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Performance evaluation of reinforced concrete buildings with softer ground floors

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Professional paper

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Performance evaluation of reinforced concrete buildings with softer ground floors

Determination of seismic performance is a significant task in the performance-based seismic analysis of structures. The consistency between analysis results obtained according to various codes is highly significant for seismic evaluation of structures. Non-linear performance of three reinforced concrete buildings having a soft storey irregularity is studied based on the finite elements analysis according to FEMA-356, FEMA-440, and Turkish Earthquake Code - 2007. Damage situations, modal properties, storey drift ratios, and global performance levels are comparatively presented for each structure, and appropriate suggestions are given.

Key words:
finite elements analysis, irregular structures, non-linear procedures, seismic performance

Stručni rad

R. Tuğrul Erdem

Ocjena ponašanja armiranobetonskih zgrada s mekim prizemljem

Određivanje seizmičkog ponašanja je važan zadatak seizmičke analize prema zahtijevanom ponašanju konstrukcije. Podudaranje rezultata dobivenih prema različitim normama od velike je važnosti u postupku seizmičkog ocjenjivanja konstrukcija. U radu je opisana nelinearna analiza ponašanja triju armiranobetonskih zgrada s mekim prizemljem primjenom metode konačnih elemenata prema normama FEMA-356 i FEMA-440, te turskim potresnim normama iz 2007. Za svaku konstrukciju je prikazano stanje oštećenosti, modalna svojstva, međukatni pomaci i globalne razine ponašanja te su dani odgovarajući prijedlozi.

Ključne riječi:
proračun s pomoću konačnih elemenata, nepravilne konstrukcije, nelinearni postupci, seizmičko ponašanje

Fachbericht

R. Tuğrul Erdem

Beurteilung des Verhaltens von Stahlbetongebäuden mit weichem Geschoss

Die Bestimmung des seismischen Verhaltens ist eine wichtige Aufgabe bei der verhaltensbasierten Erdbebenanalyse. Eine Übereinstimmung von Resultaten, die aus verschiedenen Normen folgen, spielt eine große Rolle bei der seismischen Beurteilung von Tragwerken. In dieser Arbeit wird mittels der Finite-Elemente-Methode eine nichtlineare Analyse des Verhaltens von drei Stahlbetongebäuden mit weichem Geschoss gemäß der Normen FEMA-356 und FEMA-440, sowie gemäß der türkischen Erdbebennormen aus dem Jahre 2007 durchgeführt. Für jedes Tragwerk werden Schadensbilder, modale Eigenschaften, Stockwerksverschiebungen und das globale Verhalten dargestellt, sowie entsprechende Vorschläge gegeben.

Schlüsselwörter:
Finite-Elemente-Analyse, unregelmäßige Tragwerke, nichtlineare Methoden, seismisches Verhalten

1. Introduction

Concrete is resistant to pressure and can be formed quite easily. It is widely used in construction of structures such as buildings, dams, water tunnels, bridges, and roads. Besides, it is used as protection against nuclear radiation in modern structures. Concrete can be used as a structural member and a decorative material. It is preferred in construction technology because of its noise insulation and fire resistance properties. In addition, it is shapeable, economical, and does not require frequent maintenance.

Reinforced concrete structures make up most of the existing structure stock worldwide. Due to their high rigidity, long service life, and resistance against earthquake and fire, reinforced concrete structures are used for many purposes. Analysis of these structures for different levels of earthquake intensity, and for determination of damage levels, has been in the focus of interest of scientists and engineers for the past several decades. As horizontal loads cause horizontal displacements, damage due to such displacements is observed after earthquakes. Studies on seismic safety of structures have been conducted all over the world [1-5].

Designers are required to ensure proper fire protection of structures. Due to its internal structure, concrete is considered to be the most fire resistant construction material. When subjected to fire, concrete performs well both as a structure and as a material. Heat transfer of concrete is also very slow. Because of this property, concrete walls in a building can effectively protect neighbouring spaces against fire. Fire resistance of materials and members is determined according to ASTM E119 [6]. Concrete members exhibit decent performance when subjected to ASTM E119 test. It is stated that lightweight concrete improves fire resistance of structural members.

Performance based design is a significant discipline in which design criteria are stated with regard to specific objectives to be achieved when a structure is subjected to seismic forces. The general idea is to relate performance objectives to an appropriate level of damage. The main purpose is to meet these objectives, i.e. to perform analysis in a desired manner under seismic loads. The number of studies about performance based design as related to damage level assessment has increased in recent years [7, 8]. The performance based is accepted as an effective new approach to traditional design methods, which brings specific improvements. Researchers generally agree that, in the near future, the performance based design will become increasingly represented in relevant codes [9-11].

Behaviour of structural systems can be evaluated more accurately according to non-linear methods. These methods have become more significant in determining the damage while assessing the inelastic behaviour of structures in performance analysis. While linear methods are force-based analyses in which damage situations are decided according to demand/capacity values, non-linear methods are deformation-based, and so more parameters are required to evaluate structural members. In general, all codes require that buildings have to withstand minor scale earthquakes without any damage to structural members. Besides, the buildings shall provide life safety in big scale earthquakes, and they shall not totally collapse in large scale earthquakes, so as to minimize the losses.

The concept of a reliable performance evaluation is based on specific techniques, from the design to the verification of analysis in structural engineering. The performance objectives are considered successful if performance parameters, after completing the analysis, exceed the limits defined in codes. In addition, storey drift ratios are accepted as important parameters in performance evaluation. Storey drift values are significant for performance levels of multistorey buildings under seismic loads. Since big storey drifts usually cause heavy damage or even collapse, a uniform drift distribution is required among all storeys of a structure.

Durability, rigidity and ductility are main parameters to be satisfied by earthquake resistant buildings. Horizontal and vertical discontinuities lead to structural irregularities in multistorey buildings. Ground floors of buildings are sometimes designed differently from upper ones due to economic considerations. Storey heights and walls may differ in these floors. In such cases, these floors are referred to as soft storeys, where big deformations with failures are most frequent during earthquakes [12, 13]. Seismic behaviour of buildings with the soft storey irregularity can only be determined by non-linear analyses. In this way, possible levels of damage can properly be observed.

Non-linear analyses of existing irregular reinforced concrete structures with 3, 5 and 7 storeys are performed in this paper. In such structures, the first floor is different from the remaining floors. The weight of these structures is calculated, and the period and effective mass ratio values are determined. Damage ratios of structural members are obtained according to non-linear methods defined in FEMA-356 (DCM), FEMA-440 (DCM), and TEC-2007. Finally, global performance levels of the structures, and storey drift ratios, are determined. The SAP2000 finite elements program is used for these analyses [14]. The results are compared and appropriate suggestions are given.

2. Soft storey case

The soft storey irregularity is one of the main reasons for heavy damage and collapse of multistorey buildings after seismic events. During earthquakes, ground floors with different storey heights usually behave differently compared to other storeys. There are many structures with soft storeys in the first floor. These buildings are mostly located on the main streets where they are used for commercial purposes, e.g. department stores, restaurants, banks and showrooms. These places are usually enclosed with glass windows. Brick walls are placed just above the soft storey.

In such situations, serious problems occur in the soft storey during an earthquake. Significant damage and sudden collapses can be observed due to big deformations and energy dissipation at the soft storey columns. Behaviour of a structural system having soft storey irregularity under lateral loads is presented in Figure 1.

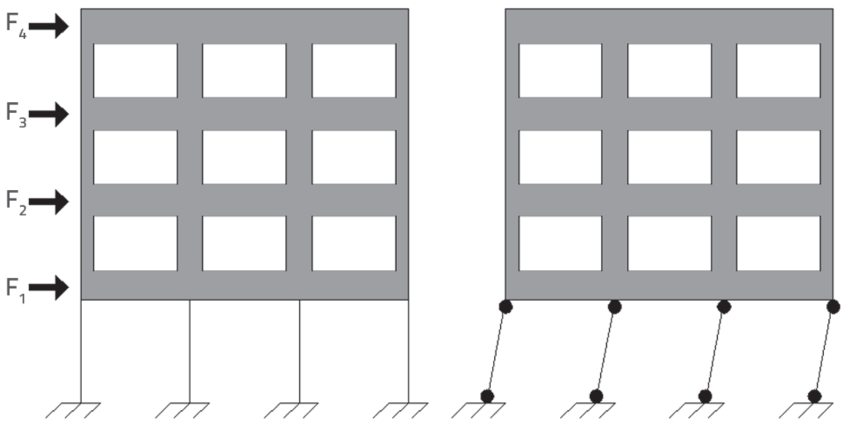

Figure 1. Behaviour of soft storey

Figure 2. Damage to soft storeys

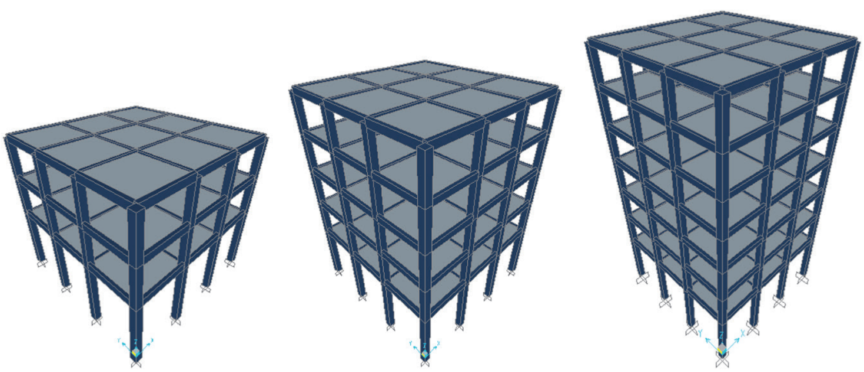

Figure 3. Three dimensional views of structures

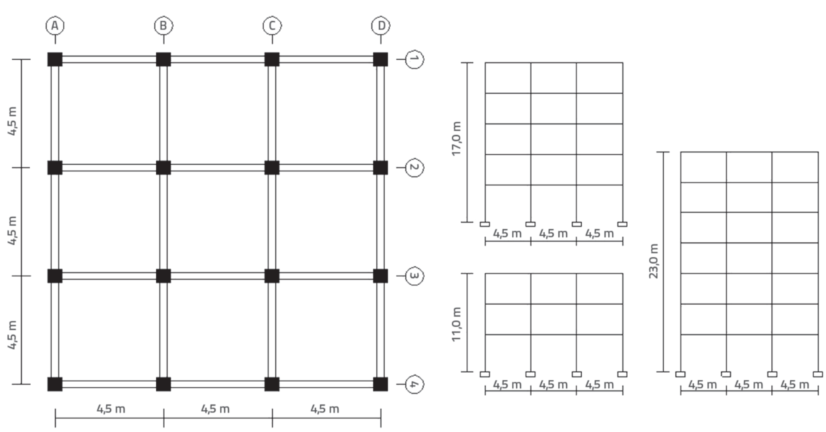

Figure 4. Geometry of buildings

The soft storey irregularity is usually seen in multistorey apartment buildings with large openings. Since deformations are concentrated in the first storeys, these storeys have been mostly affected by earthquakes registered in recent years [15, 16]. These types of buildings usually have a poor load carrying capacity, especially when subjected to lateral loads. While massive damage is usually observed in the ground storeys, the damage to upper storeys is limited. Some examples of buildings with soft storey after earthquakes are presented in Figure 2 [17].

3. Description of buildings

The sample buildings have 3, 5, and 7 storeys, respectively. These structures are intended to represent typical residential low-rise, medium-rise and high-rise reinforced concrete buildings in urban areas. The frame buildings have typical column-beam sections without any shear walls. Since storey height is an important parameter in the soft storey irregularity, it should be noted that the storey height is 3 m at all levels except for the ground floor which is 5 m in height. Outer axes of ground storeys are covered by glass windows. Walls are 20 cm in thickness in outer axes and 10 cm in thickness in the remaining storeys of the buildings under study. The soil class is Z3 according to TEC-2007, which is similar to class C defined in FEMA. The buildings are assumed to be located in an earthquake-prone area. Three-dimensional finite element models are shown in Figure 3.

The buildings are 13.5 m by 13.5 m in plan. Material properties are assumed to be 25 MPa for the concrete compressive strength and 420 MPa for the yield strength of both longitudinal and transverse reinforcement. The height of the slabs is taken to be 12 cm. The plan view and elevation of the buildings are presented in Figure 4. Sections of corner ground-floor columns are bigger compared to sections of other columns. Beam sections are constant on all storeys. Sections of structural members are shown in Table 1. Details of ground-floor column and beam sections are also shown in Figure 5.

Table 1. Section sizes of members

Structure type	Beam sections [cm]	Corner ground-floor column sections [cm]	Other column sections [cm]
3 storey	25 x 50	40 x 40	30 x 40
5 storey		45 x 45	35 x 45
7 storey		50 x 50	40 x 50

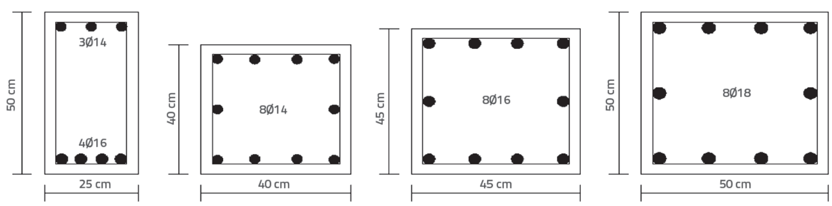

Figure 5. Section details of the beams (25x50) and columns (40x40, 45x45, 50x50)

Table 2. Properties of predominant modes

Structure type	Period [s]	Effective mass ratio [%]
3 storey	0.57	96
5 storey	0.86	93
7 storey	1.14	91

Vertical loads consist of the dead and live loads of slabs, wall loads on beams, and self loads of structural members. Weights of the studied 3, 5, and 7 storey buildings are calculated as 421.4 tons, 686.4 tons, and 941.4 tons, respectively. After analysis of each structure, modal properties are also determined. First periods of predominant modes with the related effective mass ratios are given in Table 2.

After determining the weight and modal properties, plastic hinges are assigned at two ends of columns and beams to perform non-linear analyses. The moment-rotation relationship of members is defined using the SAP2000 finite-elements analysis program. Moment-curvature analyses for structural members are utilized for this purpose according to the Semap analysis program [18], as shown in Figure 6. The modified Kent-Park Model [19] for confined concrete is used in the moment-curvature analyses of members.

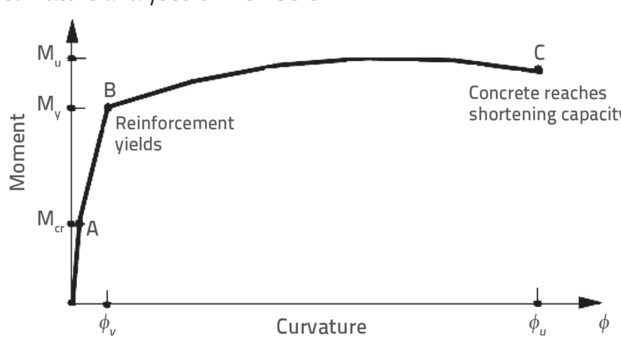

Figure 6. Moment-curvature relationship

The idealized force-deformation relationship of a plastic hinge, shown in Figure 7, is defined using the SAP2000 analysis program. By this curve, the relationship is determined by plastic hinges on structural members. Eight points are required to define the curve. However, four points are sufficient for symmetrically reinforced members.

The unloaded situation of hinge deformation is represented by point A. The yield of a structural member occurs when the F_y strength value in a hinge is reached. After the point B, the force on hinge changes according to deformation. When the displacement value reaches the point C, the plastic hinge reaches the collapsing situation. Finally, the plastic hinge completely loses its strength, and the building failure situation is defined, when points D and E are reached.

Locations of plastic hinges at structural ground-floor members are presented as an example in Figure 8. In this figure, L_p is the length of the plastic hinge.

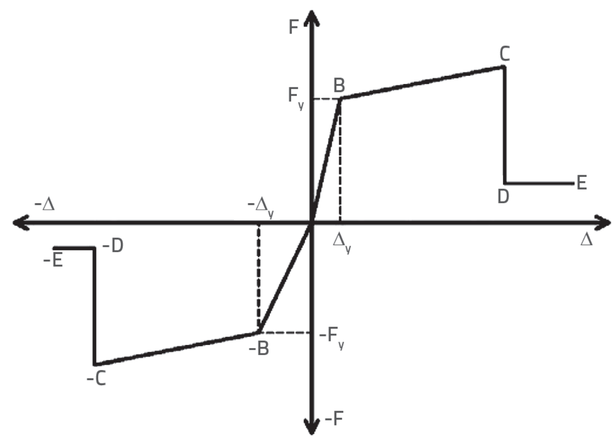

Figure 7. Idealized force-deformation relationship

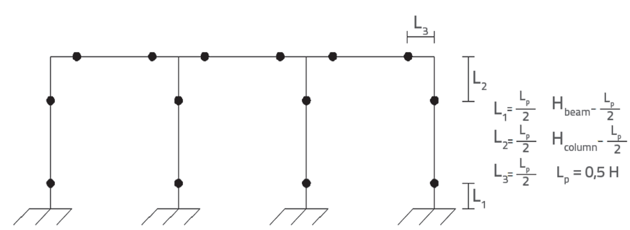

Figure 8. Locations of plastic hinges

The damage situations of the members, and the performance levels of all buildings, are then determined by comparing the plastic-rotation values with limit values defined in FEMA-356 (DCM), FEMA-440 (DCM) and TEC-2007.

4. Non-linear analysis

The non-linear pushover technique is used to evaluate seismic performance of buildings. This technique is the application of gravity loads with lateral loading. The static pushover analysis is a series of non-linear incremental static analyses that are conducted to obtain the lateral deformation and damage situations of structural members. Structural behaviour is characterized by a capacity curve representing the relationship between the base shear and roof displacement. Since reliable and rapid non-linear responses are obtained via the pushover analysis due to its simplicity and minimum computational efforts, it has been widely used by researchers instead of the elastic static or dynamic solution methods [20-22].
Non-linear analysis methods are thoroughly described in many guidelines. In recent years, displacement based methods have been preferred to force-based methods due to direct relationship between performance objectives and damage levels. Main analysis steps for non-linear methods, from the modelling phase to the structural performance checking level, are respectively presented in Figure 9.

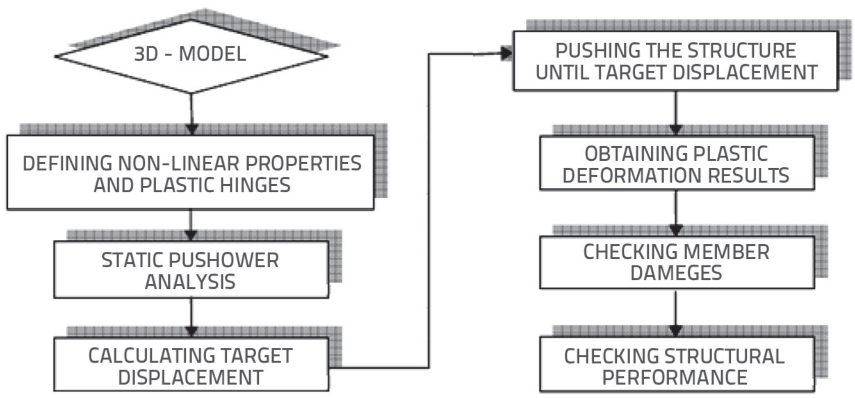

Figure 9. Non-linear analysis steps

The target displacement determination, and the acceptance criteria based on performance limit values, constitute the main difference between non-linear methods defined in codes. There are three performance levels defined similarly in codes, i.e. immediate occupancy (IO), life safety (LS), and collapse prevention (CP). At the IO level, small cracks might be seen in non-structural members but no damage is inflicted on structural members. At the LS level, only limited damages can be observed while life safety is provided. In addition, the lateral stiffness and rigidity of all structural members is preserved. At the CP level, some walls may collapse, and permanent displacements can be observed in the structure. However, the total collapse is prevented. The force-deformation relationship of plastic hinges that are used to define structural performance levels with damage situations are presented in Figure 10. There are five points that describe the behaviour according to material properties, reinforcement level, structural member type, and axial load imposed on the member.

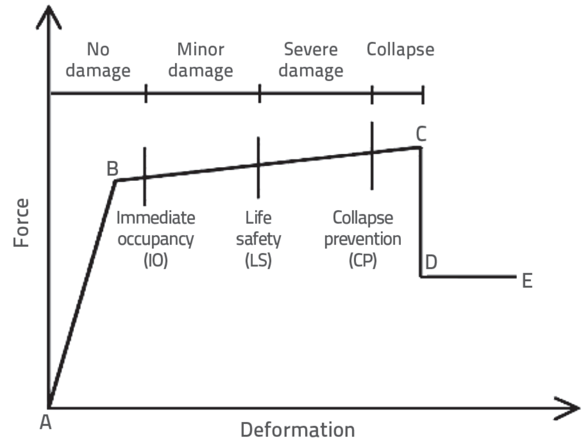

Figure 10. Structural performance levels

4.1. Performance analysis for FEMA

The target displacement value can also be calculated according to the Displacement Coefficient Method, which is defined in FEMA. In this method, the base shear force (V_t) and the peak point displacement (δ_{max}) are obtained after the static pushover analysis. This curve is then idealized and is formed of two lines. While the slope of the first line represents elastic rigidity (K_e), the second line stands for elasto-plastic rigidity (K_s). Areas under the real and idealized capacity curves shall be equal to one other, as shown in Figure 11.

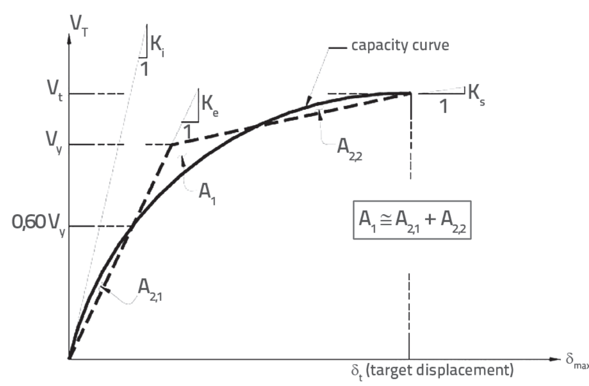

Figure 11. Determination of target displacement according to FEMA

The effective period (T_e) of the structure is calculated according to Eq. (1). While T_i is the elastic period in the related direction, K_i

Table 3. Analysis steps according to FEMA

Coefficient	FEMA-356 (DCM)	FEMA-440 (DCM)
C_o	- The first modal participation factor at the level of the displacement control node. - The modal participation factor at the level of the control node calculated using a shape vector corresponding to the deflected shape of the building at target displacement. - It is explained according to framing system and storey number in the Table 3-2 of FEMA 356.	
C_1	$C_1 = 1{,}00$ for $T_e \geq T_o$ $C_1 = \dfrac{1 + \dfrac{(R_0 - 1)T_0}{T_e}}{R_0}$ for $T_e < T_o$	$C_1 = 1 + \dfrac{(R-1)}{aT_e^2}$ $C_{1(T=0.2sn)}$ for $T < 0{,}2$ s $C_1 = 1{,}0$ for $T > 1{,}0$ s
C_2	Values of different framing systems and Structural Performance Levels shall be obtained from Table 3-3 of FEMA 356.	$C_2 = 1 + \dfrac{1}{800}\left(\dfrac{R-1}{T}\right)^2$ $C_{2(T=0,2sn)}$ for $T < 0{,}2$ s $C_2 = 1{,}0$ for $T > 0{,}7$ s
C_3	$C_3 = 1{,}00 \quad \alpha = \dfrac{K_s}{K_e} > 0$ $C_3 = 1.0 + \dfrac{\lvert\alpha\rvert (R_0 - 1)^{3/2}}{T_e} \quad \alpha = \dfrac{K_s}{K_e} \leq 0$	C_3 coefficient is not considered in FEMA 440.

Table 4. Analysis steps according to TEC-2007

1. Any point V_i, δ_t on the multiple degree of freedom capacity curve is converted to the corresponding point Sai, Sdi on the equivalent single degree of freedom capacity spectrum using the modal mass coefficient and participation factors equations.
2. A point on the capacity spectrum curve is estimated as performance point, and the spectrum curve is idealized with two linear lines.
3. Non-linear spectral displacement, $S_{di1} = C_{R1} \cdot S_{de1}$ Linear spectral displacement, $S_{de1} = \dfrac{S_{ae1}}{\left(\omega_1^{(1)}\right)^2}$ Spectral displacement ratio C_{R1} is determined by initial period $T_1^{(1)}$, $T_1^{(1)} = 2\pi / \omega_1^{(1)}$
4. If the initial period $T_1^{(1)}$ is equal or bigger than the characteristic period T_B (bigger spectrum characteristic period for the local soil class) at acceleration spectrum, then the value of $C_{R1}=1$ is adopted.
5. If the initial period $T_1^{(1)}$ is lower than the characteristic period T_B at acceleration spectrum, $C_{R1} = \dfrac{1 + (R_{y1} - 1)T_B / T_1^1}{R_{y1}}$ R_{y1} is the strength decrement coefficient in the first mode, $R_{y1} = \dfrac{S_{ae1}}{a_{y1}}$
6. After the target performance point is calculated, the converted capacity curve should be made linear with the equal areas rule, and the values of a_{y1}, R_{y1}, C_{R1} should be calculated. The target performance point is initially unknown. That is why several trial and error solutions may be necessary.

is the elastic lateral rigidity of the structure. After determining the effective period value, the target displacement is calculated according to Eq. (2). S_a is the response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration, and g is the acceleration of gravity. Analysis steps according to FEMA-356 (DCM), FEMA-440 (DCM), and Eq. (2) parameters, are given in Table 3. T_o represents a characteristic period of the response spectrum:.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{1}$$

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

4.2. Performance analysis for TEC-2007

The coordinates of capacity curve are changed to modal response, acceleration - modal response displacement, to determine the target displacement value (d_t) according to TEC-2007. This value is calculated according to initial period in TEC-2007, as shown in Figure 12. Analysis steps for TEC-2007 are given in the following Table 4.

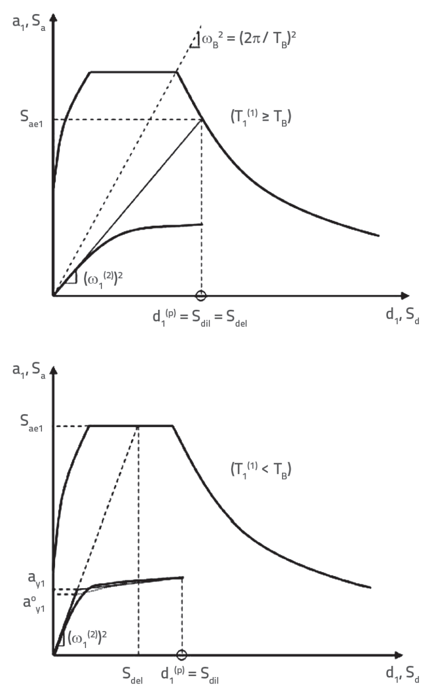

Figure 12. Determination of target displacement according to TEC-2007

5. Results

Performance analysis is a combination of design, construction and evaluation steps. This analysis has become important for identifying load patterns and damage levels, for the purpose of

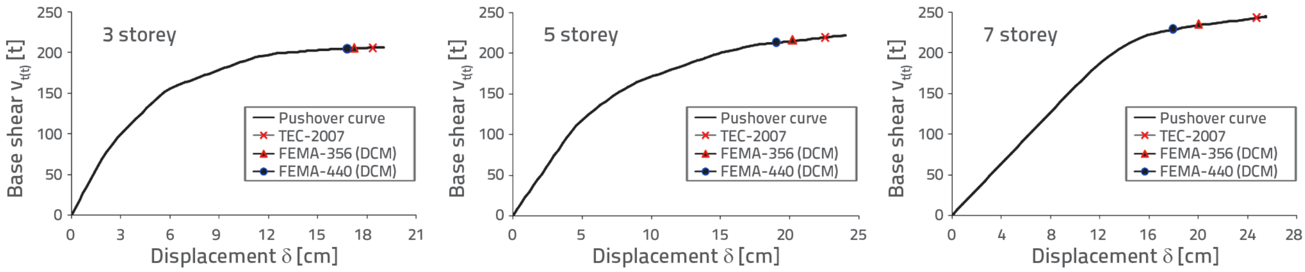

Figure 13. Target displacements of structures

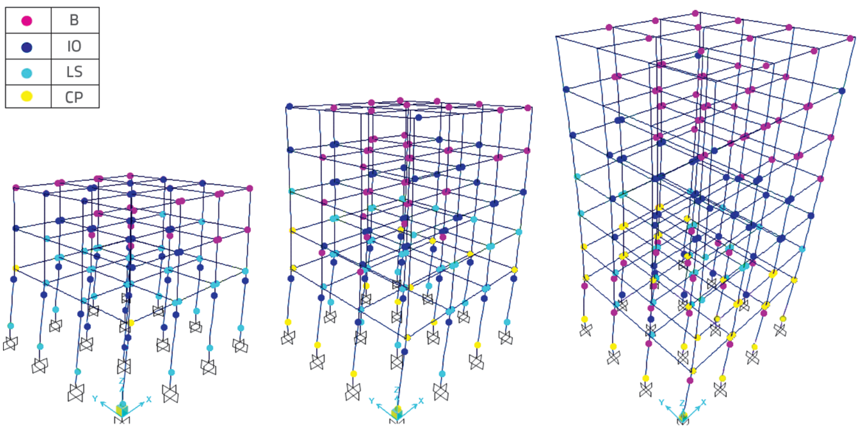

Figure 14. Plastic hinges at target displacements according to TEC-2007

evaluating the inelastic behaviour of structures during seismic events. Researchers agree that future designs will need to satisfy a set of performance objectives. General performance-analysis techniques include different approaches. Significant progress has recently been made in the development of non-linear analysis methods. The strain, deformation, and damage parameters are more reliable than stress values for determining performance of structures.

The main objective of these methods is to determine structural performance in case of seismic loads. The incremental static pushover analysis is usually applied for this purpose. In this analysis the geometry of the structural system and sections, and material properties with inelastic behaviour, are taken into consideration with regard to gradual application of horizontal forces.

The capacity curve, which represents the relationship between the base shear force and the roof displacement, is obtained for each structure after the static pushover analysis. Target displacements are calculated for FEMA-356, FEMA-440, and TEC-2007, and are marked on capacity curves as shown in Figure 13. The buildings are pushed to these target displacement values to determine damage levels for each code. After non-linear analyses are performed and the buildings are pushed to target displacements for each code, damage situations of members are classified according to structural performance limits as follows: immediate occupancy, life safety and collapse prevention. Due to the symmetrical storey plan of

Table 5. Member damage ratios for 3 storey building

Members	Storey	Damage situation	TEC-2007			FEMA-356			FEMA-440		
			IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	Number	-	10	2	-	12	-	-	12	-
		[%]	-	83	17	-	100	-	-	100	-
	2	Number	8	4	-	10	2	-	10	2	-
		[%]	67	33	-	83	17	-	83	17	-
	3	Number	12	-	-	12	-	-	12	-	-
		[%]	100	-	-	100	-	-	100	-	-
Columns	1	Number	4	12	-	8	8	-	8	8	-
		[%]	25	75	-	50	50	-	50	50	-
	2	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	3	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-

IO - immediate occupancy; LS - life safety; CP - collapse prevention

Table 6. Member damage ratios for 5 storey building

Members	Storey	Damage situation	TEC-2007			FEMA-356			FEMA-440		
			IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	Number	-	6	6	-	8	4	-	8	4
		[%]	-	50	50	-	67	33	-	67	33
	2	Number	4	6	2	8	4	-	10	2	-
		[%]	33	50	17	67	33	-	83	17	-
	3	Number	10	2	-	12	-	-	12	-	-
		[%]	83	17	-	100	-	-	100	-	-
	4	Number	12	-	-	12	-	-	12	-	-
		[%]	100	-	-	100	-	-	100	-	-
	5	Number	12	-	-	12	-	-	12	-	-
		[%]	100	-	-	100	-	-	100	-	-
Columns	1	Number	-	8	8	-	12	4	-	14	2
		[%]	-	50	50	-	75	25	-	87	13
	2	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	3	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	4	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	5	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-

Table 7. Member damage ratios for 7 storey building

Members	Storey	Damage situation	TEC-2007			FEMA-356			FEMA-440		
			IO	LS	CP	IO	LS	CP	IO	LS	CP
Beams	1	Number	-	2	10	-	4	8	-	6	6
		[%]	-	17	83	-	33	67	-	50	50
	2	Number	-	6	6	-	8	4	-	10	2
		[%]	-	50	50	-	67	33	-	83	17
	3	Number	8	4	-	10	2	-	10	2	-
		[%]	67	33	-	83	17	-	83	17	-
	4	Number	10	2	-	12	-	-	12	-	-
		[%]	83	17	-	100	-	-	100	-	-
	5	Number	12	-	-	12	-	-	12	-	-
		[%]	100	-	-	100	-	-	100	-	-
	6	Number	12	-	-	12	-	-	12	-	-
		[%]	100	-	-	100	-	-	100	-	-
	7	Number	12	-	-	12	-	-	12	-	-
		[%]	100	-	-	100	-	-	100	-	-
Columns	1	Number	-	-	16	-	4	12	-	6	10
		[%]	-	-	100	-	25	75	-	37	63
	2	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	3	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	4	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	5	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	6	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-
	7	Number	16	-	-	16	-	-	16	-	-
		[%]	100	-	-	100	-	-	100	-	-

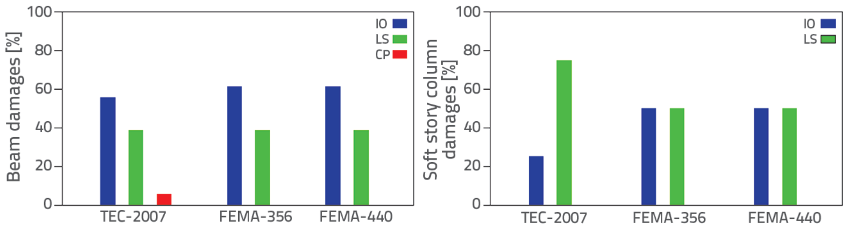

Figure 15. Damage ratios for 3 storey building

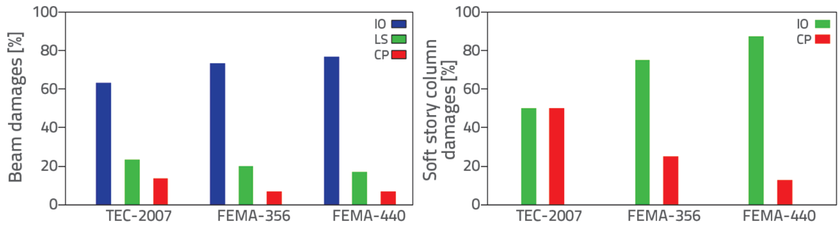

Figure 16. Damage ratios for 5 storey building

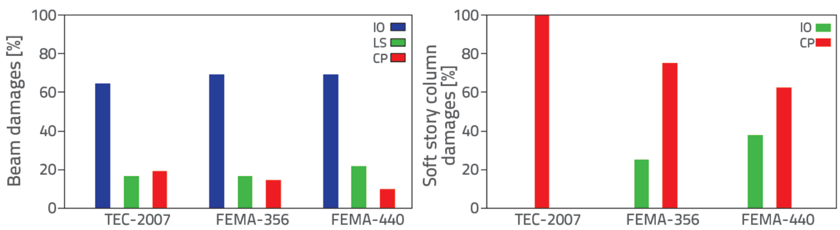

Figure 17. Damage ratios for 7 storey building

the structures in both directions, and the same section sizes, equal damage ratios are obtained for x and y directions. For this reason, the results are presented for only one direction at each storey according to TEC-2007, FEMA-356 and FEMA-440 in tables 5, 6, and 7.

The total damage ratios of structural members are calculated according to each code. Since the column damage occurs at the soft storey, the results are presented for beams and soft storey columns in figures 15, 16, and 17.

Since the storey drift performance of structures directly affects the damage situation, the storey drift ratios for the 3, 5, and 7 storey buildings are calculated at target displacements according to codes, as shown in Figure 18. It can be noticed that the biggest values are obtained on ground floors, and that the TEC-2007 gives the most conservative results when compared to other codes.

6. Conclusions and suggestions

Structural damage and collapse events are known to cause important losses. For this reason, performance based design and evaluation procedures have been developed in response to recent large scale earthquakes. In this respect, many researchers focus their studies on seismic performance of existing buildings, and on the strengthening of buildings exhibiting poor seismic performance. Recent codes place a primary emphasis on the definition of linear and non-linear performance evaluation techniques.

Non-linear analysis of structural performance can nowadays be performed by both dynamic and static procedures. These procedures are considered to be more reliable since more data about properties of material structural systems are required. However, dynamic solutions require much more time compared to static solutions. Therefore, current studies have mostly focused on methods involving static analysis. These methods are generally based on evaluation of the base shear and displacement relationship in terms of material and geometry changes.

The seismic performance of structures can be determined more realistically using displacement-based methods. Displacement-based methods rely on the relationship between the displacement demand and the lateral force carrying capacity of structures for a specific ground motion. In these methods, the displacement demand is calculated numerically. The pushover analysis is a simplified non-linear static method in which incremental seismic loads are applied until the plastic collapse mechanism is reached. The lumped plasticity approach is adopted and an inelastic behaviour is determined by plastic hinges at two ends of structural members. The capacity curve is prepared after the pushover analysis in order to determine damage situations and deformation demands for structural members.

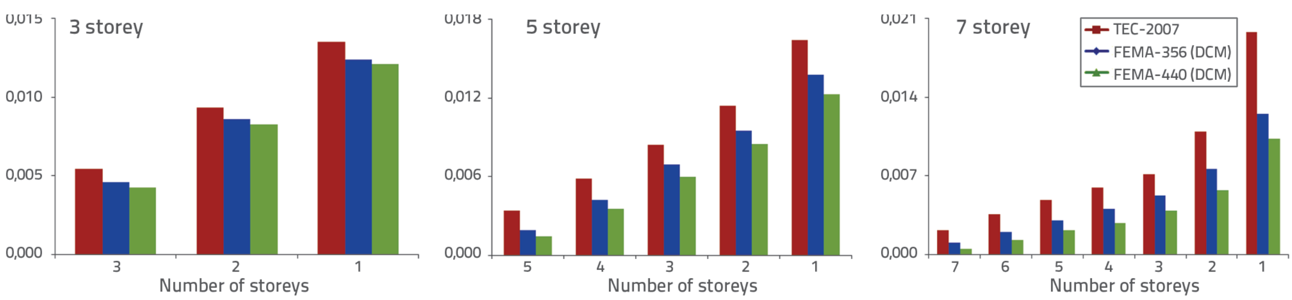

Figure 18. Storey drift ratios according to codes

A soft storey is one of important irregularities causing structural damage and losses. Main reasons for this irregularity are the stores designed for commercial purposes, which have higher storey heights and are devoid of brick walls. As behaviour of soft storeys is different from that exhibited by other storeys, and as bigger displacements are observed in soft storey columns, these buildings are highly susceptible to sudden collapse during an earthquake. Researchers have invested significant efforts to understand the behaviour of soft storeys under seismic action, which causes disproportionate lateral stresses and severe damage.

Non linear static analyses of the existing 3, 5, and 7 storey reinforced concrete buildings having soft storey irregularities are performed in this paper according to FEMA-356 (DCM), FEMA-440 (DCM), and TEC-2007. Material properties, storey plans, and section sizes, are assumed to be constant for these structures. The structures are assumed to represent typical residential low-rise, mid-rise, and high-rise buildings. Capacity curves of the buildings are obtained after non-linear pushover analyses. Each building is then pushed to target displacements for each code. Modal properties with damage ratios of structural members and storey drifts are determined for the buildings at target displacements.

After evaluation of structural performance results for the three codes, it was established that more conservative results are obtained by TEC-2007 compared to FEMA-356 and FEMA-440. As could have been expected, damage ratios reach the highest values in the first floor for each code. Light damage levels are observed in the upper floors of the structures. Damage situations of structural members increase in severity in direct proportion to the total height of the buildings. More pronounced damage and bigger storey drift ratios are registered at the 7-storey structure. While all soft storey columns reach collapse prevention level according to TEC-2007, 75 % and 63 % of them get this damage level according to FEMA-356 and FEMA-440, respectively, for the 7-storey structure. The maximum beam damage is also observed according to TEC-2007. 19.0 %, 14.3 % and 9.5 % of the beams remain at the collapse prevention level according to TEC-2007, FEMA-356 and FEMA-440 for the 7-storey structure.

Based on the results obtained for the three buildings, it can be stated that soft storey irregularities may cause heavy damage, especially in case of taller buildings. Soft storey columns can ensure the life safety level for the 3-storey structure only. On the other hand, the collapse damage situation is observed at more than one floor, especially for the beams of the 7-storey structure, according to each code. Finally, this study can be improved by further analysis of different types of structures, with strengthening techniques based on non-linear methods according to various codes.

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