Structural Engineering and Mechanics, *Vol. 48, No. 2 (2013) 257-274*
DOI: http://dx.doi.org/10.12989/sem.2013.48.2.257

Rapid seismic performance assessment method for one story hinged precast buildings

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(Received November 6, 2012, Revised September 13, 2013, Accepted October 2, 2013)

Abstract. In this study, seismic performance of one story hinged precast buildings, which represents the majority of existing lightweight industrial building stock of Turkey, was assessed. A lot of precast buildings, constructed in one of the important seismic zones of western Turkey, were investigated and building inventories were prepared. By this method, structural properties of inventory buildings and damaged precast buildings in recent earthquakes were compared. Damage estimations based on nonlinear analysis methods have shown that estimated damage levels of inventory buildings and observed damage levels in recent earthquakes are similar. Accuracy of damage estimation study and the simplicity of the one story precast building models implied that rapid seismic performance assessment method for these buildings can be developed. In this assessment method, capacity curves and vibration periods of precast buildings were calculated by using structural properties of precast buildings. The proposed assessment method was applied to inventory buildings by using two different seismic demand scenarios which reflect moderate and soft soil conditions. Comparison of detailed analysis and rapid assessment methods have indicated that reliable seismic performance estimations can be performed by using proposed method. It is also observed that distribution of damage estimations is compatible in both scenarios.

Keywords: precast industrial buildings; nonlinear analysis; seismic performance; damage estimation; rapid evaluation

1. Introduction

In Turkey, precast buildings were mostly constructed as one storey hinge jointed at the beginning of 80's owing to private investments and these structures were imitated from European construction types without questioning seismic hazards. Seismic performance of these buildings after Adana-Ceyhan earthquake in 1998, a year later Kocaeli and Duzce earthquakes was poor and they had excessive damages by various reasons (Zorbozan *et al.* 1998, Saatcioglu *et al.* 2001, Posada and Wood 2002, Tezcan and Colakoglu 2003, Arslan *et al.* 2006, Sezen and Whittaker 2006). It should be also stated that direct and indirect economic losses were reported in more than half industrial facilities due to damages after Kocaeli Earthquake (Cruz and Steinberg 2005). While studies on these structures were performing after devastating earthquakes, the other hand

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http://www.techno-press.org/?journal=sem&subpage=8 ISSN: 1225-4568 (Print), 1598-6217 (Online)

Turkish Earthquake Code (TEC)-1998 (TEC 1998) was prevailed. Thus, calculations of reinforced precast buildings were first defined. Later, former code was updated and current Turkish Earthquake Code (TEC)-2007 (TEC 2007) was introduced.

However, it is unfortunate that most of industrial buildings were constructed lack of improved codes during the aforementioned process in Turkey. Furthermore, remember that 98% of industrial regions are located in seismically active regions (Adalier and Aydingun 2001), economic losses are becoming more apparent for Turkey. Moreover, when lightweight industrial facilities are highly constructed by precast member is considered (Karaesmen 2001), rapid performance assessment of precast buildings becomes essential topic in Turkey. Kayhan (2004) investigated the seismic performance of existing precast buildings by using empirical expressions which were derived from the analysis of theoretical building models.

Further studies based on existing precast buildings were performed by Senel and Palanci (2013) in one of the important seismic zones of western Turkey, Denizli Organized Industrial Zone (DOIZ). Palanci (2010) proposed rapid evaluation method (REM) to investigate the performance of precast structures by considering detailed analysis method results and structural properties of existing building stock in Turkey.

In this study, the performance of proposed method is examined by two different seismic demand scenarios and rapid assessment method is applied on a sample existing building in DOIZ. Evaluation of analysis and REM damages indicates that distribution of damages is compatible in both scenarios. Observations have also shown that it is possible to make reliable damage estimations by proposed rapid evaluation method.

2. Structural properties of existing precast structures

Inventory study has shown that 203 industrial buildings are still in function in DOIZ. The other observation shows that 49 buildings are constructed as monolithic R/C or steel buildings. The rest of 154 precast buildings represent the great majority of industrial buildings (76%) and 102 of them are one story precast buildings among all precast buildings. Observations have also shown that all one-story buildings were constructed by using hinged connections. It should be stated the observations are compatible with studies performed by various researchers (Karaesmen 2001, Ersoy *et al.* 1995) after Adana–Ceyhan, Kocaeli and Duzce earthquakes. In Fig. 1, sketch of typical one story buildings which represents great majority of existing building stock is illustrated.

During the inventory study, structural properties such as building heights, bay widths, cross sectional dimensions, longitudinal and transverse reinforcement ratios were prepared. In the inventory study, 98 of 102 single story buildings were inspected. It is also observed that each of building has identical frame characteristics. So, each building is represented by two-dimensional frame during the statistical evaluation. Further details can be found in the study performed by Palanci (2010).

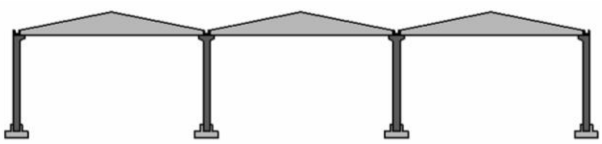


Fig. 1 Typical configuration of one story precast buildings

3. Seismic performance assessment in existing precast buildings

During the building performance assessment study, capacity curves of 98 single story precast buildings were constructed. Capacity curve of each building was obtained by combining the individual force and displacement response of columns. Non-linear response of cantilever columns was determined by moment curvature analysis. During the calculations, lumped plasticity approach was used and plastic hinges were placed at the bottom of columns.

3.1 Construction of building capacity curves

In Fig. 2, moment and curvature profile along the columns is illustrated. Moment and curvature response at plastic hinge regions are also shown on a typical frame model.

Confined and unconfined concrete behaviors are represented by Modified Kent-Park Method (Park *et al.* 1982). Maximum strain expression (Eq. (1)) that is also expressed in current TEC-2007 was used for compressive strain capacity of concrete and limited with 0.018. Strain capacity of longitudinal bars (ε_{su}) was taken as 0.06 by considering the possible buckling of longitudinal reinforcement. Maximum curvature capacity of section (ϕ_{CP}) is obtained by concrete compressive strain or steel longitudinal strain whichever occurs first.

$$(\varepsilon_{cu}) = 0.004 + 0.014(\frac{\rho_s}{\rho_{sm}}) \leq 0.018 \tag{1}$$

In Fig. 3, the first yield of column that corresponds to concrete strain of 0.002 and yield strain of steel are represented by notations ϕ_y and M_y. Flexural strength of column section M_{ny} was determined when concrete strain reached to value of 0.004. Nominal curvature of ϕ_{ny} was obtained by considering the proportion of moments. Further details of the procedure may be found by Priestley *et al.* (2007). Schematical representation of the procedure is also shown in Fig. 3. Plastic curvature capacity was determined by using Eq. (2) after calculation of ultimate and yield curvature capacity of columns. Intermediate levels "Immediate Occupancy" (ϕ_{IO}) and "Life Safety" (ϕ_{LS}) limits were also calculated by using Eqs. (3)-(4).

$$\phi_{pl} = \phi_{CP} - \phi_{ny} \tag{2}$$

$$\phi_{IO} = \phi_{ny} + 0.1 \cdot \phi_{pl} \tag{3}$$

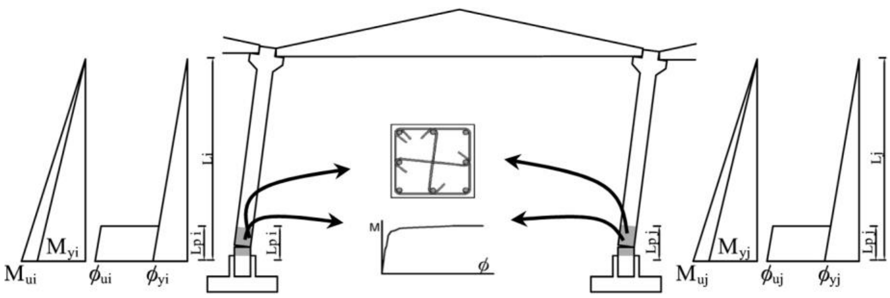


Fig. 2 Moment-curvature response of columns from typical precast frame model

$$\phi_{LS} = \phi_{ny} + \tfrac{2}{3} \cdot \phi_{pl} \tag{4}$$

Later on, member displacement capacities were calculated by using moment area theorem as given in Eqs. (5)-(8). During the calculation of plastic displacement capacity of precast columns, plastic hinge length (L_p) was taken as half of the section height as suggested in TEC-2007. Shear deformations were neglected by considering the slenderness of these structures. Strength capacity of each precast column was calculated by using Eq. (9).

$$\delta_{y_i} = \frac{\phi_{ny_i} \cdot L_i^{\,2}}{3} \tag{5}$$

$$\delta_{CP\,i} = \delta_{y_i} + (\phi_{CP\,i} - \phi_{ny_i}) L_{p_i} \left(L_i - \frac{L_{p_i}}{2} \right) \tag{6}$$

$$\delta_{LS\,i} = \delta_{y_i} + (\phi_{LS\,i} - \phi_{ny_i}) L_{p_i} \left(L_i - \frac{L_{p_i}}{2} \right) \tag{7}$$

$$\delta_{IO\,i} = \delta_{y_i} + (\phi_{IO\,i} - \phi_{ny_i}) L_{p_i} \left(L_i - \frac{L_{p_i}}{2} \right) \tag{8}$$

$$v_{t\,i} = \frac{M_{ny_i}}{L_i} \tag{9}$$

Individual force and displacement response of columns were combined for the construction capacity curve. Base shear capacity of building was calculated by summing the strength capacity

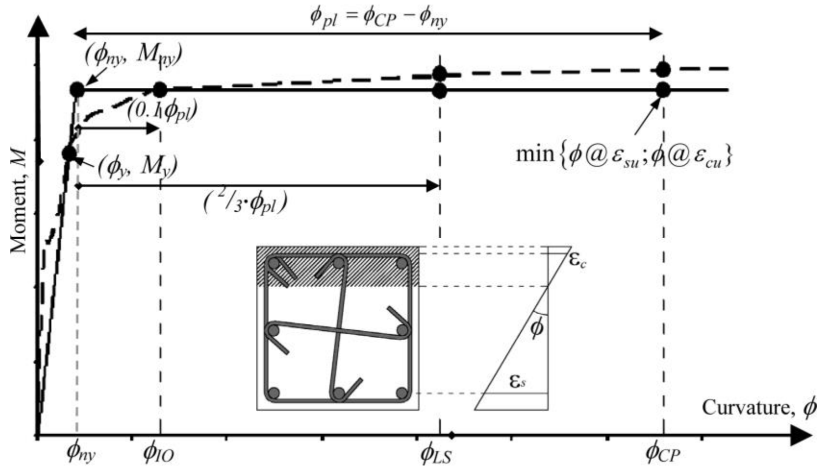


Fig. 3 Demonstration of strain based moment curvature response of precast column section

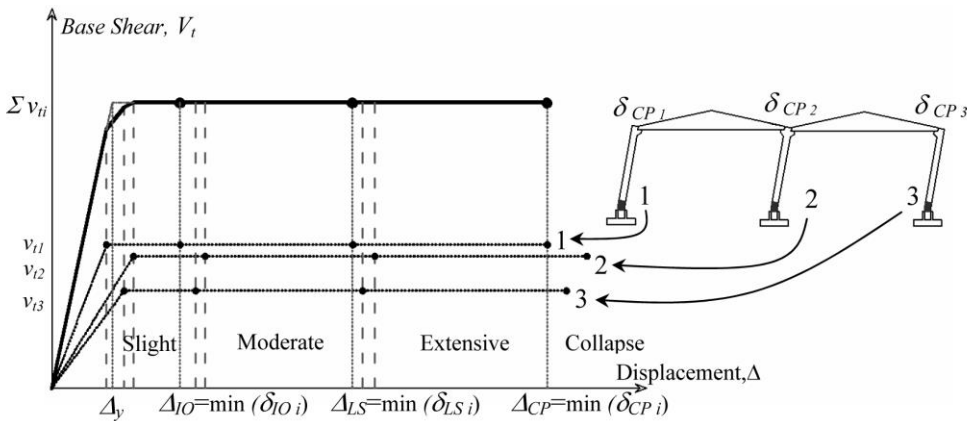


Fig. 4 Construction of building capacity curve and damage levels

of columns as given in Eq. (10). Ultimate displacement capacity of building (Δ_u) was obtained by considering the minimum of individual column displacement responses. Yield displacement of building (Δ_y) was determined by using semi-graphical method, which implies the weighted combination of individual yield displacements. Typical representation of construction building capacity and damage regions are presented in Fig. 4.

$$V_t = \sum_{i=1}^{n} v_{t_i} \tag{10}$$

3.2 Evaluation of capacity related parameters in existing precast buildings

Capacity estimation study was completed for 98 precast buildings by repeating the aforementioned process and capacity related parameters were obtained. Distribution of vibration periods, which can be related with the stiffness of precast buildings, is shown in Fig. 5. Minimum and maximum periods were determined as 1.1 and 2.8 seconds, respectively.

Fig. 6 indicates that drift capacities of majority buildings range between 3.5% and 5.5%. Additionally, the drift ratios obtained from theoretical models are also compatible with

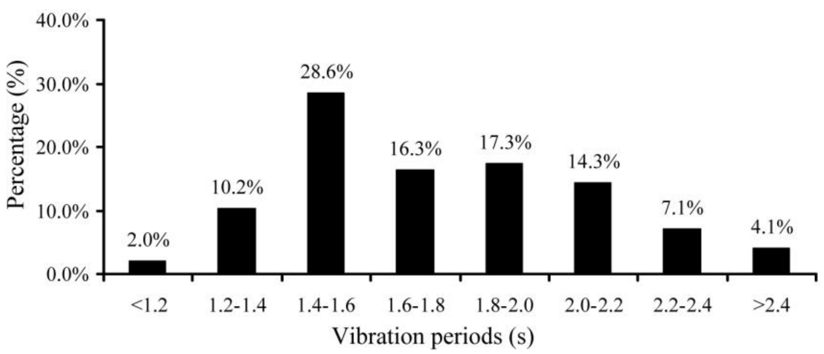


Fig. 5 Distribution of vibration periods (s) (Senel and Palanci 2013)

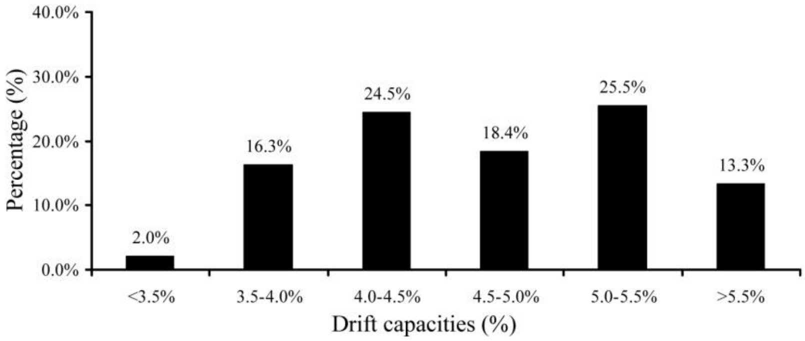


Fig. 6 Distribution of drift capacities (Δ_{CP}/L) (Senel and Palanci 2013)

experimental studies (Fischinger *et al.* 2008) in literature. However, results indicated that the ductility of the inventory buildings is not higher although they have higher drift ratios. Because elastic displacement of columns increases due to cracked section behavior and cantilever action of columns. The evaluation of observed results can be found in the study performed by Senel and Palanci (2013).

3.3 Damage estimation in existing precast buildings

During the seismic demand estimation study, two distinct demand scenarios were used. For the first scenario, geological studies in Denizli Organized Industrial Zone performed by Kılıncarslan and Kılıncarslan (2008) were used. As a result of this study, DOIZ regions correspond to B according to NEHRP classification (FEMA 450 (2003)) and spectrum characteristic periods (T_s) are around 0.46 second. Through the demand estimations, maximum spectral acceleration demand (Sa,max) was taken as 1g, which is also required for the assessment studies according to TEC-2007.

For the second scenario, the property of site class D which takes into account of weak soil properties was used. Actually, there are two reasons for choice of second scenario. Firstly, site investigations and reconnaissance reports prepared after recent earthquakes, indicated that the amount of damage was strongly affected by the weak soil properties (Arslan *et al*. 2006, Atakoy 2000). Secondly, it was desired to question that how precast structures in DOIZ would be affected from weak soil properties. In order to get second spectrum of second scenario, site amplification method of NEHRP provisions (FEMA 450 2003) was adopted. S_s and S_1 parameters of first scenario were kept constant and spectrum characteristic period (T_s) was obtained as 0.69 *s* for site class *D*. Representation of both elastic spectrums is given in Fig. 7.

During the performance assessment, equal displacement approach was used for both scenarios. It should be stated that this method is also suggested by TEC-2007 for long period structures whose vibration periods are greater than *Ts*. Sample application of equal displacement approach in 98 precast buildings are presented in Fig. 8. Seismic demands were then compared with the damage limits (Immediate occupancy, life safety and collapse). Damage cases of inventory buildings were calculated and distribution of damages is given in Fig. 9.

It is observed that 21% of inventory buildings are both extensive and collapse in the first scenario. On the other hand, more than half of buildings (81.6%) are extensive damage and collapse observed in the second scenario. These findings are also compatible with damage

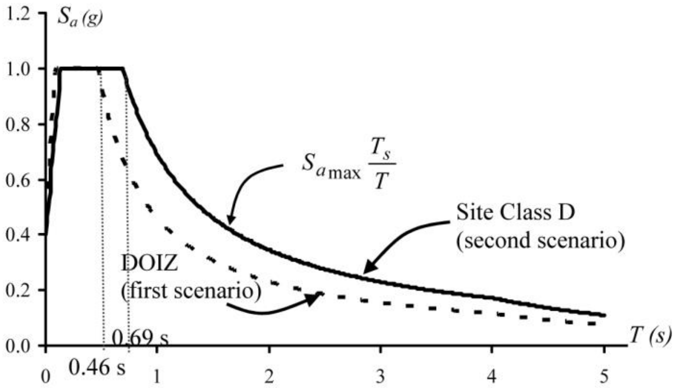


Fig. 7 First and second scenario used in the analysis of precast buildings

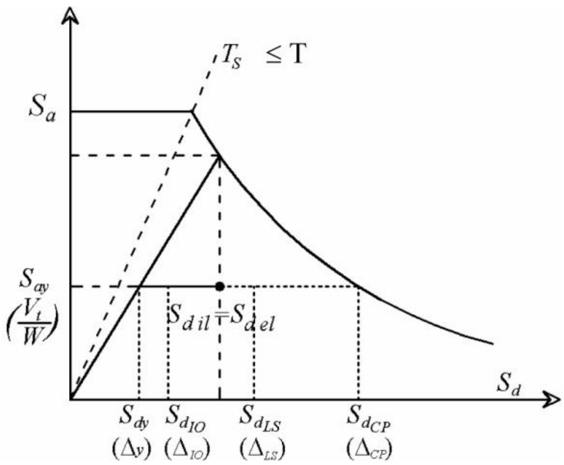


Fig. 8 Performance estimation by using “Equal Displacement Approach”

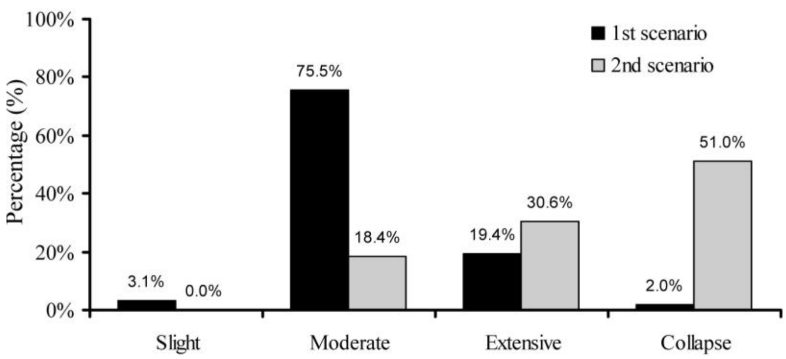


Fig. 9 Damage distribution of inventory buildings

observations performed after 1999 Kocaeli earthquake (Arslan *et al.* 2006, Saatcioglu *et al.* 2001, Atakoy 2000). It was also reported that the ratio of partially and totally collapsed precast buildings in Adapazari industrial region reached up to 80% (Ersoy *et al.* 2000).

4. Rapid seismic performance assessment in existing precast buildings

The reconnaissance studies performed after recent earthquakes have shown the importance of precast buildings since they represent great majority of industrial building stock in Turkey. Extensive damages levels and economical losses reported in past earthquakes increase to develop reliable rapid evaluation method. It is worth to state that performance estimation by rapid evaluation method mainly consists of two steps. In the first step, capacity related parameters such as vibration period, lateral strength and drift capacities of building are determined. In the second step, seismic demand of building is estimated. By this way, capacity and demand parameters can be compared and seismic performance of building can be determined.

4.1 Estimation of lateral strength and drift capacity at yielding

In order to determine lateral strength capacity of columns, Palanci (2010) is generated theoretical column models, which also includes existing precast building stock properties. Lateral strength capacities of columns are determined by using moment curvature analyses of sections. Flexural strength capacity of precast column is determined by using strain based definition of yield point. Then the relationships between the strength of columns and the structural properties such as dimensions, reinforcement ratios, and axial load levels are investigated by using multiple regression analysis. After the evaluation of theoretical and existing cross-sectional results, Eq. (11) is suggested to determine individual lateral strength capacity of columns. In this equation, L_{ort} is defined as mean of column heights in the precast frame and $N_{\%}$ is defined as mean axial load ratio of columns (Eq. (12)). Total base shear capacity of precast building was calculated by combining individual strength capacities of columns depending on the cantilever action of one story precast buildings (Eq. (11)). By this method it is aimed to estimate strength capacity of precast building without making detailed calculations and analyses. Palanci (2010) has shown that concrete compressive strength of inventory buildings are higher than 25 MPa and average axial load ratio of columns can be taken as 5%. In Fig. 10, comparison of analysis and REM lateral strength capacities and distribution of REM/analysis ratios are illustrated. It can be seen from the figure that, suggested equation is consistent with detailed analysis results.

$$v_{t,i} = \frac{73.33 N_{\%}^{0.22}\left(B_i H_i^2 \rho_{l,i}^{0.6}\right)}{L_{ort}} \quad \& \quad V_t = \sum_{i=1}^{Nc} v_{t,i} \quad \text{(N-mm)} \tag{11}$$

$$N_{\%} = \frac{N}{f_c BH} \cong 0.05 \tag{12}$$

During the calculation of displacement capacity of building at yielding, the approach suggested by Priestly *et al.* (2007) was used. According to this approach, yield displacement capacity is proportional to yield strain capacity of longitudinal reinforcement and inversely proportional to column section height. By considering the given approach, moment area theorem is used and yield displacement capacity of individual columns is estimated by Eq. (13). Later, yield displacement capacity of building (Δ_y) is estimated by minimum of individual column displacements.

$$\delta_{yi} = 1.95\left(\frac{\varepsilon_y L^2}{3H}\right) \quad \& \quad \Delta_y = \min(\delta_{yi}) \qquad \text{(mm)} \tag{13}$$

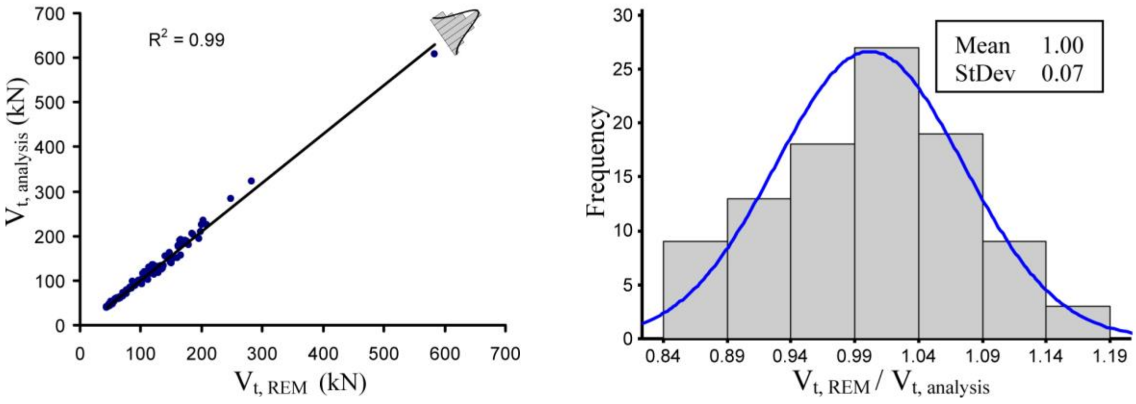


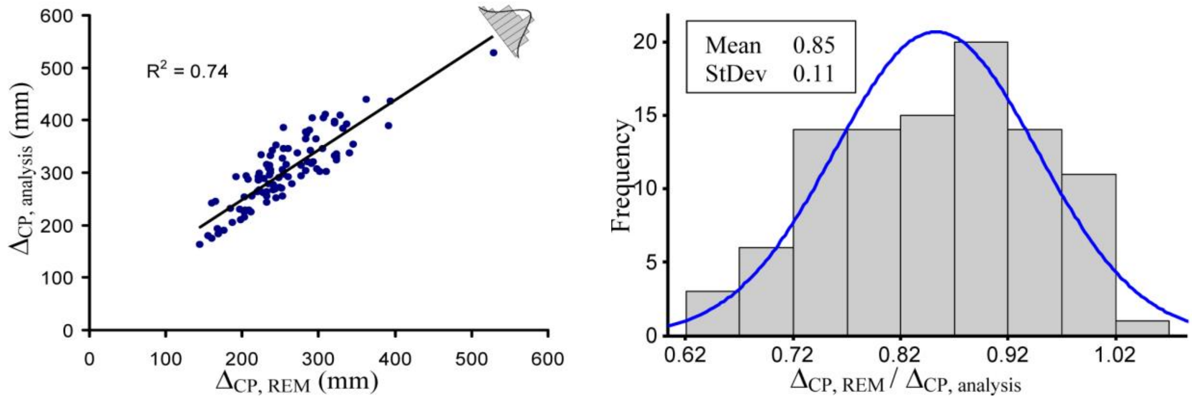
Fig. 10 Comparison of analysis and rapid evaluation method (REM) lateral strength capacities

4.2 Estimation of ultimate displacement capacity of building

In this part, moment curvature analysis of theoretical results is considered while calculating the ultimate drift capacity of buildings. The evaluation of moment-curvature results has shown that ultimate curvature capacity of column section is mainly controlled by compressive strain capacity of concrete. Concrete strain capacity, on the other hand, governed by confinement and axial load level of precast columns (Palanci 2010). Investigations have shown that axial load levels in one story precast buildings are not critical and average axial load ratio can be taken as 5% (Eq. (12)). This situation indicates that drift capacity of precast columns can be classified according to their confinement level. In fact, this assumption gives compatible approach given in Eq. (1). Therefore, confinement ratio of columns is considered as a distinctive parameter in proposed rapid assessment method. For this purpose, columns are divided into three groups (“good”, “average” and “poor”) according to their confinement qualities. Confinement ratios corresponding to each quality groups are obtained by dividing existing confinement (ρ_s) to code conformed confinement (ρ_{sm}). Moment curvature analyses of existing building columns were performed and the distribution of ultimate curvature levels corresponding to strain capacities was assessed. In Table 1, confinement quality, confinement ratio and corresponding core strain capacities of each group are given. For the estimation of curvature capacity of column Eq. (14) was proposed. It can be clearly seen that in fact, the denominator of this equation represents the neutral axis depth of column section beyond concrete cover. While assessing the results of moment curvature analyses, variation of neutral axis depths of precast column sections was also considered. Section parameters that control the neutral axis depth were investigated by using statistical methods and consequently Eq. (14) was obtained.

Table 1 Core strain limits according to lateral confinement ratios

Confinement quality	Confinement ratio (%)	Strain capacity
Good	$\rho_s/\rho_{sm} \geq 75\%$	ε_{cc} =1.30%
Average	$\%35 < \rho_s/\rho_{sm} < 75\%$	ε_{cc} =1.00%
Poor	$\rho_s/\rho_{sm} \leq 35\%$	ε_{cc} =0.70%



(a) (b)

Fig. 11 Comparison of analysis and REM displacement capacities

$$\phi_{CP} = \frac{\varepsilon_{cc}}{5.00\rho_l^{0.35}H^{0.75} - d'} \quad \text{(mm)} \tag{14}$$

In Eq. (14) ρ_l, H, and d' are defined as longitudinal reinforcement ratio, height and concrete cover (preferably assumed 20mm) of individual column. Column drift capacity (Eq. (15)) is simplified by implementing Eqs. (13)-(14) into Eq. (6). Damage limits "Immediate Occupancy" and "Life safety" were obtained from 10% and 67% of total plastic drift capacity of the building as mentioned earlier. Ultimate displacement capacity of building is estimated by minimum of individual column displacements. Analysis and REM drift capacity results are compared and distribution of REM/analysis ratios is shown in Fig. 11. It can be seen from the figure that mean value of ratio is 0.85 and this situation implies that estimated results are in safer region. It can be stated that suggested equation estimates reliable results due to lower standard deviation (11%).

$$\frac{\delta_{CP,i}}{L_{ort}} = \frac{H_i}{2}\phi_{CP,i} + 0.004\left(\frac{L_{ort}}{3H_i}\right) \quad \& \quad \Delta_{CP} = \min\left(\delta_{CP,i}\right) \quad \text{(mm)} \tag{15}$$

4.3 Estimation of seismic demand

The most affective variable in seismic demand estimation is the vibration period and it is calculated by Eq. (16) in the developed method. While calculating the vibration periods, stiffness of buildings are obtained from the elastic slope of capacity curves. The slope of capacity curve is obtained by proportion of lateral strength and displacement capacity at yield level. Building mass can easily be determined by performing vertical load (dead and live loads) analysis.

$$T = 2\pi\sqrt{\frac{m\Delta_y}{V_t}} \tag{16}$$

Seismic demand of building is calculated with Eq. (17) using equal displacement approach by considering cantilever behavior of single degree of freedom (SDOF) system. It can be seen from the equation that peak ground acceleration (PGA) and the soil type of the building is considered.

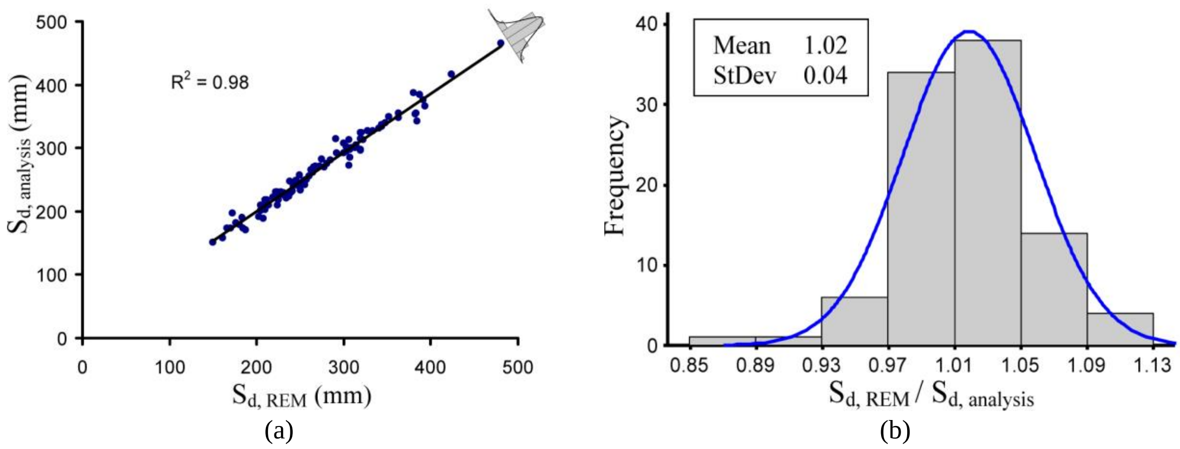


Fig. 12 Comparison of analysis and REM seismic displacement demands

$$S_{dil} = S_{ae} g \frac{T^2}{4\pi^2} \cong 0.063(PGA)T_s T \text{ (mm)} \tag{17}$$

In the equation, PGA can be taken as 0.4g for high seismic regions which is also suggested by TEC-2007. Seismic demand of inventory buildings is estimated by repeating the aforementioned process and compared with detailed analysis demands. Distribution of REM/analysis ratios and comparison of results is shown in Fig. 12. It can be clearly seen that the variation of ratios are considerably low (Fig. 12(b)). This situation indicates the reliability of estimated lateral strength and the yield displacement capacity of buildings. It should be also noted that vibration periods of buildings are compatible with each other (Fig. 12(a)).

5. Summary of rapid evaluation method

Before the results, it is deemed useful to summarize the stages of the Rapid Evaluation Method (REM) procedure to be applied.

1. Determine the column dimensions for the critical frame and obtain lateral strength capacity of individual columns by Eq. (11). Then, calculate the base shear capacity of frame by summing of individual lateral strengths capacities.
2. Calculate the individual yield displacement capacity of columns by Eq. (13) and determine the yield capacity of building by taking minimum of column displacements.
3. To calculate ultimate curvature capacity of individual columns, use Table 1 to obtain core strain limits and calculate the ultimate capacity curvature of each column by Eq. (14). Convert the predetermined capacity curvatures to drift capacities by Eq. (15) and estimate the ultimate displacement capacity of building by minimum of individual column drift capacities. Damage limits of building; Immediate Occupancy (IO) and Life Safety (LS) are obtained by 10% and 67% of total plastic drift capacity of building.
4. Perform the vertical load analysis to obtain mass of building by considering dead and live loads.

5. Use the predetermined base shear capacity (step 1), yield capacity (step 2), and mass (step 4) to calculate the natural vibration period of building by Eq. (16).

6. Use the natural vibration period determined in the last step and calculate seismic demand by Eq. (17).

7. Determine the seismic performance of the building in terms of displacement by comparing seismic demand (step 6) and damage limit capacities of building (step 3).

6. Verification of proposed model

In this part of the study, procedure expressed in Chapter 4 is repeated for the inventory buildings and damage distributions of each scenario are presented in Figs. 13-14. It can be seen from the Fig. 13 that more than half of buildings (75.5%) have moderate damage and there are so less slight damaged (3.1%) buildings. In addition, slight and moderate damages are calculated (0.0%) and (18.4%) in the second scenario, respectively. It is worth to point out that sum of extensive damage and collapse percentages is estimated around 31.7% by proposed method while it is determined around 21.4% by nonlinear analysis methods for the first scenario. In the second scenario, similar damage distributions are also observed. It can be concluded from the observed damage distributions that analysis and REM damages for collapse and extensive damages are closer in both scenarios.

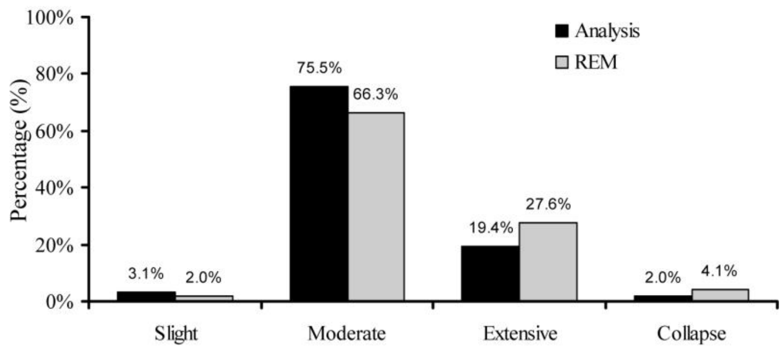


Fig. 13 Comparison of analysis and REM damages for first scenario

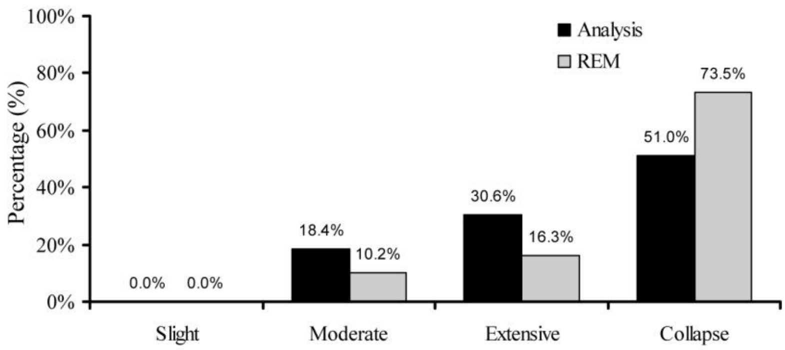


Fig. 14 Comparison of analysis and REM damages for second scenario

It should be stated that although displacement capacity (Fig. 11) and seismic demand (Fig. 12) are estimated reliably, analysis and REM damage distributions may quite differ from each other. However, the difference between rapid evaluation method and analysis results stem from narrow plastic displacement capacity of precast buildings relatively. This situation increases the sensitivity of assessment study to even smaller changes in displacement demand (Palanci 2010, Senel and Palanci 2013).

After determination of damages, plastic deformation ratio is also calculated for each building by Eq. (18). It is clear from the equation that plastic deformation ratio is defined herein as a proportion of seismic demand and capacity in terms of plastic displacement on capacity curve. So that, plastic deformation ratio is calculated for each building in the inventory by considering analysis (nonlinear analysis method) results and proposed rapid assessment method cases. Later, average of plastic deformation ratios for each damage (slight, moderate, extensive, and collapse) is obtained for both scenarios. The observed plastic deformation ratios for both scenarios are presented in Figs. 15-16. Comparison of results in terms of plastic deformation ratios clearly indicates that observed ratios are also compatible in both scenarios.

$$\%\Delta_{pl} = \frac{S_{dil} - S_{dy}}{S_{dCP} - S_{dy}} \ \text{(mm)} \tag{18}$$

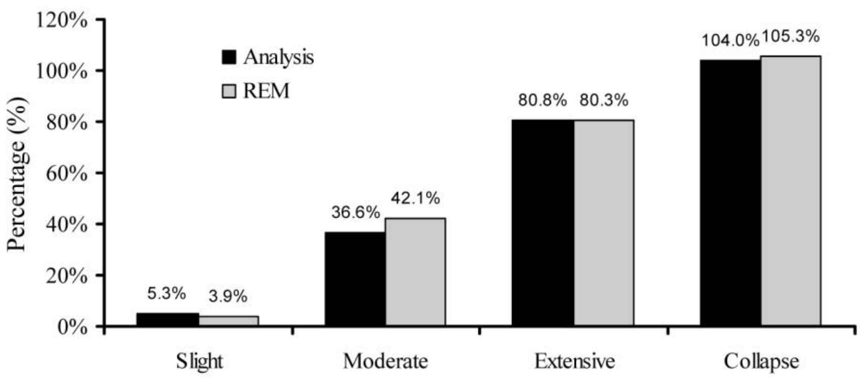


Fig. 15 Analysis and REM average plastic deformation ratios for first scenario

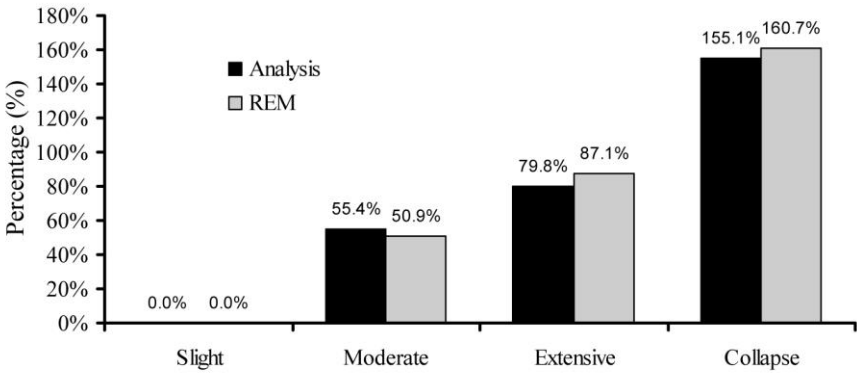


Fig. 16 Analysis and REM average plastic deformation ratios for second scenario

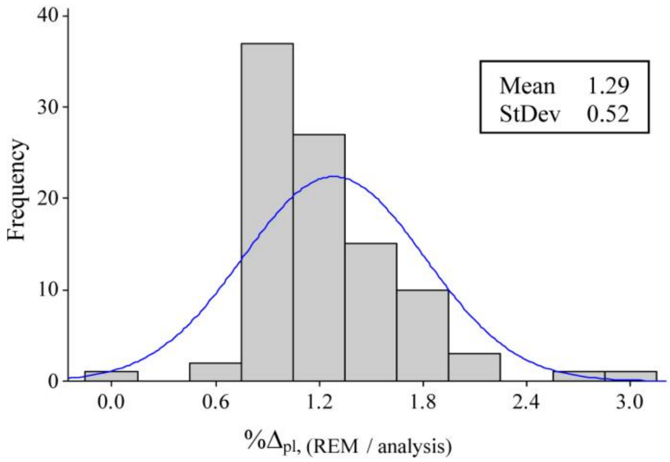


Fig. 17 Distribution of plasticity ratios for the first scenario

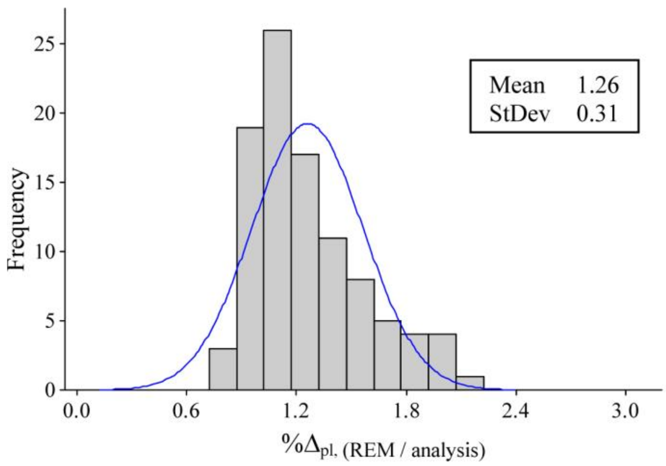


Fig. 18 Distribution of plasticity ratios for the second scenario

Consequently, REM and detailed analysis plastic deformation ratios are divided and observed ratios are illustrated in Figs. 17-18. It can be clearly seen from the figures that in average, REM overestimates the plastic deformation ratios around 1.25 times. Nevertheless, it can be said that observed ratios are reasonable due to the nature of rapid evaluation methods.

7. Application of rapid evaluation method on existing building

In this stage of study, proposed method is applied to existing sample building in DOIZ. Properties of purlin and roof girders of building are given in Table 2 and typical representation of two dimensional frame of building is illustrated in Fig. 19. Structural properties of column members (dimensions, longitudinal & transverse reinforcement, and axial load ratio) are given in Table 3.

Table 2 Roof girder and purlin properties of existing sample building

Girder No	Girder Length	Purlin length	Number of Purlin	Purlin section area	Heading area of girder	Mid area of girder
#	m	m	-	m^2	m^2	m^2
1	15	7.5	9	0.019	0.137	0.213
2	20	7.5	11	0.019	0.137	0.213
3	20	7.5	11	0.019	0.137	0.213
4	20	7.5	11	0.019	0.137	0.213

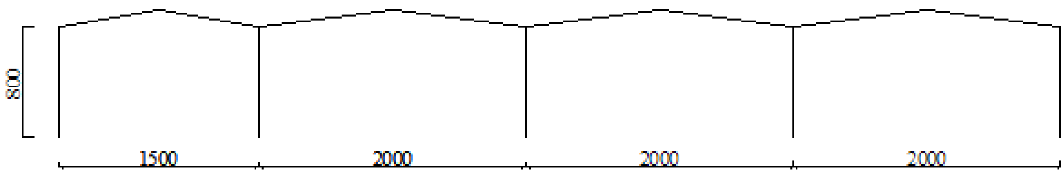


Fig. 19 Typical representation of sample building frame (cm)

Table 3 Properties of column members

Column No	Column dimensions		Transverse Reinforcement Strength	Longitudinal Reinforcement Ratio	Volumetric transverse Reinforcement Ratio	Axial Load
	B	H	f_{ywd}	ρ_l	ρ_s	
#	mm	mm	MPa	%	%	kN
1	400	400	420	1.00%	0.42%	143.7
2	450	450	420	1.01%	0.37%	296.1
3	450	450	420	1.01%	0.37%	328.4
4	450	450	420	1.01%	0.37%	328.4
5	400	400	420	1.00%	0.42%	176

Table 4 Capacity estimation of each column in existing sample building

Column No	L_{ort}	H	v_t Eq. (11)	ρ_s / ρ_{sm}	δ_y Eq. (13)	ε_{cc} (Table 1) (avarage quality)	ϕ_{CP} Eq. 14	δ_{CP} Eq. (15)
-	mm	mm	kN	%	mm	%	rad/mm	mm
1		400	19.12	63.97%	218.40	1.0%	$1.446\ 10^{-4}$	444.68
2		450	27.34	56.17%	194.13	1.0%	$1.288\ 10^{-4}$	421.43
3	8000	450	27.34	56.17%	194.13	1.0%	$1.288\ 10^{-4}$	421.43
4		450	27.34	56.17%	194.13	1.0%	$1.288\ 10^{-4}$	421.43
5		400	19.12	63.97%	218.40	1.0%	$1.446\ 10^{-4}$	444.68

Rapid evaluation procedure is pursued as mentioned earlier and column capacity parameters are obtained. Later on, lateral strength and displacement capacity of sample building are estimated. Both REM and analysis capacity parameters are given in Table 5. Comparison of REM and analysis capacity curves is also shown in Fig. 20. It can be clearly seen from the figure that capacity curves are almost compatible with each other.

Table 5 Capacity parameters of existing sample building

Assessment Method	V_t kN	Δ_y / L_{ort} %	Δ_{IO} / L_{ort} %	Δ_{LS} / L_{ort} %	Δ_{CP} / L_{ort} %
REM	120.27	2.43%	2.71%	4.32%	5.27%
Analysis	115.67	2.39%	2.74%	4.40%	5.45%

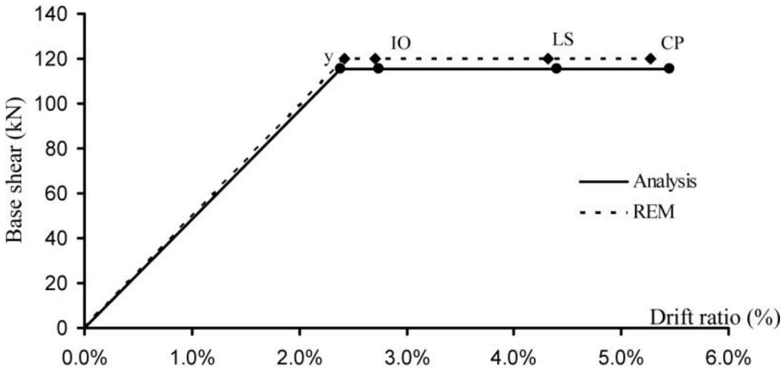


Fig. 20 Comparison of analysis and rapid evaluation method capacity curves

Table 6 Seismic demand and performance results

Scenario	Assessment method	T_1 s.	S_{dil} / L_{ort} %	Δ_{pl} %	Damage
1st scenario	REM	2.40	3.42%	35.12%	Moderate
	Analysis	2.43	3.47%	35.14%	Moderate
2nd scenario	REM	2.40	5.14%	95.38%	Extensive
	Analysis	2.43	5.20%	91.79%	Extensive

The last, seismic demand is estimated for both scenarios. Since the vibration period of building was obtained so close by REM and analysis, seismic displacement demands were so close to each other. In both assessment methods, performance of sample building is determined as “Moderate” and “Extensive” for moderate and weak soil sites, respectively.

8. Conclusions

Extensive inventory study based on 98 precast buildings constructed in one of the important seismic zones of western Turkey, Denizli Organized Industrial zone (DOIZ), was performed and building inventories were prepared.

Structural properties of inventory buildings located in DOIZ and damaged precast buildings in recent earthquakes were compared and similarities of properties were especially emphasized. Compatibility of analysis and observed damages in recent earthquakes indicates that nonlinear analysis methods followed for the seismic performance assessment study can be simplified to a rapid assessment method. For this reason, capacity curves of precast buildings were investigated and lateral strength and inelastic displacement capacities of buildings were formulated by using structural properties such as member and building dimensions, longitudinal and transverse reinforcement ratios, etc. Then, vibration period of buildings, which directly controls the seismic demand levels, were also formulated. Accuracy of proposed method was tested by using two different demand scenarios which reflect weak and moderate soil properties. The results of detailed and rapid assessment methods yield similar damage ratios.

Compatibility of heavier damage estimations (extensive and collapse), which control the retrofit or demolish decisions, indicates that proposed method can be used efficiently for one story hinged precast industrial buildings. By this method, rapid performance assessment of existing one story precast buildings, which represent the great majority of lightweight industrial building stock of Turkey, can be determined. It is also expected that work and labor required for the large scale assessment studies can be reduced dramatically by the proposed method.

Acknowledgements

The research reported in this paper was supported by the Scientific and Technological Research Council of Turkey (TUBITAK) under Project No: 110M255 and by the Turkish Precast Concrete Association. The authors wish to express their gratitude also to directorate of Denizli Organized Industrial Zone for providing design projects of precast buildings.

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