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A COMPARATIVE EVALUATION OF PERFORMANCE BASED ANALYSIS PROCEDURES ACCORDING TO 2007 TURKISH EARTHQUAKE CODE AND FEMA-440

R.Tuğrul Erdem, Muhiddin Bağcı and Ali Demir
Department of Civil Engineering, Celal Bayar University, 45140, Manisa, Turkey
muhiddin.bagci@bayar.edu.tr

Abstract- Chapter 7 of the 2007 Turkish Earthquake Code (TEC) entitled "Assessment and Strengthening of Existing Buildings" sets standards for assessment and rehabilitation of existing buildings. Linear elastic and non-linear evaluation procedures of 2007 TEC are applied to reinforced concrete buildings. 2007 TEC gives two methods for assessment and rehabilitation of existing buildings. Linear elastic and non-linear static evaluation procedures are proposed for structural evaluation. A performance-based evaluation methodology is used under three levels of earthquake ground motion intensities with different return periods. The performance acceptance criteria are based on demand to capacity ratios at critical sections for the linear evaluation procedures (equivalent seismic load method and mode superposition method) and material strains for the non-linear evaluation procedures (the non-linear static pushover analysis). Member performance limits are described for three damage levels, considering the anticipated failure mode and ductility capacity of each member. Structural performance is then calculated by accounting for the distribution of member damages over the building.. Consistency between the results of the methods used for seismic evaluation of existing buildings is of prime importance. For this purpose, an irregular structure of in plane (A3 type) and 3 story is considered. The target performance level of the building is obtained by applying the linear and the non-linear evaluation procedures. The results are given comparatively on the target performance level of the building. Furthermore non-linear procedure results are compared between 2007TEC and FEMA-440 Capacity-Spectrum Method (CSM) and Displacement-Coefficient Method (DCM)
Keywords- 2007 TEC, FEMA-440, Seismic Performance, Linear and Non-Linear Evaluation Procedures.

1. INTRODUCTION

Analysing structures for various levels of earthquake intensity and checking some local and/or global criteria for each level has been a popular academic exercise for the last couple of decades, but the crucial development that occurred relatively recently was the recognition of the necessity for such procedures by a number of practising engineers influential in code drafting. In Turkey, after a number of recent earthquakes, (especially 1999 Kocaeli and Duzce earthquakes), it was realised that the structures built in industrialised countries aware of the seismic risk are safe enough. One the other hand, the cost of damage inflicted in these structures by earthquakes, as well as the indirect cost resulting from business interruption, need for relocation, etc., can be difficult to tolerate. These points to the need to address the problem of designing a structure for multiple performance levels (limit states), 2007 TEC which gives information for buildings to be built in disaster areas includes chapters (Chapter 2 and 3) where methods are given for reinforced concrete buildings to be designed. The Code also has a new chapter (Chapter 7) where the linear and the non-linear evaluation methods are given for seismic safety evaluation of existing buildings[1]. The assessment procedures in the 2007 TEC are based on linear elastic analyses (equivalent seismic load method, mode superposition method); non-linear analyses (pushover analysis with equivalent seismic load method and mode superposition method) and non-linear time history analysis. The linear methods can be regarded as an extension of the method used for the newly designed buildings to the existing buildings. The Code assumes a specific seismic load reduction factor R_a by requiring precautions for obtaining a structural system of high ductility. However, in existing building demand and capacity ratio of the cross sections evaluated and compared to their limiting values given in the Code. The main reason of the difference is due to variations of ductility in the members of the existing buildings. It is possible to ensure a specific ductility level in the buildings to be designed. However, one has to take into account the present level of ductility in the existing building. The non-linear evaluation method considers the elasto-plastic behaviour of the structural system and has two application procedures: Incremental equivalent seismic load by considering contributions of the single mode or multi modes and the non-linear analysis of the system.

The damage caused by the 1989 Loma Prieta and 1994 Northridge, in California – USA, made it possible to reconsider not only the current performance criteria regarding the strength of materials but also add more realistic criteria based on displacement and strain. With this concept, Guidelines and Commentary for Seismic Rehabilitation of Buildings – the ATC-40 [2] Project by the Applied Technology Council (ATC), and NEHRP Guidelines for the Seismic Rehabilitation of Buildings – FEMA-356 [3] by the Federal Emergency Management Agency (FEMA) have been developed. Later on, in order to examine the results further on, FEMA-440 [4] have been developed. Furthermore FEMA-440 evaluates the capacity-spectrum method and displacement-coefficient method that are anticipated in ATC-40 and FEMA-356 displacement-coefficient method.

The main objective of this study is to assess the seismic performances of the selected building by the linear elastic and non-linear evaluation procedures. After the assessment, a critical comparative evaluation can bedone from obtained results. Global performances of the buildings are estimated from the member performances and from the inter-storey drifts for both two methods. The results are compared to each other, and critically evaluated. In this study, SAP2000 program is used for linear and the non-linear evaluation methods [5].

2. DESCRIPTION OF STRUCTURE

A three story irregular structure in plane (A3 type) is considered to represent low-rise RC buildings for the study. This consists of a typical beam–column RC frame building with no shear walls, located in a high-seismicity region of Turkey. 3-story building is designed according to Earthquake Code, considering both gravity and seismic loads (a design ground acceleration of 0.4*g* and soil class Z3, which is similar to class C soil of FEMA-356 (2000), are assumed). Material properties are assumed to be 20MPa for the concrete compressive strength and 420 MPa for the yield strength of both longitudinal and transverse reinforcements. 3-story building is 12.5 m by 12.5 m in plan (Fig. 1). The story heights are 3.2 m.

The dimensions of columns are 450 mm ×450 mm. All beams are T-shaped and their dimensions are 250 mm ×450 mm. The effective width of the beams are calculated according to TS500[6]. The vertical loads consist of dead and live loads of slabs, wall loads on beams and dead loads of columns and beams. Modal properties of the vibration modes and related effective mass ratios are given in Table 1.

The following methods are implemented for a selected building: The first method is to conduct linear elastic analysis, combined with the capacity analysis in order to determine the column and beam capacities of each building under seismic effects. Then the demands to capacity ratios are determined in order to decide on the member performances. From this analysis, the performance level of the building is determined by comparing the related demand to capacity ratios with the limit values proposed in the 2007 TEC.

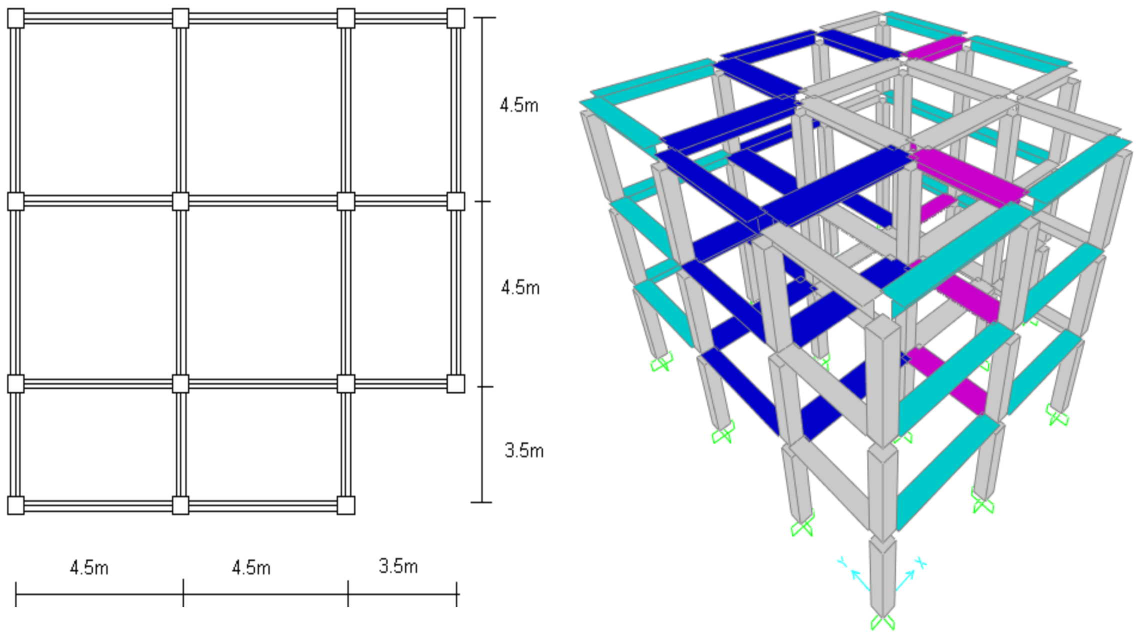

Fig. 1. Plan and three dimensional view of 3-story building

Table 1. Periods of vibration modes and related effective mass ratios

X Direction			Y Direction			Z Direction	
Mode	**Period (s)**	**ux (%)**	**Mode**	**Period (s)**	**uy (%)**	**Mode**	**Period (s)**
1	0.50	88	2	0.50	88	3	0.44
4	0.16	10	5	0.16	10	6	0.14
8	0.09	2	7	0.09	2	9	0.08

Third, sixth and ninth modes appear to be torsinol ones.

Table 2. Dynamic characteristic of 3-story building

Mode Amplitudes	Story Heights	1. mode	2. mode	4. mode	5. mode	7. mode	8. mode
		X Direction	Y Direction	X Direction	Y Direction	X Direction	Y Direction
Φ_3	9.6 m	1.000	1.000	-1.000	-1.000	1.000	1.000
Φ_2	6.4 m	0.778	0.778	0.411	0.411	-1.529	-1.529
Φ_1	3.2 m	0.374	0.374	0.908	0.908	1.422	1.424
Φ_0	0 m	0.000	0.000	0.000	0.000	0.000	0.000

The second method is to conduct non-linear static analysis (pushover) for a building. This pushover analysis is performed by a 3D model in SAP2000. Instead of default values for hinge properties in SAP2000, the moment curvature diagrams are obtained by an Excel Macro as well as the interaction diagrams for each member of the building. Using the results obtained from analysis, the performance level of the building is estimated by comparing strain values corresponding to plastic rotations with the limit values of strain values given in the proposed earthquake code. The results obtained by the procedures discussed in these two methods are compared in order to verify the procedures and the performance limit states proposed by the 2007 TEC. Previously performed similar work has been discussed in the regular structure and mode response method was not used in analysis [7, 8, 9, 10]. The seismic evaluation is available according to other countries code [11, 12, 13, 14].

2.1. Linear Elastic Evaluation Procedures

Performance limits of sections are obtained by determining the demand/capacity ratios (member r factors) of each of them. There are two linear analysis methods given in the 2007 TEC. They are Equivalent Seismic Load Method and Mode Superposition Method. These analyses are employed for performance assessment.

Equivalent seismic load method analysis is limited to 8 story buildings with total height not exceeding 25 m without basement, and not possessing torsion irregularity. While calculating total equivalent seismic load, seismic load reduction factor Ra = 1 is taken. The right side of the equation is multiplied with λ coefficient. This λ coefficient is taken 0.85 for buildings with more than two stories except basement. Mode superposition method can be applied to all buildings without any restrictions. The signs of internal member forces and capacities under an earthquake excitation direction are taken as the signs consistent with the dominant mode shape at this direction [15].

Damage limits are expressed in terms of the demand/capacity ratios for ductile members at their critical cross sections. Ductile concrete frame members are controlled by the flexural failure mode where shear capacity exceeds shear force developed when the member reaches its flexural capacity. The demand/capacity ratio for beams and columns is the ratio of earthquake moment to the residual capacity moment at the critical section, where the residual capacity moment is the difference between the flexural capacity and the dead load moment. Demand/capacity ratios are used in determining the member damage levels of ductile member using linear analysis methods.

$$r = \frac{M_E}{M_A} = \frac{M_E}{M_K - M_D} = \frac{M_E}{M_K - M_{G+Q}} \leq r_s \tag{1}$$

The calculated member r factors for beams and columns are compared with the r limits given in 2007 TEC to determine the member damage regions in accordance with Fig. 2.

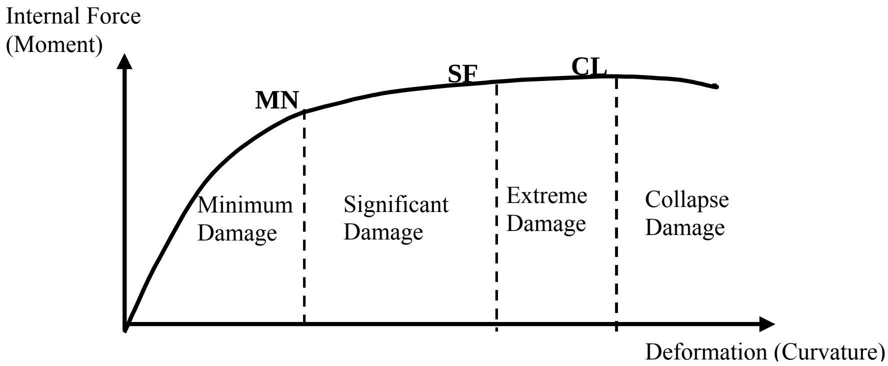

Fig. 2. Damage limits and damage states in a ductile member.

Structural members are classified as “ductile” and “brittle” with respect to their mode of failure in determining the damage limits. Member damage levels for ductile structural members are divided into three limits. They are 3 Minimum Damage Limit (MN), Safety Limit (SF) and Collapse Limit (CL) as indicated in Fig. 2. The corresponding damage regions are also given on the same figure. Structural members whose damage levels are less than MN limit are considered in Minimum Damage Region (MD), between MN and SF limits are considered in Significant Damage Region (SD), between SF and CL limits are considered in Extreme Damage Region (ED). And if the members damage levels are greater than CL limit, they are in Collapse Damage Region (CD).

In reinforced concrete beam-column joints, joint shear capacity should be larger than the shear at the horizontal cross section transmitted from the column above and the longitudinal reinforcement of beams spanning into the joint when they reach their yield capacity. Otherwise all jointing members are accepted as brittle. All capacities are calculated by employing the existing material strengths. In the linear elastic evaluation procedures, structural analysis is performed also by SAP2000 to obtain the demand values, whereas the capacity values are calculated by another Excel Macro. The results of linear procedures for 3 story RC building are given in Table 3, 4.

Table 3. Damage regions and ratios for beams at x and y directions (linear procedures)

Story No	Equivalent Seismic Load Method						Mode Superposition Method						
	MD	%	SD	%	ED	CD	MD	%	SD	%	ED	%	CD
1	0	0	11	100	0	0	0	0	9	82	2	18	0
2	0	0	11	100	0	0	0	0	11	100	0	0	0
3	11	100	0	0	0	0	11	100	0	0	0	0	0

Table 4. Damage regions and ratios for columns at x and y directons (linear procedures)

Story No	Equivalent Seismic Load Method						Mode Superposition Method					
	MD	%	SD	%	ED	CD	MD	%	SD	%	ED	CD
1	1	7	14	93	0	0	0	0	15	100	0	0
2	12	80	3	20	0	0	10	67	5	33	0	0
3	15	100	0	0	0	0	15	100	0	0	0	0

2.2. Non-linear Evaluation Procedures

The purpose of the non-linear analysis methods is to determinate the structural performance of existing buildings under seismic loads effect. After the plastic rotation demands of ductile members are calculated, these values are compared with deformation capacities defined in Table 5.

Incremental static pushover analysis can be employed for performance assessment. Incremental equivalent seismic load method is limited to 8 story buildings with total height not exceeding 25 m without basement, and not possessing torsion irregularity. Incremental mode superposition method is incremental application of linear mode superposition method and can be applied to all buildings. Non-linear flexural behaviour in frame members are confined to plastic hinges, where the plastic hinge length L_p is assumed as half of the section depth (L_p=h/2). Pre-yield linear behaviour of concrete sections is represented by cracked sections, which is $0.40EI_o$ for beams and varies between $(0.40\text{-}0.80)EI_o$ with the axial stress for columns. Strain hardening in the plastic range may be ignored, provided that the plastic deformation vector remains normal to the yield surface.

The objective is to carry out non-linear static analysis under incrementally increasing seismic forces distributed in accordance with the dominant mode shape in the earthquake excitation direction. Seismic forces are increased until the earthquake displacement demand is reached. Internal member forces and plastic deformations are calculated at the demand level. A capacity diagram is obtained from the incremental analysis which is expressed in the "base shear force - roof displacement" plane. Then the coordinates of this plane is transformed into "modal response acceleration versus modal response displacement".

Building displacements, internal deformations and forces can be calculated at the modal displacement demand by appropriate transformations using the first mode properties. The plastic rotations obtained at the member plastic hinge locations are then used for calculating the plastic curvature demands at these critical sections.

$$\varphi_p = \frac{\theta}{L_p} \tag{2}$$

Concrete compressive strains and steel tensile strain demands at the plastic regions are calculated from the moment-curvature diagrams. Moment-curvature diagrams of the critical sections are obtained by using appropriate stress-strain rules for concrete and steel. Finally, the calculated strain demands are compared with the damage limits given in Table 5 to determine the member damage states in view of Fig. 2. In Table 5, ε_{cu} is the concrete strain at the outer fibre, ε_{cg} is the concrete strain at the outer fibre of the confined core, ε_s is the steel strain and (ρ_s/ρ_{sm}) is the ratio of existing confinement reinforcement at the section to the confinement required by the 2007 TEC.

Table 5. Concrete and steel strain limits at the fibres of cross section for damage state

Damage Limit	Strain Limit	
	Concrete Strain (ε_{cu})	Steel Strain (ε_{su})
Minimum Damage Limit (MN)	$(\varepsilon_{cu})_{MN} = 0.0035$	$(\varepsilon_s)_{MN} = 0.010$
Safety Damage Limit (SF)	$(\varepsilon_{cu})_{SF} = \min\left[0.0035 + 0.010\frac{\rho_s}{\rho_{sm}}\right] \leq 0.0135$	$(\varepsilon_s)_{SF} = 0.040$
Collapse Damage Limit (CL)	$(\varepsilon_{cu})_{CL} = \min\left[0.0040 + 0.014\frac{\rho_s}{\rho_{sm}}\right] \leq 0.018$	$(\varepsilon_s)_{CL} = 0.060$

2.2.1. Incremental Equivalent Seismic Load Method (IEM)

Non-linear static analysis have been performed using SAP2000 which is a general-purpose structural analysis program. A three-dimensional model of structure is created in SAP2000 to carry out non-linear static analysis. Beam and column elements are modelled as non-linear frame elements with lumped plasticity by defining plastic hinges at both ends of beams and columns. SAP2000 provides default or the user-defined hinge properties options to model non-linear behaviour of components.

Inel and Ozmen studied possible differences on the results of pushover analysis due to default and user-defined non-linear component properties [16]. They observed that although the model with default hinge properties seemed to provide reasonable displacement capacity for the well-confined case, the displacement capacity was quite high compared to that of the poorly-confined case. Thus, this study implements user-defined hinge properties. The definition of user-defined hinge properties requires moment–curvature analysis of each element. Mander et al. (1998) model for unconfined and confined concrete and typical steel stress–strain model with strain hardening for steel are implemented in moment–curvature analyses [17]. Transverse reinforcements are considered for the potential plastic hinge regions, with 100 mm spacing representing the ranges in typical construction. Strain-hardening of longitudinal reinforcement has been taken into account and the ultimate strength of the reinforcement is taken as 420 MPa. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. Gravity loads were in place during lateral loading. In all cases, lateral forces were applied monotonically in a step-by-step non-linear static analysis. The applied lateral forces were proportional to the product of mass and the first mode shape amplitude at each story level under consideration. P–Δ effects were taken into account. Although the first mode shape is used in this study, a non-modal shape vector such as an inverted triangular shape may be used for the lateral load pattern. In pushover analysis, the behaviour of structure is characterized by a capacity curve that represents the relationship between the base shear force and the displacement of the roof. Base shear capacity – roof displacement curve for non-linear procedures is converted into spectral displacement – spectral acceleration curve and called as modal capacity curve as shown in Fig. 3.

Using the results obtained from analysis, the performance level of the building is estimated by comparing strain values corresponding to plastic rotations with the limit values of strain values given in the proposed earthquake code. Building earthquake performance level is determined after determining the member damage states, as explained above. Damage limits and ratios for performance levels are given for equivalent seismic load method as shown in Table 6.

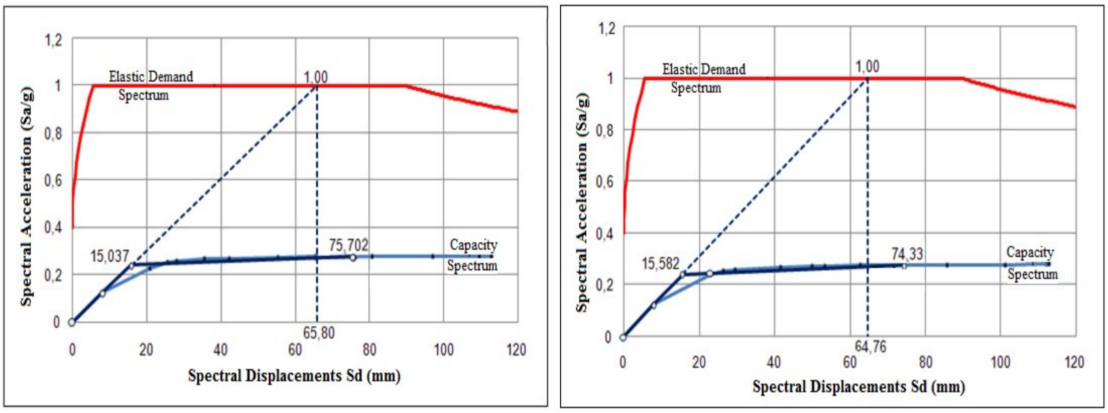

Fig. 3. Modal capacity curve at x and y directions for incremental equivalent seismic load method

Table 6. Damage limits and ratios for beams and columns at x and y directions

Members	Story No	Incremental Equivalent Seismic Load Method					
		MN	%	SF	%	CL	%
Beams	1	0	0	11	100	0	0
	2	0	0	11	100	0	0
	3	11	100	0	0	0	0
Columns	1	0	0	15	100	0	0
	2	15	100	0	100	0	0
	3	15	100	0	0	0	0

2.2.2. FEMA-440 Evaluation Procedure

FEMA-440 evaluates and improves the simplified inelastic analysis procedures. The recommendations in FEMA-440 resulted in immediate improvements in the nonlinear static analysis procedures. The improved procedure for the Capacity-Spectrum Method consists of new estimates of equivalent period and damping. The capacity-spectrum method uses the secant corresponding to the maximum inelastic displacement. Notably, there is a simple algebraic relationship relating the inital and periods if they are both known. Subsequently, the graphic representation of a method can be de-coupled from the underlying relationships that are used to estimate displacements. The peak displacement of a nonlinear system is estimated as the intersection of the capacity curve and an elastic response spectrum that is reduced to account for energy dissipated by the yielding structure. The underlying basis of the capacity-spectrum method which is thoroughly documented in ATC-40 is the concept of an "equivalent linear" system. Displacement-coefficient method tends to overestimate the global deformation demands with respect to the capacity spectrum method. Improvements to the displacement-coefficient method were proposed in FEMA-440.

FEMA-440 defines three limit states, related to structural damage. The limit states are immediate occupancy (IO), life safety (LS) and collapse prevention (CP) that correspond to minimum damage limit, safety damage and collapse damage limit in 2007 TEC. Damage limits and ratios for performance levels are given for FEMA-440 Capacity-Spectrum and Displacement-Coefficient methods as shown in Table 7.

Table 7. Damage limits and ratios for beams and columns at x and y directions

Members	Story No	Fema-440 Capacity-Spectrum Method						Fema-440 Displacement-Coefficient Method					
		IO	%	LS	%	CP	%	IO	%	LS	%	CP	%
Beams	1	3	27	8	73	0	0	0	0	11	100	0	0
	2	11	100	0	0	0	0	11	100	0	0	0	0
	3	11	100	0	0	0	0	11	100	0	0	0	0
Columns	1	0	0	15	100	0	0	0	0	15	100	0	0
	2	15	100	0	0	0	0	15	100	0	0	0	0
	3	15	100	0	0	0	0	15	100	0	0	0	0

3. COMPARISONS BETWEEN THE ANALYSIS RESULTS

The reference design spectrum in the Code has 10% probability of exceeding in 50 years. Based on Turkish strong motion data, it is estimated that the spectral ordinates for 50% probability of exceeding in 50 years are half of the reference spectrum whereas the ordinates for 2% probability of exceeding in 50 years are 1.5 times that of the reference spectrum. Accordingly, the target performance levels of buildings are described in 2007 TEC.

Building earthquake performance level is determined after determining the member damage states as it published in 2007 TEC. Three performance levels, immediate occupancy (IO), life safety (LS), and collapse prevention (CP) are considered as specified in this code and several other international guidelines such as ATC-40, FEMA-356 and FEMA-440.

The selected performance level for building is the life safety level. For equivalent seismic load method, 100% of the beams are in significant damage region in the first and second story. In the third story 100% of the beams are in minimum damage region. For mode superposition method, in the first story, 82% of the beams are in significant damage region and %18 of the beams are in extreme region. In the second story %100 of the beams are in significant damage region and %100 of the beams are in minimum damage region in the third story as it calculated in equivalent seismic load method.

The columns are calculated for each direction as shown in Table 4. For equivalent seismic load method, in the first story, 93% of the columns are in significant damage region and 7% of the columns are in minimum damage region. In the second story, 80% of the columns are in minimum damage region and 20% of the columns are in significant damage region. In the third story 100% of the columns are in minimum damage region. For mode superposition method, 100% of the columns are in significant damage region in the first story. 67% of the columns are in minimum damage region and 33% of the columns are in significant damage region in the second story. In the third story, all columns are in minimum damage region.

For incremental equivalent seismic load method, at both directions in the first and second story, all beams provide safety limit. In the third story 100% of the beams provide minimum damage limit at x and y directions. 100% of the columns provide safety limit at both axes in the first story. In the second and third story, all columns provide minimum damage limit.

Damage limits are calculated for FEMA-440 capacity-spectrum and displacement-coefficient methods at both directions as shown in Table 7. In the first story, for capacity-spectrum method, 27% of the beams provide immediate occupancy level and 73% of the beams provide life safety level. Nonetheless, 100% of the beams provide life safety level for displacement-coefficient method. All beams provide immediate occupancy level in the first and second stories for both methods. 100% of the columns provide life safety level in the first story and all columns provide immediate occupancy level in the second and third stories for both methods.

Finally, story–drift ratio are calculated at each story and compared with the limits defined in the code. Story drift ratio is not exceeding 2% in any story. In all cases the building satisfies life safety (LS) level.

There are total results of beams and columns for linear and non-linear evaluation procedures at figures below. The results are the superposition of the x and y directions.

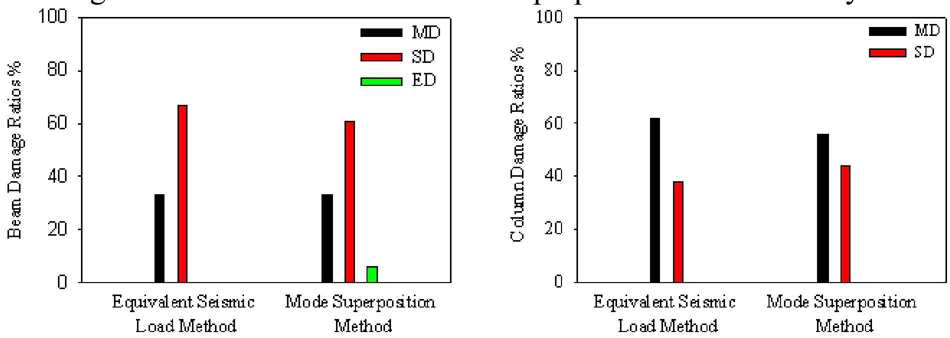

Fig. 4. Total results for linear procedures

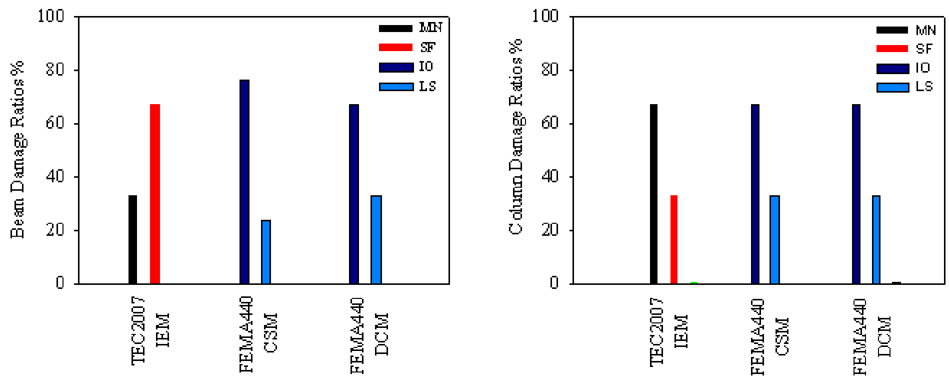

Fig. 5. Total results for non-linear procedures

4. CONCLUSIONS

It took time to implement the performance based methods in the 2007 TEC. In this paper, the results of linear evaluation procedures (equivalent seismic load method and mode superposition method) and non-linear evaluation procedure (non-linear static pushover analysis) according to 2007 TEC and FEMA-440 capacity-spectrum and displacement-coefficient methods are studied. The selected structure is a 3-storey RC frame building and it has an irregularity in plane (A3 type).

Fig. 4 shows results of member damage ratios for linear procedures that are defined in 2007 TEC. Non-linear procedures results according to 2007 TEC, FEMA-440 capacity-spectrum and displacement-coefficient methods are seen in Fig. 5. As it is expected, there are differences between them, since the three methods base on different assumption having various degrees of approximations. However, the results seem to be within the acceptable limits. Due to the dissimilar assumption the evaluation methods do not yield identical results for the performance levels which seem to be not very surprising.

The mod superposition method of the 2007 TEC generally gives more conservative member damage levels as compared with those given by the equivalent seismic load method as expected. All members are ductile in the existing building. There are some structural members in extreme damage region in mod superposition method. So, the results of linear evaluation procedures are more conservative than the results of non-linear evaluation procedure. Non linear procedure results of 2007 TEC and FEMA-440 displacement-coefficient method are more conservative than the results of FEMA-440 capacity spectrum method. Although the building has an irregularity, the span lengths at both directions and story heights have equal values. Because of this reason, the same performance the same damage limit (life safety) is provided for all cases

2007 TEC gives equal importance to the linear and the non-linear procedures but the performance evaluation by using both procedures for a building can be different. This may cause conflict between owners and designers because both procedures are legally valid. The non-linear evaluation procedure provides detailed information about the seismic behavior of the structure. However, the procedure requires more material and structural data. One should keep in mind that when a structure is a non-engineered one or when the geometrical and mechanical structural data are not reliable, then it is questionable to apply the non-linear procedure.

All Procedures in 2007 TEC are complicated for hand calculations and engineers need to trust the software products, that automate the procedures, for the results. This will make the procedures “black box” and engineers would never know if there is something wrong with the software procedure. Procedures can be simplified for better understanding. By using one of the evaluation methods one should keep in mind that the outcomes of the evaluation are not more accurate as the data used for procedure such as, the geometry, the reinforcement quality and detail and the concrete quality in the building.

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