PERFORMANCE EVALUATION OF A VERTICALLY IRREGULAR RC BUILDING

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ABSTRACT :

The earthquake performance of a vertically irregular existing building, which has been retrofitted afterwards, is investigated herein this study. This type of irregularity, which is due to non-coinciding longitudinal axes of the columns on the periphery and only corner columns for two neighboring stories, generally exists between the ground and the first floors. A six-story reinforced concrete (RC) building with structural materials of C20 class concrete and S420 class reinforcing steel, which was designed considering the Turkish Earthquake Resistant Design Code 1975 (TERDC-1975) regulations, has the mentioned type of irregularity with console length and console depth of 1.25m each. The building has four and three spans at *x-x* and *y-y* directions, respectively. Story heights are same and equal to 3.0m. The 3D model of the structural system is used for numerical solutions by the use of SAP2000 and ZEUS-NL softwares. Pushover analyses are carried out for each of the *x-x* and *y-y* directions and capacity spectrum method, which is based on static pushover analysis, is used to obtain the performance levels of the existing structural systems. The strengthened system is analyzed similarly and performance evaluation is realized, based on the demand and capacity curves. Furthermore, comparisons are carried out and discussed in details.

KEYWORDS : Performance, reinforced concrete building, vertical irregularity.

1. INTRODUCTION

The experiences from past earthquakes have shown that the regularity either in plan or in elevation is of crucial importance on earthquake performance of structures. In the building stock of Turkey, the consoled buildings have important ratio. Besides plan irregularities, vertically irregular buildings have also important portion of the existing buildings. Generally, there are two types of consoled buildings that are commonly used: The first type is constructed by use of cantilever beams. In this case, the periphery columns of the building are not connected to each other with beams; hence the periphery beams are offset to the end of the cantilever beams. Consequently, the periphery frame system of the building cannot be established. In the case of second type of consoled building application, the structural system becomes vertically irregular, due to the axes of the columns around periphery of the building that do not coincide with each other on the ground and the first floors. The columns of these two floors are connected through corbels between two offset columns, although it is not permitted anymore according to the new Turkish Earthquake Resistant Design Code, TERDC, (2007). By using force based approach, Güler (1996) has studied the behavior of this kind of a vertically irregular building. Most of the codes require sophisticated computational procedures, if the structural system is irregular one. Moehle (1984) has also studied the seismic response of vertically irregular buildings. The Kocaeli Earthquake of August 17, 1999 affected a wide range of area including the biggest city of Turkey, Istanbul and the reinforced concrete buildings, including those consoled ones are tested once and important lessons are learned on the behavior of consoled structures (Güler and Altan, 2004). Currently, performance based approach is getting popular either for the structures to be designed or existing buildings, even for the buildings with irregularities. Chintanapakdee and Chopra (2004) has studied a number of vertically irregular frame buildings up to 12 stories, which satisfy the strong column-weak beam issue, by using modal pushover analysis method. In their study, irregularity due to the existence of soft story is studied in details. The current TERDC-2007, which is the latest version, covers a new chapter on performance evaluation of existing buildings by applying nonlinear static procedure. Brief information about the new chapter and methodology is given in the following part of this paper.

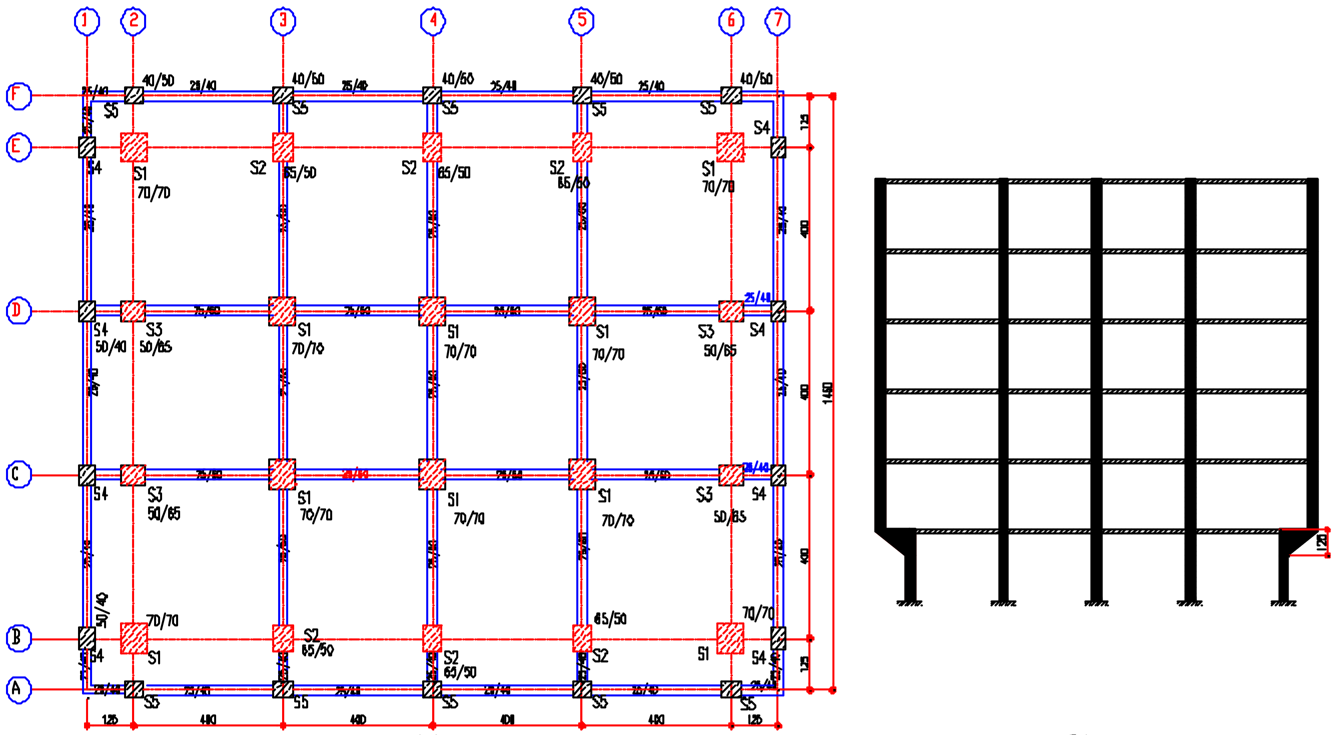
In this study, a six-story building with second type of vertical irregularity, which was designed according to TERDC-1975 regulations, is investigated in details. The structural system of the building is 3D-modeled by using SAP2000 first, and afterwards with ZEUS-NL software. 3D pushover analysis is carried out for each of the *x-x* and *y-y* directions and by applying capacity spectrum method, which is based on nonlinear static pushover analysis, performance levels of the existing structural system is obtained. As a second case, the strengthened system is also analyzed by means of similar procedures and performance evaluation is realized, based on the demand and capacity curves.

2. TURKISH EARTHQUAKE RESISTANT DESIGN CODE 2007, (TERDC-2007)

TERDC-2007 describes the plan and elevation irregularities in details and some limitations are also given in design and analysis methods for buildings having different kind of irregularities. As it is mentioned above, TERDC-2007 contains a new chapter on performance evaluation of structures and two analysis procedures are given: linear and nonlinear analysis methods. In the first (elastic) method, the analysis is carried out by considering occupancy and seismic load reduction factors equal to 1.0. Equivalent seismic load method can be used to determine base shear force if the torsional effects are insignificant, the height of the structure less than 25m or less than 8 stories high and mass participating ratio for first mode is higher than 70%; otherwise mode superposition method should be used taking into account the effects of higher modes. The second (nonlinear) method aims to obtain plastic deformation or force demands regarding the brittle or ductile behavior of structural elements. Three nonlinear analysis methods are given in TERDC 2007: incremental equivalent seismic load method, incremental mode superposition method and time domain (dynamic) method. The first two methods are related to incremental pushover analysis. Nonlinear material behavior may be idealized by utilizing the plastic hinge hypothesis. The nonlinear force-deformation characteristics of structural elements are incorporated in the form of moment-rotation (or curvature) relationships assigned to plastic hinges. It is assumed that plastic deformations are distributed uniformly along the plastic hinge length that is half of the element depth. After carrying out the pushover analysis by increasing the lateral load such that a target displacement at roof level is encountered, a coordinate transformation is made from base shear force-top story displacement curve to modal acceleration–modal displacement curve, and top story displacement demand can be calculated for the desired earthquake direction. The performance levels of the elements are then evaluated by comparing the sectional strains with the limiting strains defined for different performance levels. Building performance level is concluded by considering beam, column and shear-wall performance levels.

3. THE ANALYSED VERTICALLY IRREGULAR RC BUILDING

The RC building analyzed herein this paper, is a six-story reinforced concrete structure consisting of an orthogonal frame system and it is designed considering the TERDC-1975 regulations. The layout of the ground floor and the elevation of the building are shown in Figure 1. However, this kind of irregularity it is not permitted anymore since the 1998 version of TERDC and absolutely neither in the TERDC-2007. As it is shown in the plan and elevation drawings of the building, the axes of the columns around the periphery of the building do not coincide with each other on the ground and first floors. The columns on the corner and on the perimeter in plan of these two floors are connected with corbels, which make the structural system a vertically irregular one; hence irregular frame system is arranged. However, the internal columns of the building are continuous from the ground to the top floor. The story heights are 300 cm and the slab thickness is 12 cm. The corbels have a height of 40 cm and 125 cm at the tip and at the joint at the column surface, respectively. The column sizes and corresponding longitudinal reinforcements are given in Table 3.1. The periphery beams supported on consoles have 25cm/40cm dimensions, while the rest of the frame beams are 25 cm/60 cm. The longitudinal beam reinforcement is tabulated in Table 3.2. The static and dynamic numerical analysis is carried out by using the SAP2000 package, assuming linear elastic behavior. For the structural system, the material classes of concrete and steel and modulus of elasticity is considered as C20 (f_{ck}=20MPa) and S420 (f_{yk}=420MPa) and 28×10^{6} kN/m^{2}, respectively. The structural system of the building is considered as space frames having shell elements representing floors. The columns are assumed to be fixed at base at the foundation level. The first 12 mode free vibration periods of the structure are given in Table 3.3. As it is seen, the mass participation ratio is equal to 0.74 (74%) and greater than 0.70 (70% code limit) for the first mode and the height of the building is less than 25 m, which means that the incremental equivalent seismic load procedure can be used as defined in TERDC-2007.



(a) (b)

Figure 1 Slab plan (a); and elevation of the vertically irregular building (b).

Table 3.1 Columns sizes and total longitudinal reinforcement

Columns	**b × h** (cm×cm)	**Reinforcement**
A2, F2, A3, F3, A4, F4, A5, F5, A6, F6	40 × 50	8Ø20 mm
B1, B7, C1, C7, D1, D7, E1, E7	40 × 50	8Ø20 mm
B3, B4, B5, C2, C6, D2, D6, E3, E4, E5	50 × 65	12Ø20 mm
B2, B6, E2, E6, C3, C4, C5, D3, D4, D5	70 × 70	16Ø20 mm

Table 3.2 Beam dimensions and longitudinal reinforcements

Regular frame beams: 25 cm/60 cm		
Inner spans	2Ø12 mm (top)	3Ø14 mm (bottom)
Supports	2Ø16 mm(top,left)	2Ø14 mm (top, right)
Edge span	2Ø12 mm (top)	3Ø12 mm (bottom)
Supports	3Ø16mm (top,left)	2Ø16 mm (top, right)
Periphery beams: 25 cm/40 cm		
Inner spans	2Ø12 mm (top)	3Ø14 mm (bottom)
Supports	2Ø16 mm (top, left)	2Ø14 mm (top, right)
Edge span	2Ø12 mm (top)	3Ø14 mm (bottom)
Supports	3Ø16 mm (top, left)	2Ø16 mm (top, right)

Table 3.3 Free vibration periods and modal participation ratios for first 12 modes of the existing building

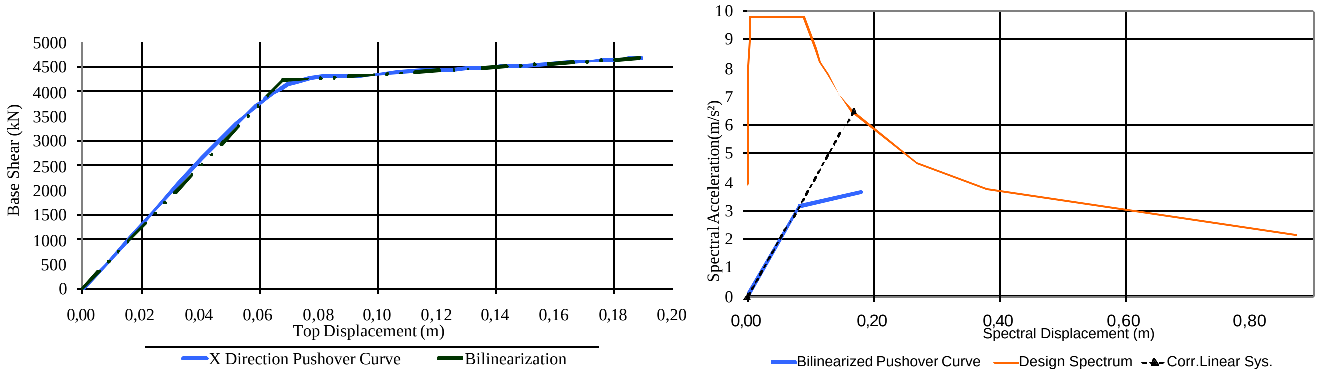
Mod number	Period (*sec*)	Circular Frequency (*rad/sec*)	Modal Participation Ratios (*x* direction)	Modal Participation Ratios (*y* direction)
1	0.803921	7.816	0.000	0.741
2	0.764914	8.214	0.741	0.741
3	0.216512	29.020	0.741	0.862
4	0.213068	29.489	0.861	0.862
5	0.114017	55.108	0.861	0.930
6	0.112592	55.805	0.929	0.930
7	0.086433	72.694	0.929	0.930
8	0.070169	89.544	0.929	0.975
9	0.06994	89.836	0.974	0.975
10	0.049656	126.530	0.995	0.975
11	0.049529	126.860	0.995	0.995
12	0.039419	159.390	1.000	0.995

3.1. Performance of the Building

Seismic performance assessment of the investigated vertically irregular building has been realized by 3D modeling of the frame structure by introducing the elements into SAP2000 and ZEUS-NL computer programs, separately. For each modeling frame elements are used for columns and beams and shell elements are used for slabs. Special attention is paid for defining the corbels located at periphery of the building. During the implementation of the method by using SAP2000 package, moment-curvature relationships of column and beam sections have been introduced by using Mander *et al.* (1988) model and a bilinear elasto-plastic stress-strain model for concrete and steel is assumed. Depending on the sectional characteristics and site observations, it is preferred to neglect the confinement effect in concrete, since the stirrup tightening was insufficient. Strain-hardening of steel is also neglected in the stress-strain relationship.

As a first step as it is defined in the TERDC-2007, the pushover curve (base shear at ordinate and roof displacement at apses) is obtained for a target top story displacement of 19cm is encountered, both for *x-x* and *y-y* directions. Only *x-x* direction results are exhibited in Figures 2 and 3. In Figure 2(a), bilinearly transformed version of the pushover curve is also shown. For obtaining the modal capacity diagram, a coordinate transformation to the pushover curve is performed and considering a seismic event occurrence within the next 50 years with a probability of exceedence of 10%, the modal displacement demand is calculated by comparing the demand curve which is obtained from the relevant elastic design spectrum. However, as it can be seen from Figure 2(b), a performance point could not be calculated for the existing building. Since the software follows FEMA-356 regulations and limits, the output for the elements are rotations at plastic hinges and these values are automatically compared with the selected performance criteria. However in the TERDC-2007, strains are controlled with certain limits at the plastic hinges of structural elements. Therefore, the required strains at sections are computed from the plastic hinge rotations of the program outputs and compared with the specific safety limits. Assuming a "life-safety" performance level for this residential building, the computations indicated that the structure fails to satisfy the required limits; consequently it is decided to strengthen the structural system.

Similar procedure and calculations are carried out by modeling the building with ZEUS-NL software. Despite the two computer programs, namely SAP2000 and ZEUS-NL, have different approaches and assumptions for modeling and numerical treatment of the structural systems, the results obtained are shown to be in good agreement as it is illustrated in Figure 3.



(a) (b)

Figure 2 Pushover curve (a) and capacity *vs*. demand estimation for frame system in *x-x* direction (b).

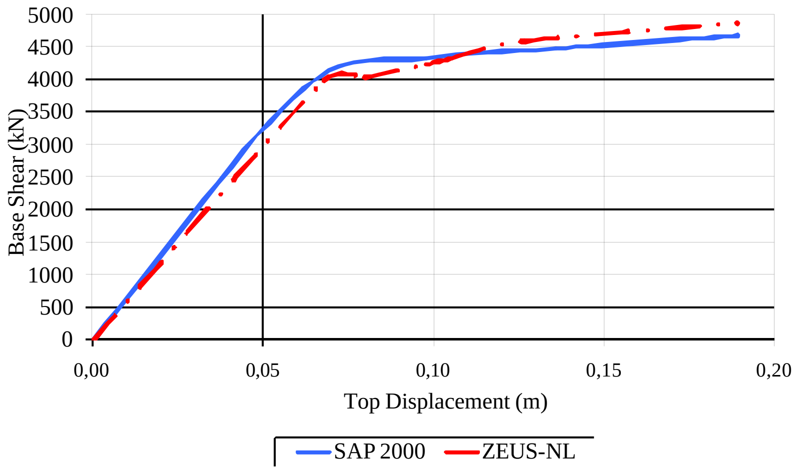


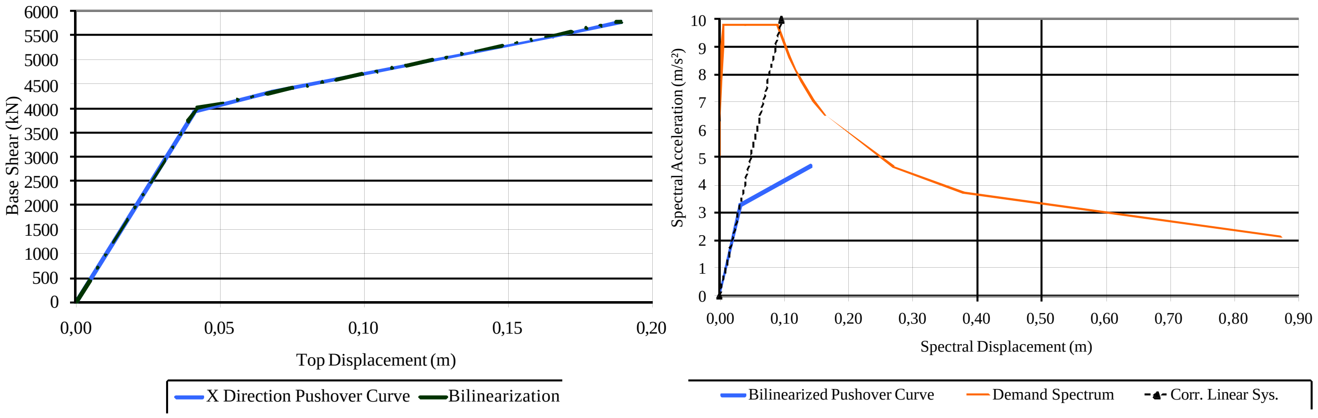
Figure 3 Pushover curves for *x-x* direction obtained by SAP2000 and ZEUS-NL programs.

3.2. Performance of the Strengthened Building

For strengthening the RC frame building, shear walls are introduced into the structural system. Two shear walls, which are in the *x-x* direction of the building are placed along *B* and *E* axes, through the 4*th* and the 5*th* axes, while two other are placed in *y-y* direction along the 2*nd* and 6*th* axes connecting through *C* and *D* axes. For increasing the efficiency in seismic resistance, four of the shear walls are preferred to be constructed on the periphery of the building.

When the 3D modeled strengthened system is analyzed by means of free vibration by using SAP2000, first three modes are found out to be shortening to 0.532s, 0.518s and 0.116s, respectively. The mass participation ratio for the first mode is computed as 77.4% with an increment of 3.4%, as expected depending on the increased stiffness for the entire structure. Similar analysis procedure for the as-built structure is applied for the strengthened state of the building. Figure 4 shows the performance point, which is calculated by the use of the new capacity and demand curves for a life-safety performance level. It is understood that for the case of strengthened state, the building satisfies the required limits for life-safety. Once again, strengthened system is modeled by ZEUS-NL and similar computations are carried out. Figure 5 shows a comparison of the results obtained by the two computer programs by means of the calculated base shear-roof displacement variation.

In last step of the incremental pushover analysis, plastic hinge patterns for strengthened system are investigated and locations are exhibited in Figure 6. Pink and blue colors for hinges correspond to immediate occupancy and life safety performance levels, respectively. When the total number of plastic hinges is considered, it yields that structural system of the building satisfies life-safety performance level.



(a) (b)

Figure 4 Pushover curve (a) and the capacity-demand estimation for strengthened system (b) for *x-x* direction

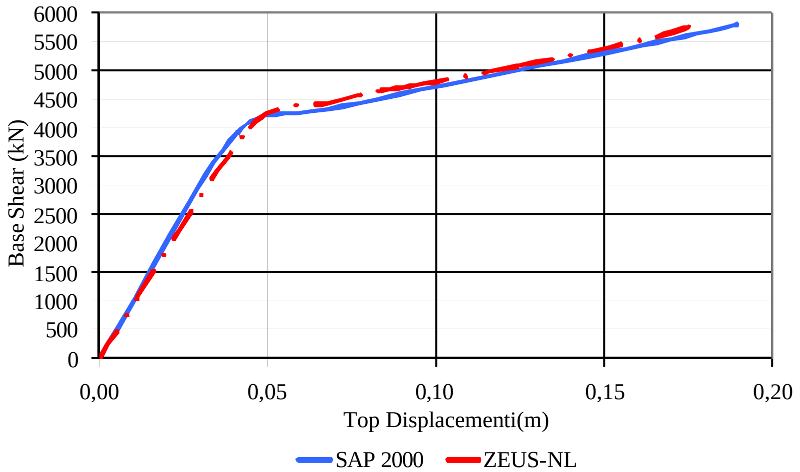


Figure 5 Pushover curves for *x-x* direction of the strengthened building obtained by SAP2000 and ZEUS-NL.

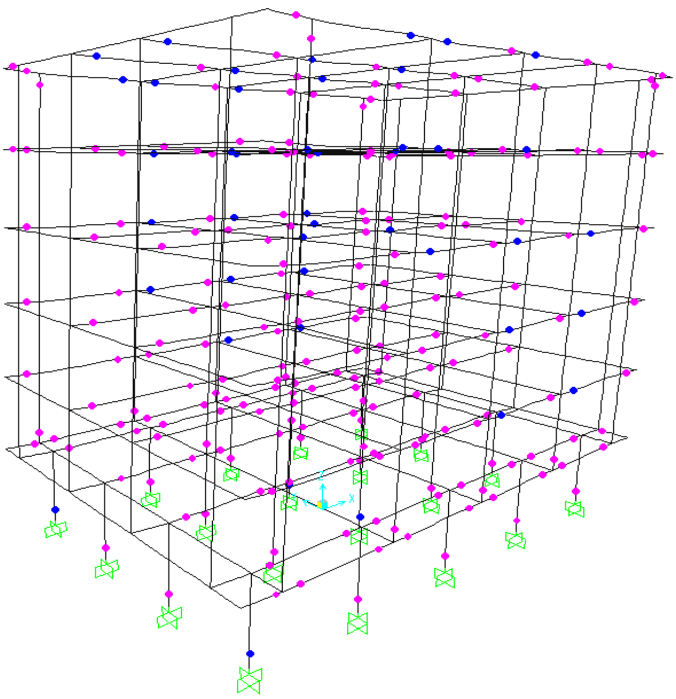


Figure 6 Plastic hinge patterns of strengthened building at the end of the pushover by SAP2000

4. CONCLUSIONS

In this paper a six-story building with second type of vertical irregularity, which was designed according to TERDC-1975 regulations, is investigated in details according to the current TERDC-2007 limits considering a life-safety performance level. The structural system of the building is 3D-modeled by using SAP2000 first, and afterwards with ZEUS-NL software. 3D pushover analysis is carried out for each of the *x-x* and *y-y* directions and by applying capacity spectrum method, which is based on nonlinear static pushover analysis, performance levels of the existing structural system is obtained. As a second case, the strengthened system is also analyzed by means of similar procedures and performance evaluation is realized, based on the demand and capacity curves. The performance evaluation of the existing vertically irregular RC building has yielded the following results so far:

- The analysis results that are derived from the computations indicate that the as-built structure fails to provide life-safety performance level. When the structure is strengthened by adding a number of four shear walls through the height of the building, then this performance level is satisfied.
- Although current engineering practice prefers to apply nonlinear static procedures for the seismic performance evaluation of existing buildings, one should remember that these analysis methods are limited with the amount of the irregularities in plan and elevation, number of stories and first mode mass participation factor.
- Nonlinear modeling of geometric discontinuities caused by corbels may result in modeling errors. Therefore, in spite of defining plastic hinges under bending moments, one should especially introduce shear hinges at corbels, since their structural behavior is more sensitive to shear forces.

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