpha_j)} \tag{2b}$$

$$\theta_3 = \frac{\alpha_j(2+\alpha_i)}{4(3+\alpha_j)+\alpha_i(4+\alpha_j)} \tag{2c}$$

$$\theta_4 = \frac{\alpha_i(3+\alpha_j)}{4(3+\alpha_i)+\alpha_j(4+\alpha_i)} \tag{2d}$$

$$\theta_5 = \frac{\alpha_i\alpha_j}{4(3+\alpha_i)+\alpha_j(4+\alpha_i)} \tag{2e}$$

$$\theta_6 = \frac{\alpha_j(3+\alpha_i)}{4(3+\alpha_i)+\alpha_j(4+\alpha_i)} \tag{2f}$$

in which $\alpha_i$ and $\alpha_j$ are the stiffness indexes, and they can be used to obtain the rotational spring stiffness terms at $i$ and $j$ end of the beam element as follows

$$k_i = \alpha_i \frac{EI}{L} \tag{3a}$$

$$k_j = \alpha_j \frac{EI}{L} \tag{3b}$$

where $k_i$ and $k_j$ are the rotational spring stiffness terms. These parameters ($k_i$ and $k_j$) can change from 0 to $\infty$.

The coefficients in Eq. (2) given for semi-rigid connections may also be identified by connection percentages, and may be represented as follows (Kartal 2004, Filho *et al*. 2004)

$$\theta_1 = \frac{r_i + r_j + r_{ij}}{3} \tag{4a}$$

$$\theta_2 = \frac{2r_i + r_{ij}}{3} \tag{4b}$$

$$\theta_3 = \frac{2r_j + r_{ij}}{3} \tag{4c}$$

$$\theta_4 = r_i \tag{4d}$$

$$\theta_5 = r_{ij} \tag{4e}$$

$$\theta_6 = r_j \tag{4f}$$

where, $r_i$, $r_j$ and $r_{ij}$ are the correction factors, and described as follows

$$r_i = \frac{3v_i}{4 - v_i v_j} \tag{5a}$$

$$r_j = \frac{3v_j}{4 - v_i v_j} \tag{5b}$$

$$r_{ij} = \frac{3v_i v_j}{4 - v_i v_j} \tag{5c}$$

where $v_i$ and $v_j$ are the fixity factors, and represent the semi-rigid connections defined as percentages. If the Eqs. (2) and (4) are equated, a relationship between the rotational spring stiffness and the connection percentage is obtained as follows (Monforton and Wu 1963, Sekulovic and Salatic 2001)

$$k_{i,j} = \frac{3EIv_{i,j}}{(1 - v_{i,j})L} \tag{6a}$$

$$v_{i,j} = \frac{k_{i,j}L}{3EI + k_{i,j}L} \tag{6b}$$

## 3. Calculation of target displacement according to different design codes

### 3.1 Target displacement in ATC-40

ATC-40 is based on the equivalent linearization. According to ATC-40, the capacity spectrum method uses the secant stiffness at maximum displacement to compute the effective period, and relates the effective damping to the area under the hysteresis curve (Fig. 2). The target displacement in the ATC-40 is calculated by

$$\delta_t = C_0 S_d\,(T_{eq}, \beta_{eq}) \tag{7}$$

in which coefficient $C_0$ is the fundamental mode participation factor and $S_d(T_{eq}, \beta_{eq})$ is the maximum displacement of a linearly-elastic SDOF system with equivalent period ($T_{eq}$) and the equivalent damping ratio ($\beta_{eq}$), which are given by

$$T_{eq} = T_0 \sqrt{\frac{\mu}{1 + \alpha\mu - \alpha}} \tag{8}$$

$$\beta_{eff} = \beta_{eq} = \kappa\beta_0 + 5 = \frac{63.7\kappa(a_y d_{pi} - d_y a_{pi})}{a_{pi} d_{pi}} + 5 \tag{9}$$

where $T_0$ is the initial period of vibration of the nonlinear system, $\mu$ is the maximum displacement ductility ratio, $\alpha$ is the post-yield stiffness ratio, $\beta_0$ is the equivalent viscous damping, $\kappa$ is the adjustment factor to approximately account for changes in hysteretic behavior in structure, and $\beta_{eff}$ is the effective viscous damping. Spectral reduction factors, $SR_A$ and $SR_V$, are given by

$$SR_A = \frac{3,21 - 0,68\ln(\beta_{eq})}{2,12} \tag{10}$$

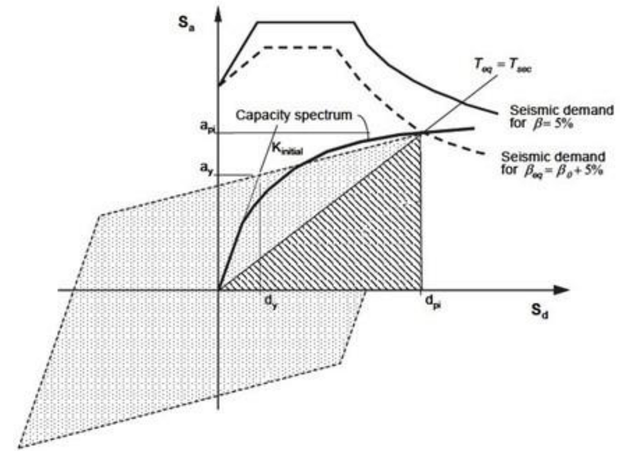



Fig. 2 Capacity-spectrum method of equivalent linearization in ATC-40

$$SR_V = \frac{2,31 - 0,41\ln(\beta_{eff})}{1,65} \tag{11}$$

### 3.2 Target displacement in FEMA-356

The target displacement in the FEMA-356 can be obtained from

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{12}$$

where $S_a$ is the response spectrum acceleration at the effective fundamental period and the damping ratio of the construction under consideration. $g$ is the acceleration of gravity. $T_e$ is the effective fundamental period of building in the direction under consideration, and can be given by

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{13}$$

Here, $K_i$ and $K_e$ are the elastic and effective lateral stiffnesses of the structure, respectively. $K_e$ is obtained by idealizing the pushover curve as a bilinear relationship. $C_0$ is the coefficient to relate the spectral displacement of an equivalent single-degree of freedom (SDOF) structural system to the roof displacement of the multi-degree of freedom (MDOF) building at the control joint. $C_0$ coefficient can either be taken as the first mode participation factor or can be selected from tabulated values in the FEMA-356. $C_1$ is the coefficient factor to relate the expected maximum inelastic and elastic displacements of the SDOF system, and is calculated from

$$C_1 = \begin{cases} 1,0 & for \quad T_e \geq T_S \\ \dfrac{1,0 + (R-1)\,\mathrm{T_S}/\mathrm{T_e}}{R} & for \quad T_e < T_S \\ 1,5 & for \quad T_e < 0,1\,s \end{cases} \tag{14}$$

in which $T_S$ is the characteristic period of the response spectrum. $R$ is a coefficient representing the ratio of elastic and yield strengths, and is obtained by

$$R = \frac{S_a}{V_y / W} C_m \tag{15}$$

where $W$ is the effective seismic weight, $V_y$ is the yield strength of the building obtained from the pushover curve of the structure, and $C_m$ is the effective modal mass factor. $C_2$ is the modification factor to represent the effect of pinched hysteretic shape, stiffness degradation, and strength deterioration on the maximum displacement response. The parameter $C_2$ can be selected from the tabulated values depending on the framing system and structural performance level in FEMA-356. Furthermore, $C_2$ can be taken as 1,0 for nonlinear analysis. $C_3$ is a coefficient to represent increased displacement due to $P$-$\Delta$ effects, and given by

$$C_3 = \begin{cases} 1,0 & \textit{for} \quad \alpha \geq 0 \\ 1,0 + \dfrac{|\alpha|\,(R-1)^{3/2}}{T_e} & \textit{for} \quad \alpha < 0 \end{cases} \tag{16}$$

in which $\alpha$ is the ratio of post-yield stiffness to the effective elastic stiffness. The parameter $\alpha$ can be obtained from idealizing the pushover curve as a bilinear relationship.

### 3.3 Target displacement in FEMA-440

The target displacement in FEMA-440 is obtained by

$$\delta_t = C_0 S_d (T_{eff}, \beta_{eff}) \tag{17}$$

where coefficient $C_0$ is the fundamental mode participation factor, $S_d(T_{eff}, \beta_{eff})$ is the maximum displacement of a linearly-elastic SDOF system with effective period ($T_{eff}$) and the effective damping ratio ($\beta_{eff}$). The improved formulas for the effective period and effective damping ratio in FEMA-440 are given as follows

$$T_{eff} = \begin{cases} \left[0,2(\mu-1)^2 - 0,038(\mu-1)^3 + 1\right] T_0 & \mu < 4,0 \\ \left[0,28 + 0,13(\mu-1) + 1\right] T_0 & 4,0 \leq \mu \leq 6,5 \\ \left[0,89\left(\sqrt{\dfrac{(\mu-1)}{1+0,05(\mu-2)}} - 1\right) + 1\right] T_0 & \mu > 6,5 \end{cases} \tag{18}$$

$$\beta_{eff} = \begin{cases} 4,9(\mu-1)^2 - 1,1(\mu-1)^3 + \beta_0 & \mu < 4,0 \\ 14,0 + 0,32(\mu-1) + \beta_0 & 4,0 \leq \mu \leq 6,5 \\ 19\left[\dfrac{0,64(\mu-1)-1}{0,64(\mu-1)^2}\right]\left(\dfrac{T_{eff}}{T_0}\right)^2 + \beta_0 & \mu > 6,5 \end{cases} \tag{19}$$

### 3.4 Target displacement in TEC-2007

To use the incremental equivalent seismic load method,

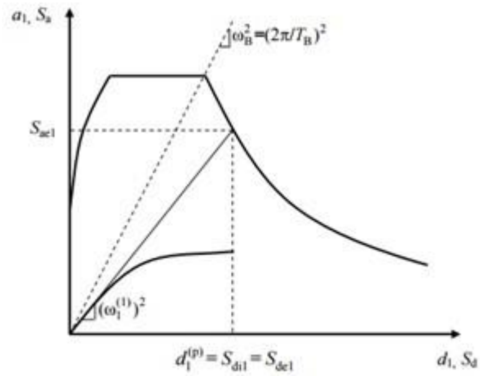



Fig. 3 Capacity and demand curve for $T \geq T_B$

the number of floors of the building excluding the basement should not exceed eight. The torsional irregularity coefficient ($\eta_{bi}$), that is calculated in accordance with the elastic linear behavior without considering additional eccentricity, should meet the condition $\eta_{bi}$<1,4 for each floors. Also, in the earthquake direction considered, the ratio of the effective mass concerning the first (fundamental) vibration mode calculated by considering the linear elastic behavior to the total mass of the building (except for the masses of the basement floors covered by the rigid shear walls) must be greater than 0,70.

The target displacement in TEC-2007 is obtained by

$$u_{N1}^{(p)} = \Phi_{N1} \Gamma_1 d_1^{(p)} \tag{20}$$

where $\Phi_{N1}$ is the amplitude of the first mode in roof of the structure, and $\Gamma_1$ is the fundamental mode participation factor. Modal displacement belonging to the first mode, $d_1^{(p)}$, can be calculated by using the formula as follows

$$d_1^{(p)} = S_{di1} \tag{21}$$

$$S_{de1} = \frac{S_{ae1}}{(\omega_1)^2} \tag{22}$$

where $S_{de1}$ is the linear elastic spectral displacement, $S_{ae1}$ is the spectral acceleration at the fundamental vibration period and the damping ratio of the construction under consideration, $\omega_1$ is the frequency of corresponding period. If the fundamental vibration period from the linear dynamic analysis ($T$) is equal to or greater than the characteristic period of acceleration spectrum ($T_B$), the inelastic spectral displacement ($S_{di1}$) is assumed to be equal to the elastic spectral displacement (Fig. 3). Otherwise, the inelastic spectral displacement is calculated by

$$S_{di1} = C_{R1} S_{de1} \tag{23}$$

$$C_{R1} = \begin{cases} 1,0 & \textit{for} \quad T \geq T_B \\ \dfrac{1 + (R_y - 1) T_B / T}{R_y} & \textit{for} \quad T < T_B \end{cases} \tag{24}$$

where $C_{R1}$ is the spectral displacement ratio. $R_{y1}$ is the reduction coefficient of strength for the first mode and obtained by

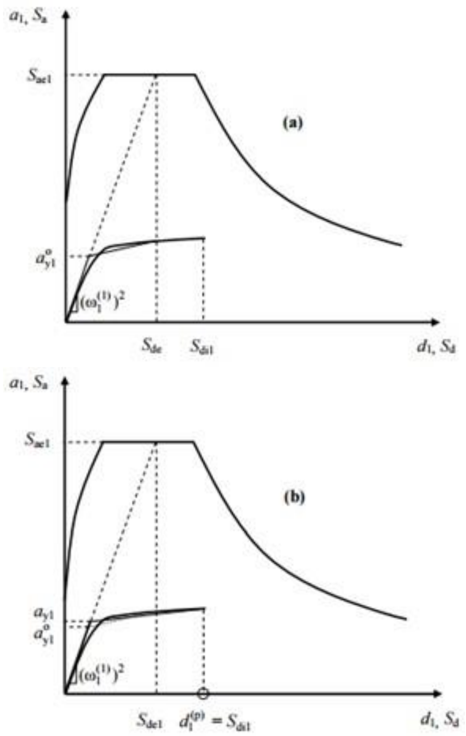



Fig. 4 Capacity and demand curve for $T<T_B$

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{25}$$

in which $a_{y1}$ is calculated from the graphs shown in Fig. 4.

## 4. Numerical application

Prefabricated structures are constructed by bolted connections of separated members. The design and analysis of these structures have been generally performed by defining fully hinges for the connection of separated members at the joint of junction. In practice, these connections are not fully hinged. Therefore, assumption of semi-rigid connections (partially fixity) instead of fully hinge connections is a more realistic approach for bolted connections used in the prefabricated elements. Therefore, the aim of this study is to investigate the effects of semi-rigid connections on seismic performance of prefabricated structures. Nonlinear static analysis (pushover analysis) of a selected RC prefabricated structure is performed using SAP2000 structural analysis program by considering various connection percentages for bolted connections. The self-weight of the structure and $P$-$\Delta$ effects are included in the analyses.

A three-dimensional view and plan view of the selected RC prefabricated structure are shown in Figs. 5-6, respectively. The structure has two bays in the $x$-direction with 20,7 m. The height of the structure is 8,10 m. The system has six bays in the $y$-direction. Total area covered by the structure is 1876,66 $m^2$.

The RC prefabricated structure is constructed in city of Kayseri in Turkey. The city is located in Earthquake Zone 3.

Fig. 5 A three dimensional view of the selected RC prefabricated structure

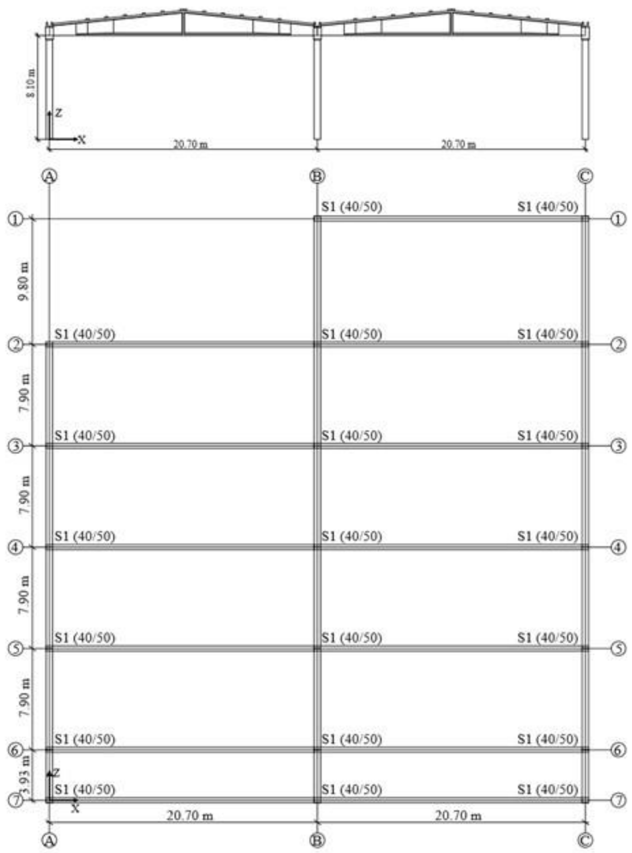
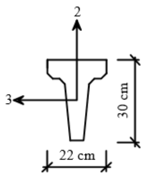
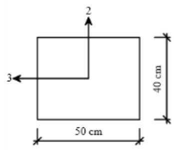
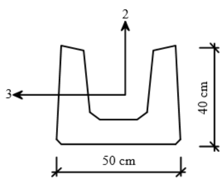



Fig. 6 Plan view of the selected RC prefabricated structure

This structure was built on soil class Z3. According to TEC-2007, the design ground acceleration of the zone is 0,2 g, and the characteristic periods ($T_A$ and $T_B$) for soil class Z3 are 0,15 and 0,60 seconds. All frame elements in the prefabricated structure were designed according to the requirements of TEC-2007. The concrete and reinforcing steel classes are considered as C30 ($f_{ck}$=30 MPa) and S420 ($f_{yk}$=420 MPa), respectively. The Young's modulus and the weight per unit volume of concrete is $32\times10^6$ kN/m$^2$ and 25 kN/m$^3$, respectively.

The connection percentages for bolted connections of the selected RC prefabricated structure and the equivalent rotational spring stiffness calculated by Eq. (6) are given in

Table 1 Connection properties for bolted connections of the prefabricated structural elements for local axis 2 in SAP2000

| Frame Element Type | | Length (m) | Moment of Inertia ($cm^4$) | Connection Percentage (%) | Rotational Spring Stiffness (kNm/rad) |
|---|---|---|---|---|---|
|  | Purlin Beam-1 | 3,93 | 5.996 | 0 | 0 |
| | | | | 25 | 488,223 |
| | | | | 50 | 1.464,671 |
| | | | | 75 | 4.394,015 |
| | | | | 100 | ∞ |
| | Purlin Beam-2 | 7,90 | 5.996 | 0 | 0 |
| | | | | 25 | 242,875 |
| | | | | 50 | 728,627 |
| | | | | 75 | 2.185,883 |
| | | | | 100 | ∞ |
| | Purlin Beam-3 | 9,80 | 5.996 | 0 | 0 |
| | | | | 25 | 195,787 |
| | | | | 50 | 587,363 |
| | | | | 75 | 1.762,089 |
| | | | | 100 | ∞ |
|  | Short Column | 0,02 | 266.700 | 0 | 0 |
| | | | | 25 | 4.267.200,000 |
| | | | | 50 | 12.801.600,000 |
| | | | | 75 | 38.404.800,000 |
| | | | | 100 | ∞ |
|  | Gutter Beam-1 | 3,93 | 148.500 | 0 | 0 |
| | | | | 25 | 25.046,310 |
| | | | | 50 | 75.138,931 |
| | | | | 75 | 225.416,793 |
| | | | | 100 | ∞ |
| | Gutter Beam-2 | 7,90 | 148.500 | 0 | 0 |
| | | | | 25 | 12.459,746 |
| | | | | 50 | 37.379,240 |
| | | | | 75 | 112.137,721 |
| | | | | 100 | ∞ |
| | Gutter Beam-3 | 9,80 | 148.500 | 0 | 0 |
| | | | | 25 | 10.044,081 |
| | | | | 50 | 30.132,244 |
| | | | | 75 | 90.396,734 |
| | | | | 100 | ∞ |

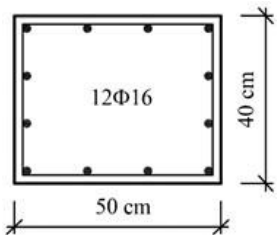



Fig. 7 The cross-section and reinforcement details of the columns

Tables 1-2 for the local 2 and 3 axes in SAP2000 program.

The cross-section dimension of the columns is 40×50 cm (Fig. 7). Longitudinal bars in all columns are 12Φ16. The confinement bars are 64Φ8.

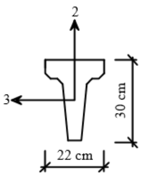
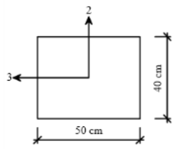
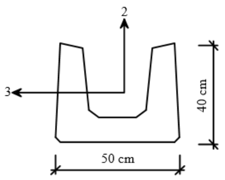
The columns are considered as non-linear frame elements, whereas the beams of prefabricated structure are considered as linear frame elements. The stress-strain relationship of the confined and unconfined concrete (Mander *et al*. 1988) and steel material models used in nonlinear analysis are given in Fig. 8.

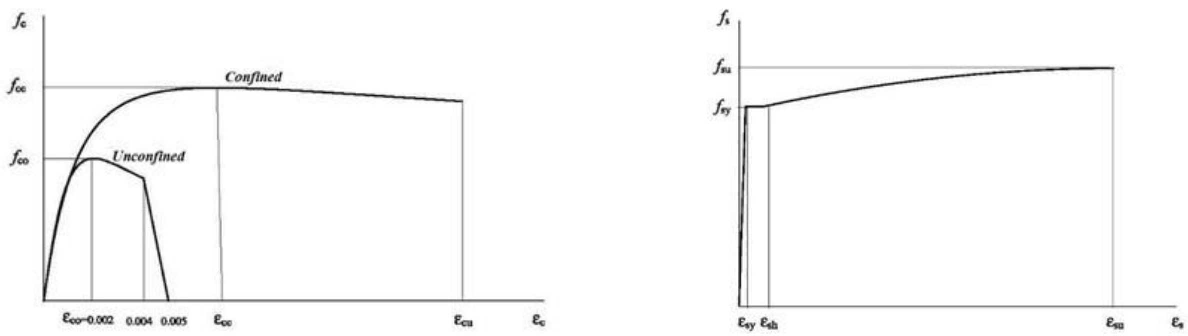
It is assumed that plastic hinges occur only at the lower end of columns of the selected RC prefabricated structure by taking into account the damages previously occurred in the prefabricated buildings during earthquakes. The plastic hinges is defined as P-M-M hinge in SAP2000 depending on the bar distributions at the cross-section of columns given in Fig. 7.

Initial effective bending stiffnesses $(EI)_e$ of the cracked sections has been calculated for the nonlinear static analysis. The calculated values are assigned at the related column elements. Thus, the reduction at the stiffness of the frame sections due to plastic deformations and cracking of concrete is taken into account in the nonlinear analysis.

The fundamental periods of the selected RC prefabricated structure with uncracked and cracked sections

Table 2 Connection properties for bolted connections of the prefabricated structural elements for local axis 3 in SAP2000

| Frame Element Type | | Length (m) | Moment of Inertia (cm$^4$) | Connection Percentage (%) | Rotational Spring Stiffness (kNm/rad) |
|---|---|---|---|---|---|
| | | | | 0 | 0 |
| | | | | 25 | 2.053,536 |
| | Purlin Beam-1 | 3,93 | 25.220 | 50 | 6.160,610 |
| | | | | 75 | 18.481,832 |
| | | | | 100 | ∞ |
| | | | | 0 | 0 |
| | | | | 25 | 1.021,569 |
|  | Purlin Beam-2 | 7,90 | 25.220 | 50 | 3.064,708 |
| | | | | 75 | 9.194,126 |
| | | | | 100 | ∞ |
| | | | | 0 | 0 |
| | | | | 25 | 823,510 |
| | Purlin Beam-3 | 9,80 | 25.220 | 50 | 2.470,530 |
| | | | | 75 | 7.411,591 |
| | | | | 100 | ∞ |
| | | | | 0 | 0 |
| | | | | 25 | 6.667.200,000 |
|  | Short Column | 0,02 | 416.700 | 50 | 20.001.600,000 |
| | | | | 75 | 60.004.800,000 |
| | | | | 100 | ∞ |
| | | | | 0 | 0 |
| | | | | 25 | 12.091,603 |
| | Gutter Beam-1 | 3,93 | 148.500 | 50 | 36.274,809 |
| | | | | 75 | 108.824,427 |
| | | | | 100 | ∞ |
| | | | | 0 | 0 |
| | | | | 25 | 6.015,189 |
|  | Gutter Beam-2 | 7,90 | 148.500 | 50 | 18.045,569 |
| | | | | 75 | 54.136,708 |
| | | | | 100 | ∞ |
| | | | | 0 | 0 |
| | | | | 25 | 4.848,979 |
| | Gutter Beam-3 | 9,80 | 148.500 | 50 | 14.546,938 |
| | | | | 75 | 43.640,816 |
| | | | | 100 | ∞ |




(a) Concrete material model (b) Steel material model

Fig. 8 Concrete material model (a) and steel material model (b) used for nonlinear analysis

are obtained for *x* and *y*-directions, and given in Table 3. It is shown from Table 3 that the fundamental periods in *x*-direction decrease from 1,0494 sec to 0,8844 sec for the structure with uncracked sections and from 1,6457 sec to 1,2298 sec for the structure with cracked sections with the connection percentages increasing from 0% to 100%. Similarly, the fundamental periods in y-direction decrease from 2,5189 sec to 1,1427 sec for the structure with

Table 3 Fundamental periods of the selected RC prefabricated structure with uncracked and cracked sections in *x* and *y*-directions

| Connection Percentange (%) | Fundamental Periods (sec) | | | |
|---|---|---|---|---|
| | *x*-direction | | *y*-direction | |
| | Uncracked Section | Cracked Section | Uncracked Section | Cracked Section |
| 0 | 1,0494 | 1,6457 | 2,5189 | 3,0801 |
| 25 | 0,9219 | 1,2866 | 1,6622 | 1,9956 |
| 50 | 0,8964 | 1,2435 | 1,3660 | 1,5778 |
| 75 | 0,8900 | 1,2363 | 1,2480 | 1,4580 |
| 100 | 0,8844 | 1,2298 | 1,1427 | 1,3612 |

uncracked sections and from 3,0801 sec to 1,3612 sec for the structure with cracked sections with the connection percentages increasing from 0% to 100%. Comparing the fundamental periods of the selected RC prefabricated structure with uncracked and cracked sections in *x* and *y*-directions, it is apparent that the stiffness of the structure increases due to the semi-rigid connections.

## 5. Evaluation of target roof displacements and base shear forces

The seismic demands of a structure are often computed by nonlinear static analysis of the structure subjected to monotonically increasing lateral forces until the target value of roof displacement is reached. This roof displacement is estimated from the earthquake-induced deformation of an inelastic SDOF system derived from the pushover curve (Chintanapakdee and Chopra 2003, Goel 2011). Therefore, the seismic performance of a structure with nonlinear static analysis is evaluated by the target roof displacement of the structure.

In this section, the target roof displacements of the selected RC prefabricated structure are calculated to investigate the effects of semi-rigid connections on seismic performance of prefabricated structures. The nonlinear analysis procedures proposed by the codes of ATC-40, FEMA-356, FEMA-440, and TEC-2007 are considered in the calculations. The nonlinear static analysis (pushover analysis) of the selected RC prefabricated structure is performed using SAP2000 structural analysis program. In the analyses, the connection percentages given in Table 1 and 2 for bolted connections of the structure are considered.

The target roof displacement of the selected prefabricated structure is determined by using the nonlinear analysis procedures presented in ATC-40, FEMA-356, FEMA-440, and TEC-2007. The required coefficients to calculate target roof displacements of the selected RC prefabricated structure for nonlinear static analysis in the codes of ATC-40, FEMA-356, FEMA-440, and TEC-2007 are given in Table 4.

The pushover curves are obtained for dead loads and a unit seismic load by considering the connection percentages of 0% (hinged connection), 25%, 50%, 75%, and 100% (rigid connection) for bolted connections of the selected RC

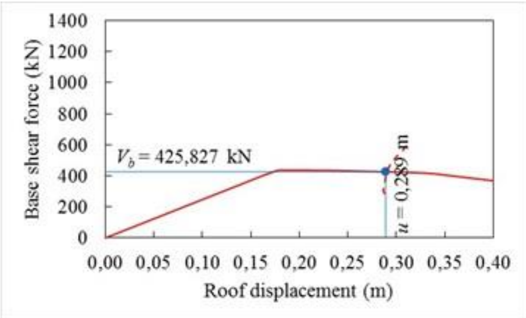
Table 4 The coefficients required to calculate target roof displacements of the selected RC prefabricated structure for nonlinear static analysis in ATC-40, FEMA-356, FEMA-440, and TEC-2007

| Coefficients | | | ATC-40 | FEMA-356 | FEMA-440 | TEC-2007 |
|---|---|---|---|---|---|---|
| $C_0$ | Connection Percentage (%) | 0 | - | 1,3469 | - | - |
| | | 25 | | 1,1063 | | |
| | | 50 | | 1,0655 | | |
| | | 75 | | 1,0605 | | |
| | | 100 | | 1,0556 | | |
| $T_{eff}$ | Connection Percentage (%) | 0 | 2,9830 | - | - | - |
| | | 25 | 1,8420 | | | |
| | | 50 | 1,3510 | | | |
| | | 75 | 1,2650 | | | |
| | | 100 | 1,2130 | | | |
| $T_{sec}$ | Connection Percentage (%) | 0 | - | - | 3,0410 | - |
| | | 25 | | | 1,8790 | |
| | | 50 | | | 1,3780 | |
| | | 75 | | | 1,2910 | |
| | | 100 | | | 1,2370 | |
| $\beta_{eff}$ | Connection Percentage (%) | 0 | 0,1540 | - | 0,0620 | - |
| | | 25 | 0,1340 | | 0,0640 | |
| | | 50 | 0,1260 | | 0,0630 | |
| | | 75 | 0,1280 | | 0,0640 | |
| | | 100 | 0,1290 | | 0,0640 | |
| $C_1$ | | | - | 1,0 | - | - |
| $C_2$ | | | - | 1,1 | - | - |
| $C_3$ | | | - | 1,0 | - | - |
| $C_{R1}$ | | | - | - | - | 1,0 |

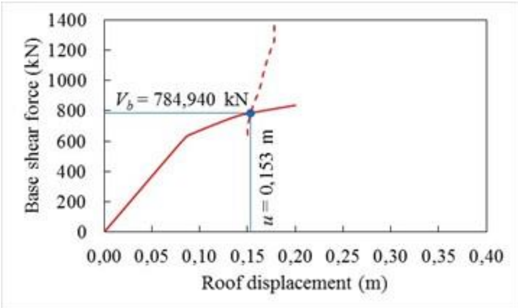
prefabricated structure. These curves are obtained for the selected RC prefabricated structure subjected to the design earthquake in *y*-*y* direction. Here, the design earthquake is the earthquake which has probability of exceedance in 50 years is 10%.

The pushover curves are presented in Figs. 9-12 for ATC-40, FEMA-356, FEMA-440 and TEC-2007 are presented, respectively. In addition, the target roof displacements and the base shear forces of the selected RC prefabricated structure obtained for ATC-40, FEMA-356, FEMA-440, and TEC-2007 are given in Table 5.

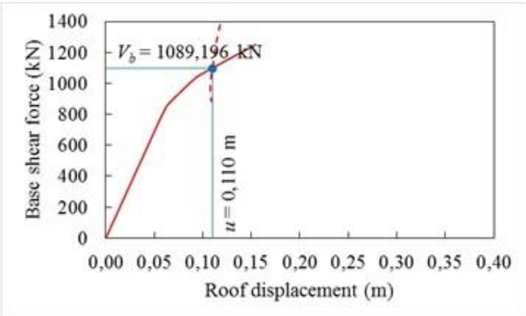
It is shown from Figs. 9-12 and Table 5 that the target roof displacements decrease from 0,289 m to 0,097 m for ATC-40, from 0,364 m to 0,122 m for FEMA-356, from 0,304 m to 0,102 m for FEMA-440, and from 0,320 m to 0,119m for TEC-2007 with the connection percentages increasing from 0% to 100%. The target roof displacements obtained from the selected RC prefabricated structure with fully hinged connections are greater than those from the selected RC prefabricated structure with semi-rigid connections. Therefore, the selected RC prefabricated structure with fully hinged connections behaves more flexible than that with semi-rigid connections. In fact, the prefabricated structures have semi-rigid connections, and they cannot be as flexible as the hinged connected structures.



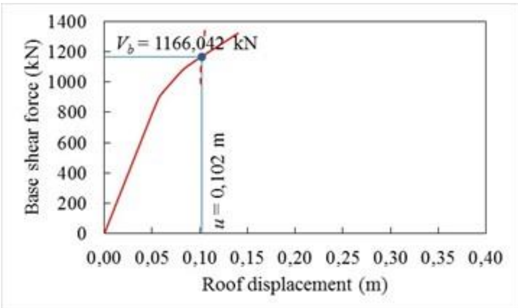

(a) The connection percentage of 0%



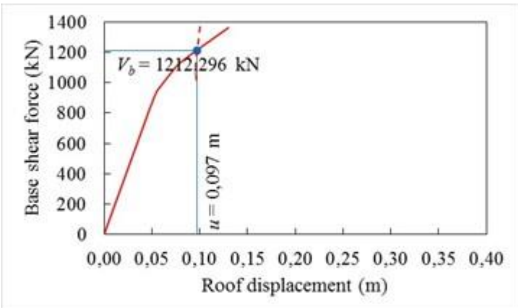

(b) The connection percentage of 25%




(c) The connection percentage of 50%

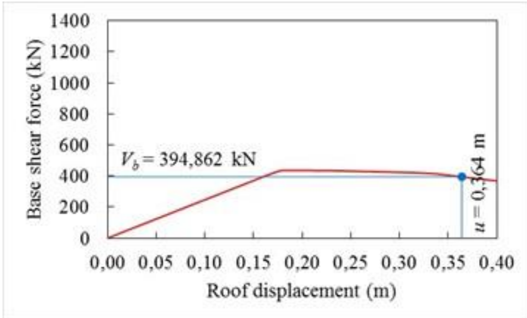



(d) The connection percentage of 75%

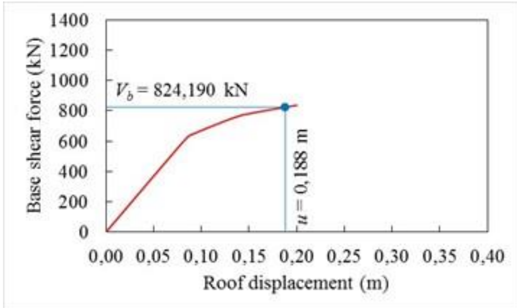



(e) The connection percentage of 100%

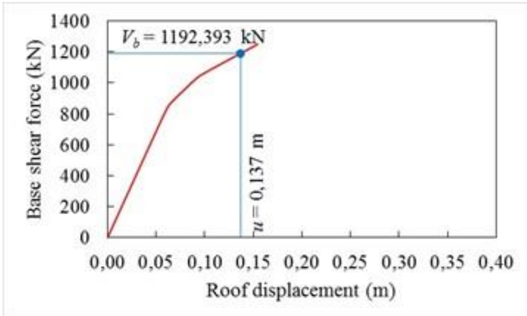
Fig. 9 Pushover curves for ATC-40 by considering the connection percentages of 0% (hinged connection), 25%, 50%, 75%, and 100% (rigid connection)



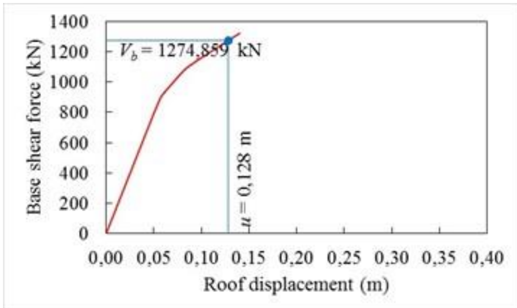

(a) The connection percentage of 0%



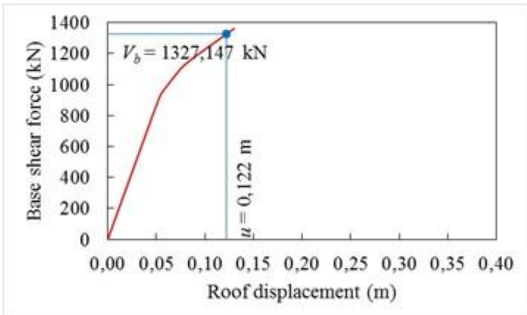

(b) The connection percentage of 25%




(c) The connection percentage of 50%

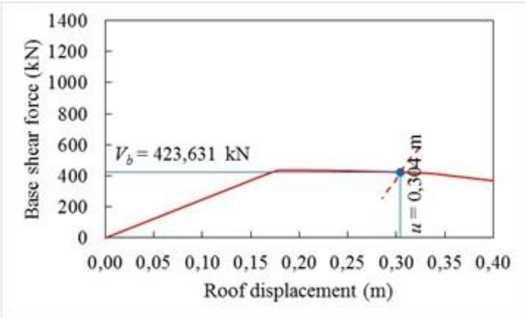



(d) The connection percentage of 75%

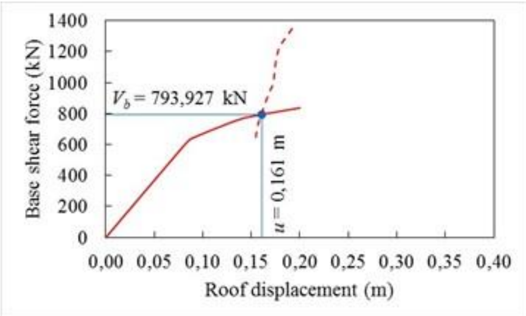



(e) The connection percentage of 100%

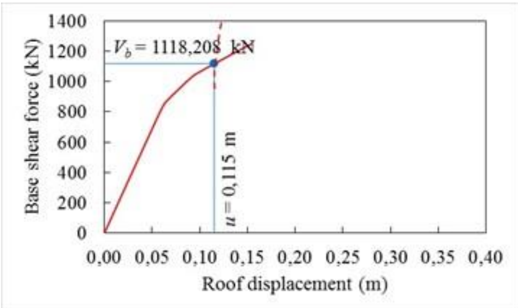
Fig. 10 Pushover curves for FEMA-356 by considering the connection percentages of 0% (hinged connection), 25%, 50%, 75%, and 100% (rigid connection)



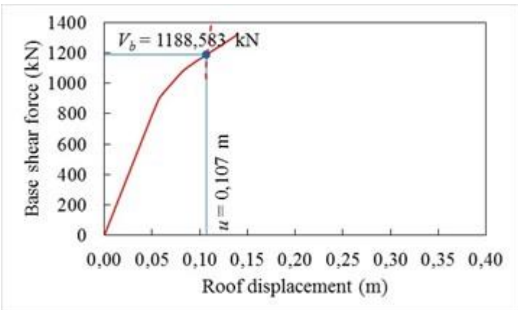

(a) The connection percentage of 0%



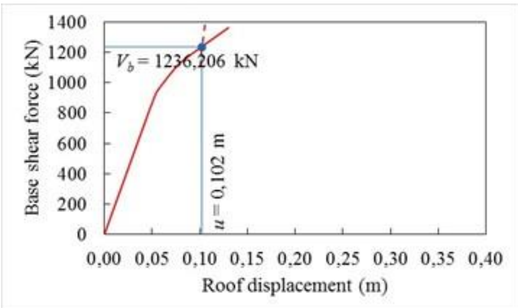

(b) The connection percentage of 25%




(c) The connection percentage of 50%

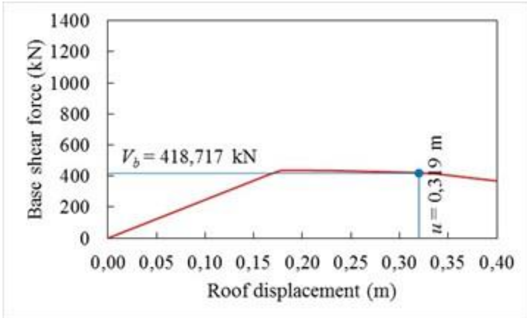



(d) The connection percentage of 75%

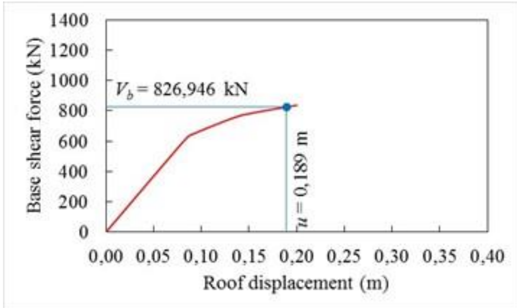



(e) The connection percentage of 100%

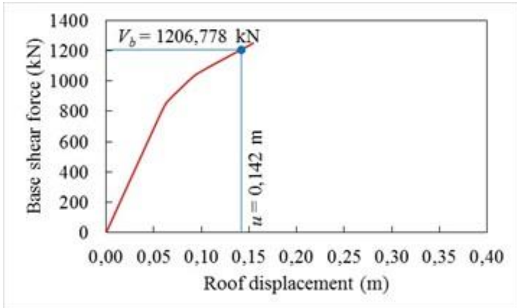
Fig. 11 Pushover curves for FEMA-440 by considering the connection percentages of 0% (hinged connection), 25%, 50%, 75%, and 100% (rigid connection)



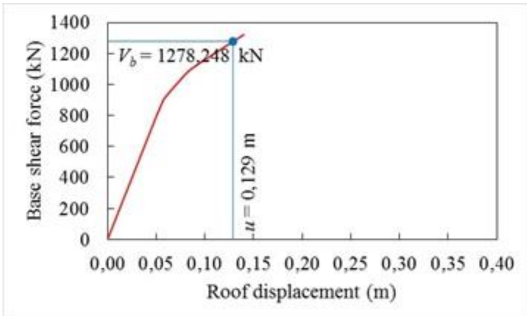

(a) The connection percentage of 0%



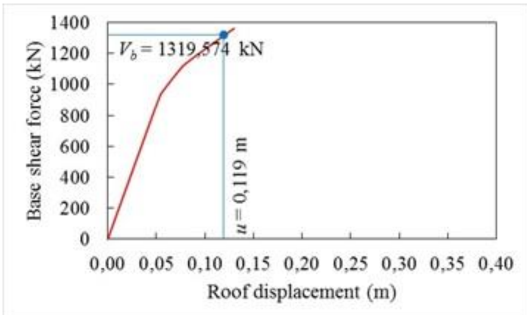

(b) The connection percentage of 25%




(c) The connection percentage of 50%




(d) The connection percentage of 75%




(e) The connection percentage of 100%

Fig. 12 Pushover curves for TEC-2007 by considering the connection percentages of 0% (hinged connection), 25%, 50%, 75%, and 100% (rigid connection)

Table 5 Target roof displacements (TRDs in meter) and base shear forces (BSFs in kN) obtained for ATC-40, FEMA-356, FEMA-440, and TEC-2007

| | | Nonlinear Static Procedure | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | ATC-40 | | FEMA-356 | | FEMA-440 | | TEC-2007 | |
| | | TRDs | BSs | TRDs | BSs | TRDs | BSs | TRDs | BSs |
| Connection percentages (%) | 0 | 0,289 | 425,827 | 0,364 | 394,862 | 0,304 | 423,631 | 0,320 | 418,717 |
| | 25 | 0,153 | 784,940 | 0,188 | 824,190 | 0,161 | 793,927 | 0,189 | 826,946 |
| | 50 | 0,110 | 1089,196 | 0,137 | 1192,393 | 0,115 | 1118,208 | 0,142 | 1206,778 |
| | 75 | 0,102 | 1166,042 | 0,128 | 1274,859 | 0,107 | 1188,583 | 0,129 | 1278,248 |
| | 100 | 0,097 | 1212,296 | 0,122 | 1327,147 | 0,102 | 1236,206 | 0,119 | 1319,574 |

In contrast to the target roof displacements, the base shear forces increase from 425,827 kN to 1212,296 kN for ATC-40, from 394,862 kN to 1327,147 kN for FEMA-356, from 423,631 kN to 1236,206 kN for FEMA-440, and from 418,717 kN to 1319,574 kN for TEC-2007 with the connection percentages increasing from 0% to 100%. The increase in the base shear forces can be seen by comparison of Figs. 9-12, and also from Table 5. In addition, according to Table 5, for the smallest connection percentage (25%), the base shear forces increase 84% for ATC-40, 109% for FEMA-356, 87% for FEMA-440, and 97% for TEC-2007 compared to fully hinged connection (0%). All comparisons are concluded that the base shear forces obtained from the selected RC prefabricated structure with fully hinged connections are significantly smaller than those from the structure with semi-rigid connections. This situation points out that unexpected damages due to the earthquakes may be occurred at intersections of the column-foundation.

According to the results given by Figs. 9-12, and Table 5, the fully hinged-connection assumption may not be suitable for the prefabricated structures. It is also seen that the prefabricated structures are subject to higher base shear forces. For this reason, in the design and analysis of the prefabricated structures, it should be considered the effects of semi-rigid connections.

## 5. Conclusions

In this study, the effects of semi-rigid connections on seismic performance of prefabricated structures are investigated. Nonlinear static analyses (pushover analysis) of a selected RC prefabricated structure are performed with SAP2000 structural analysis program by considering various connection percentages (0%, 25%, 50%, 75%, and 100%) for bolted connections. Pushover curves of the selected RC prefabricated structure with the connection percentages are obtained from the nonlinear static analysis. The target roof displacements and the base shear forces obtained from the pushover curves according to ATC-40, FEMA-356, FEMA-440, and TEC-2007 codes are compared each other. It can be reached the following conclusions:

• The prefabricated structures cannot be as flexible as the hinged connected structures since they have semi-rigid connections, in fact.

• Comparing the fundamental periods of the selected RC prefabricated structure with uncracked and cracked sections in *x* and *y*-directions, it is apparent that the stiffness of the structure increases due to the semi-rigid connections.

• Nonlinear procedures in ATC-40, FEMA-356, FEMA-440, and TEC-2007 are consistent with each other.

• All comparisons show that the base shear forces obtained from the selected RC prefabricated structure with hinged connections are significantly smaller than those from the structure with semi-rigid connections. This situation points out that unexpected damages due to the earthquakes can be occurred at the column-foundation intersections.

• It is seen that the prefabricated structures are subject to higher base shear forces. Therefore, the hinged-connection assumption may not be suitable for the prefabricated structures.

Finally, the effects of semi-rigid connections should be considered in design and analysis of the prefabricated structures.

## References

Arslan, M.H., Korkmaz, H.H. and Gülay, F.G. (2006), "Damage and failure pattern of prefabricated structures after major earthquakes in turkey and shortfalls of the turkish earthquake code", *Eng. Fail. Anal.*, **13**, 537-557.

ASCE 41-06 (2007), Seismic Rehabilitation of Existing Buildings, American Society of Civil Engineers, Washington, USA.

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

Cavdar, O. and Bayraktar, A. (2014), "Pushover and nonlinear time history analysis evaluation of a RC building collapsed during the Van (Turkey) earthquake on october 23, 2011", *Nat. Hazard.*, **70**, 657-673.

Chen, W.F. and Lui, E.M. (1991), *Stability Design of Steel Frames*, CRC Press Inc., Taylaor and Francis Group, Florida, USA.

Deyanova, M., Pampanın, S. and Nascımbene, R. (2014), "Assessment of single-storey precast concrete industrial buildings with hinged beam-column connection with and without dowels", *Proceedings of the 2nd European Conference on Earthquake Engineering and Seismology*, Istanbul, Turkey, August.

Doğan, M., Özbaşaran, H. and Günaydın, A. (2010), "Effects of seismic loading to prefabricated connections", *Anadolu Univ. J. Sci. Technol.*, **11**, 47-58.

FEMA-356 (2000), Prestandart and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency, Washington, USA.

FEMA-440 (2005), Improvement of Nonlinear Static Seismic Analysis Procedures, Federal Emergency Management Agency, Washington, USA.

Filho, M.S., Guimarães, M.J.R., Sahlit, C.L. and Brito, J.L.V. (2004), "Wind pressures in framed structures with semi-rigid connections", *J. Brazil. Soc. Mech. Sci. Eng.*, **26**(2), 180-189.

Goel, R.K. (2011), "Variability and accuracy of target displacement from nonlinear static procedures", *ISRN Civil Eng.*, **2011**, Article ID 582426, 16.

Gunawardena, T., Ngo, T. and Mendis, P. (2016), "Behaviour of multi-storey prefabricated modular buildings under seismic loads", *Earthq. Struct.*, **11**(6), 1061-1076.

Kartal, M.E. (2004), "The effect of partial fixity at nodal points on the behaviour of the truss and prefabricated structures", MSc. Dissertation, Zonguldak Karaelmas University, Zonguldak, Turkey. (in Turkish)

Kartal, M.E., Başağa, H.B., Bayraktar, A. and Muvafık, M. (2010), "Effects of semi-rigid connection on structural responses", *Elec. J. Struct. Eng.*, **10**, 22-35.

Magliulo, G., Fabbrocino, G. and Manfredi, G. (2008), "Seismic assessment of existing precast industrial buildings using static and dynamic nonlinear analyses", *Eng. Struct.*, **30**(9), 2580-2588.

Mander, J.B. Priestly, M.J.N. and Park, R. (1988), "Theoretical stress-strain model for confined concrete", *J. Struct. Div.*, ASCE, **114**(8), 1804-1826.

McGuire, W., Gallagher, R.H. and Ziemian, R.D. (1999), *Matrix Structural Analysis*, 2nd Edition, John Wiley & Sons Inc., USA.

Monforton, G.R. and Wu, T.S. (1963), "Matrix analysis of semi-rigidly connected frames", *J. Struct. Div.*, ASCE, **89**(ST6), 13-42.

Nguyen, P. and Kim, S. (2014), "Nonlinear inelastic time-history of three-dimensional semi-rigid steel frames", *J. Constr. Steel Res.*, **101**, 192-206.

Popa, V. and Cotofana, D. (2009), "Probabilistic seismic assessment of single story precast concrete buildings", *Proceedings of the 11th WSEAS International Conference on Sustainability in Science Engineering*, Timişoara, Romania, May.

SAP2000 (2015), Structural Analysis Program, Computers and Structures Inc., Version: 17.1.1, Berkeley, California, USA.

Sarkar, P., Prasad, A.M. and Menon, D. (2016), "Seismic evaluation of rc stepped building frames using improved pushover analysis", *Earthq. Struct.*, **10**(4), 913-938.

Sekulovic, M. and Salatic, R. (2001), "Nonlinear analyses of frames with flexible connections", *Comput. Struct.*, **79**, 1097-1107.

Senel, S.M. and Kayhan, A.H. (2010), "Fragility based damage assessment in existing precast industrial buildings: a case study for Turkey", *Struct. Eng. Mech.*, **34**(1), 39-60.

TEC-2007 (2007), Ministry of Public Works and Settlement of Republic of Turkey, Turkish Earthquake Code, Ankara, Turkey.

Tehranizadeh, M., Amirmojahedi, M. and Moshref, A. (2016), "Simplified methods for seismic assessment of existing buildings", *Earthq. Struct.*, **10**(6), 1405-1428.

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