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Seismic Assessment of RC Buildings According to FEMA 356

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Abstract: This study investigates the seismic performance and vulnerability analysis of a 7-storey code-conforming with International Building Code 'IBC 2009', Reinforced Concrete (RC) building with Special Moment Resisting Frame (SMRF). One of the most important tasks in performance-based evaluation is the capacity assessment of current buildings in conformance with the present codes. Accurate assessment of the actual lateral load resistance and the potential modes of failure is an essential tool to improve the current building performance to the existing ductile design level recommended by existing codes and also, determination and design of the proper retrofit and rehabilitation system. In this study, a non-linear static (pushover) analysis was carried out for the building incorporating SAP 2000 and the ultimate capacity of the building based on the FEMA 356 guideline was established. Evaluation of the yield, plastic and ultimate rotation capacities of beams and columns was done through the application of an analytical procedure. The generated capacity curves were used to assess the performance of the building and life safety performance level (C-3) was used to describe the limiting damage condition which may be considered satisfactory for a building under Basic Safety Earthquake 1(BSE-1).

Key words: Special moment resisting frame, pushover analysis, reinforced concrete structures, eartquake, non-linear static

INTRODUCTION

Earthquakes are considered to be the most hazardous phenomenon which could incur countless damages to engineered structures. Due to the unpredictable and random nature of earthquakes, application of improved engineering tools in order to analyze the structures subjected to these forces is required. A study of earthquake damage in retrospect and present indicates the vulnerability of structures to severe damage and/or collapse resulting from moderate to strong ground motion. Severe damages to engineered structures including buildings, industrial and port facilities and bridges along with great economic losses could stem from an earthquake with a moderate magnitude. Hence, reverse seismic evaluation is of great importance and development of more reliable seismic standards and codal provisions which could facilitate the analysis and design of new engineering facilities, compared to those currently in use is one possible means of achieving this objective. Achievement of an appropriate solution in terms of capacity and performance in order to evaluate the seismic performance of buildings is necessary for earthquake resistant design. In case of severe earthquakes, it is believed by the engineering community responsible for seismic design process that the conventional elastic design and analysis procedures are incapable of considering different aspects of the seismic performance of structures. Furthermore, incorporation of inelastic time-history analysis which is more accurate and powerful would be impractical due to its expensive computational nature. One of the most promising tools in terms of evaluation of seismic performance of current structures is the non-linear static analysis (pushover analysis). The seismic capacity of the structural system and its relevant components could be estimated through the use of pushover analysis. The material characteristics and dimensions of the members are studied to calculate the seismic capacity of the structural system and its relevant components which is determined through the application of pushover analysis. Hence, this method is incorporated to evaluate the seismic performance of the structure and quantify characteristics, such as strength, drift and deformation capacity when subjected to design ground motion. In order to check the performance of the building against the demand of present International Building Code 'IBC 2009' (INC, 2009) non-linear static analyses were carried out to generate the capacity curve using finite element software, SAP 2000 (CSI, 2012) and analyses have been performed for default hinge case. The deformation capacity of reinforced concrete components including beams and columns were modelled in the form

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of plastic hinges incorporating Federal Emergency Management Agency (FEMA, 2000) provisions for the default case.

MATERIALS AND METHODS

Pushover analysis methodology: There has been an extensive development in the fields of non-linear static analysis over the past few years and this has been attributed to the capability of new computers in supporting state of the art computer programs. A simple indication of pushover analysis is demonstrated in Fig. 1. The procedure involves monitoring progressive yielding of the structure and drawing the relevant capacity curve through the use of non-linear static analysis of a given structure. A lateral loading shape is hired to push the structure in conformance with the mode shape of the pre-yielding building to the level of specific target displacements. However, vertical earthquake loading is neglected. Consequently, the base shear-roof displacement plot is termed the capacity curve (Fig. 1). In order to calculate the strength and deformation demands to compare with available capacities, the internal forces and deformations are calculated to meet the target displacement levels (Fig. 2).

Displacement coefficient method from FEMA 356/ASCE 41: According to ASCE 41 (Members, 2007) and its pre-standard FEMA (2000), the fundamental method to calculate the displacement in non-linear static procedure is the displacement coefficient method which is known as the coefficient method in FEMA (2000). The maximum global displacement simply termed as the target displacement is estimated through, the use of the displacement coefficient method through the modification of linear elastic response of an equivalent SDOF system. A series of coefficients namely C_0 through C_3 are multiplied by the spectral displacements of the SDOF system to achieve the previous mentioned objective. Calculation of target displacement is indicated in Fig. 3. By applying a graphical procedure incorporating an idealized force-deformation curve (i.e., pushover curve), the effective period, T_e is calculated from the initial period, T_i which forms a relationship between base shear and roof displacement as an indication of the stiffness loss once the inelastic behavior of the system commences. The linear stiffness of the equivalent SDOF system is represented by means of the effective period, T_e. Determination of the equivalent spectral acceleration, S_a of a SDOF system is done through the use of the effective period and incorporation of elastic response spectrum. A damping ratio of 5% seems to be

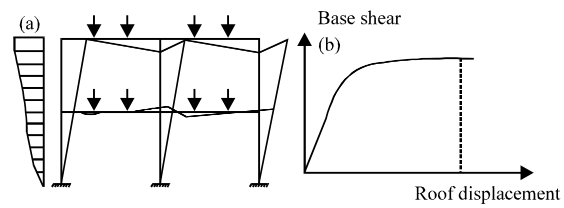


Fig. 1: Illustration of pushover analysis

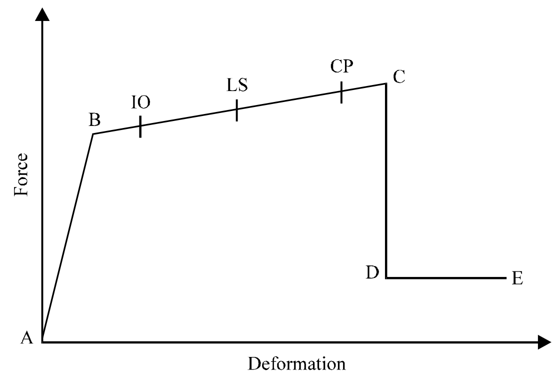


Fig. 2: Typical load-deformation relation and target performance levels

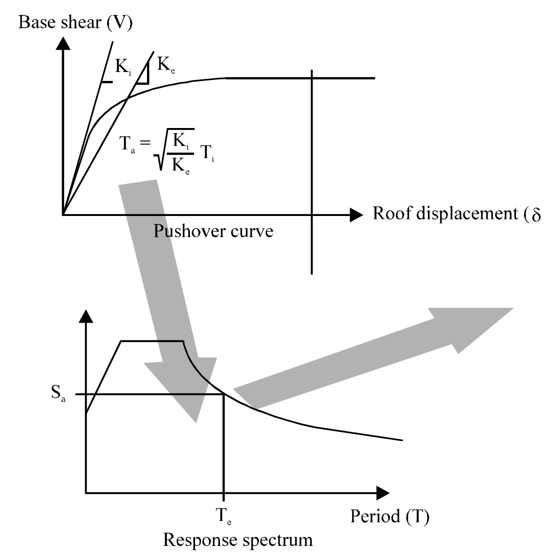


Fig. 3: Sehematic illustration of the process of estimating target displacement. $\delta_t = C_D\, C_1\, C_2\, C_3\, S_a\, T_o^2/4\pi^2 g$ = Target displacement: C_o = Converts SDOF spectral displacement to MDOF roof displacement (elastic); C_1 = Expected maximum inelastic displacement devided by elastic displacement; C_2 = Effect of pinched hysteretic shape, stiffness degradation and strength deterioration; C_3 = Increased displacement due to dynamic P-Δ effects

appropriate for the structure within the elastic range in this procedure. Next, the spectral acceleration is incorporated to determine the peak elastic spectral displacement using Eq. 1:

$$S_d = \frac{T_{eff}^2}{4\pi^2} S_a \tag{1}$$

The four coefficients used by the displacement coefficient method are used in 2 steps: First, to convert the peak elastic spectral displacement to elastic displacement at the roof. Second, to convert elastic spectral displacement to inelastic displacement at the roof.

Code provisions for plastic hinge rotation capacities of RC members: The first step in non-linear procedures of FEMA (2000) would be to define a relationship between load and deformation. The curve in Fig. 4 indicates such a relationship. First, it is clear that the unloaded condition is represented by point A. Then, the nominal steel yield strength is demonstrated by point B. Next, almost 0-10% of the initial slope (line AB) is assumed for the line BC. Next, the resistance of point C would be equal to that of nominal strength. After that the initial failure of the member is represented by line CD. The initial failure includes shear failure following initial yield, spalling of concrete and fracture of the bending reinforcement. After that the residual strength of the member is represented by line DE which could either be non-zero or zero in different circumstances. Finally, the deformation limit is represented by point E. Yet, the limiting deformation is defined by the initial failure at C and therefore, the deformation of point E would be the same as point C in addition to a zero resistance. The hinge rotation behavior of RC members is defined through points A-E in conformance with FEMA. The definition of acceptance criteria for the hinge is done through the use of 3 more points namely Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The purpose of this evaluation is to determine whether the building's structural system meets the Life-Safety Building Performance Level (3-C) according to the FEMA 356 (FEMA, 2000).

Lateral load patterns distribution: The relative amounts of shears, moments and deformations of the structure are determined through the distribution of lateral inertial forces. The structure will eventually yield and its stiffness characteristics will change due to the consistent variation in distributed lateral inertial forces. The extremes of this distribution will be influenced by the intensity of the earthquake shaking and the degree of non-linear response

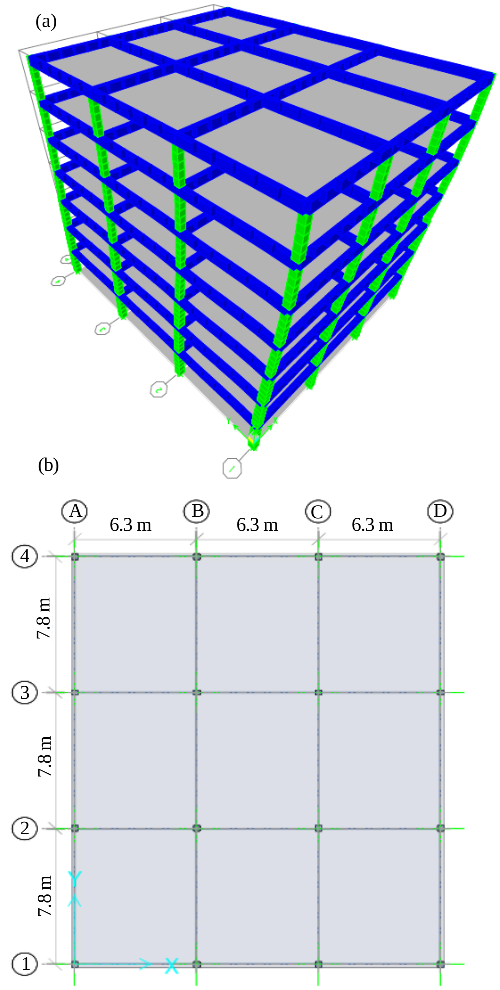


Fig. 4: Relationship between load and deformation

of the relevant structure. The range of the design actions may be bound by the use of >1 lateral load pattern. The three following patterns were selected for this study which are as follows:

- The first pattern has a triangular linear static vertical distribution proportional to the values of Cvx. Application of this load pattern is allowed once proportion of the total mass participating in the fundamental mode exceeds 75% in the preferred direction and the uniform distribution is incorporated as well
- A vertical distribution proportional to the shape of the fundamental mode in the direction under consideration. Application of this load pattern is allowed once proportion of the total mass participating in the fundamental mode exceeds 75% in the preferred direction
- A uniform distribution consisting of lateral forces at each level proportional to the total mass at each level

Table 1: Factors to translate lower bound material properties to expected strength

Material property (QCL)	Factors
Concrete compressive strength	1.50
Reinforcing steel tensile and yield strength	1.25

Table 2: Material properties

Concrete material properties	Values
ρ (kg m^{-3})	245
γ (kg fm^{-3})	2400
τ	0.2
E (MPa)	1.5×0.043√fc
F_y (MPa)	420
F_u (MPa)	300
f≠c (MPa)	21

Component gravity loads for load combinations: The following component gravity forces, QG are incorporated to combine with the previous mentioned seismic loads (Eq. 2):

$$QG = 1.1(QD + QL) \qquad (2)$$

Expected strength material properties: Once the evaluation of the behavior of deformation-controlled actions is preferred, the use of expected strength, QCE would be beneficial. The statistical mean value of yield strengths, Qy, belonging to a population of similar components is also termed QCE and considers the strain hardening and plastic section development. Application of the lower bound component strength, QCL is allowed once the evaluation of the behavior of force-controlled actions is in progress. QCL is defined as the statistical mean minus one standard deviation of the yield strengths, Qy for a population of similar components. Incorporation of expected or lower-bound materials properties, respectively are allowed once determination of expected or lower-bound strengths of components by means of calculation are favored. Table 1 illustrates the expected strength material properties factor.

Configuration and analytical modeling of progressive collapse design: In order to demonstrate, the process of design against progressive collapse, a typical reinforced concrete structure has been taken into account. The occupancy of the structure is <500 people and therefore is categorized as Occupancy Category II per UFC3-301-01 (UFC, 2013). The structure under investigation is a seven-story SMRF and its intended function would be for office use.

Modeling assumptions:

- Systems of gravity: Two way slab
- Vertical support: Columns
- Lateral: Special Moment Resistant Frames (SMRF)
- Foundation: Shallow footings

Table 3: Gravity loads

Load types	Weight (kN m^{-2})
Slab	0.17×24 = 4.08
Flooring dead load	1.9
Live load	5
Total load	10

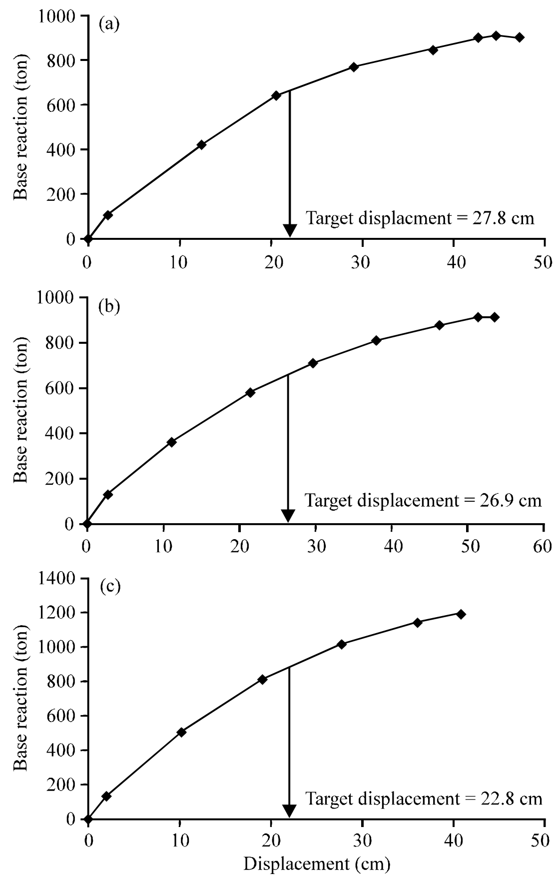


Fig. 5: Push over curve for 3 different lateral load pattern: a) Tringular static pattren; b) Shape mode pattren; c) Uniform distribution patrren

- Wind load (W) was calculated per ASCE 7-05 incorporating 110 mph with exposure B and importance factor equal to 1.0 (ASCE, 2006)
- Earthquake load (E) is assumed based on IBC 2009 (INC, 2009) using response modification factor R = 10, Site class = B, Occupancy importance = 1 and design spectral accelerations (S_{DS}&S_{D1}) = 0.75 and 0.39 based on this data the Seismic Design Category (SDC) determine D

Table 2 and 3 demonstrate the concrete material properties and loading assumptions, respectively. The plan view of the proposed RC structure, as well as removal column location is indicated in Fig. 5.

RESULTS AND DISCUSSION

Analysis and design of model structure

Design requirement by ACI 318-08/IBC 2009: The column and beam dimensions along with the details of arrangement of longitudinal reinforcement are shown in Table 4.

Pushover analysis by FEMA 356: After designing and detailing the reinforced concrete frame structure, a non-linear pushover analysis was carried out for evaluating the structural seismic response. Incorporation of gravity loads and a representative lateral load pattern is an essential step of the pushover analysis. Application of the lateral loads were done in a monotonic manner through a step-by-step nonlinear static analysis. By the time ground shaping occurred, the applied forces in the x direction were accelerations which represented the forces that the structure was about to bear. There was a sequential yielding of elements in the event of incrementally increasing loads. Consequently, the structure experienced s stiffness change at each event. Based on the specifications of FEMA (2000) given in SAP 2000 (CSI, 2012), the capacity curves (Base shear vs. roof displacement capacities) were generated for Default (DF) hinge properties. Figure 5 demonstrates the capacity curve for three different lateral load distribution patterns as well as the target displacement.

Assessment of plastic hinge formation: The formation of plastic hinges resulting from the 3 different lateral load distribution patterns have been achieved in conformance with FEMA (2000) rules (Table 5-7). Determination of the position and plastic rotation of hinges in beams and columns are revealed through the formation of plastic hinges in beams and columns. Also, determination of the hinges which have reached 1 of the 3 FEMA (2000) limit states, namely; IO, LS and CP is done through the formation of plastic hinges.

Table 4: Frames elements dimension

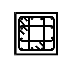
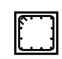
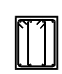
Element types	Levels	Locations	Dimension (mm)	Longitudinal reinforcement	Transvers reinforcement	Shapes
Column	1 and 2	Interior	650×650	24 Ø 28	Ø 10 @ 150 mm 4 each direction	
		Long and short side	650×650	16 Ø 28	Ø 10 @ 150 mm 3 each direction	
		Corner	500×500	16 Ø 22	Ø 10 @ 150 mm 3 each direction	
	3 and 4	Interior	500×500	12 Ø 32	Ø 10 @ 200 mm 4 each direction	
		Long and short side	500×500	12 Ø 28	Ø 10 @ 200 mm 4 each direction	
		Corner	500×500	12 Ø 22	Ø 10 @ 200 mm 4 each direction	
	5 and 7	Interior	450×450	12 Ø 25	Ø 10 @ 150 mm 2 Each direction	
		Long and short side	450×450	12 Ø 20	Ø 10 @ 150 mm 2 each direction	
		Corner	450×450	12 Ø 18	Ø 10 @ 150 mm 2 each direction	
Beam	1 and 2	Transverse and longitudinal (interior)	300×600	Top = 5 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
		Transverse and longitudinal (exterior)	300×600	Top = 4 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
	3 and 4	Transverse and longitudinal (interior)	300×600	Top = 6 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
		Transverse and longitudinal (exterior)	300×600	Top = 5 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
	5 and 7	Transverse and longitudinal (interior)	300×600	Top = 5 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	
		Transverse and longitudinal (exterior)	300×600	Top = 4 Ø 20 Bot = 3 Ø 20	Ø 10 @ 150 mm	

Table 5: Salient features of hinge formation for triangular static pattern

Steps	Displacement (cm)	Base force (ton)	A to B	B to IO	IO to LS	LS to CP
0	0.00	0.00	560	0	0	0
1	2.08	107.85	556	4	0	0
2	12.33	424.00	408	152	0	0
3	20.48	640.08	380	152	28	0
4	29.02	775.89	298	146	116	0
5	37.59	859.71	262	108	184	6
6	42.61	901.20	254	110	160	14
7	44.61	911.60	254	106	146	30

Table 6: Salient features of hinge formation for shape mode-1 pattern

Steps	Displacement (cm)	Base force (ton)	A to B	B to IO	IO to LS	LS to CP
0	0.00	0	560	0	0	0
1	2.67	126.31	536	24	0	0
2	11.07	358.88	432	128	0	0
3	21.25	579.67	404	106	50	0
4	29.46	712.27	348	108	98	6
5	37.81	805.35	300	102	120	36
6	45.98	875.51	284	78	134	48
7	50.65	907.09	274	84	112	50

Table 7: Salient features of hinge formation for uniform distribution pattern

Steps	Displacement (cm)	Base force (ton)	A to B	B to IO	IO to LS	LS to CP
0	0.00	0.00	560	0	0	0
1	1.74	128.31	556	4	0	0
2	10.08	511.46	418	142	0	0
3	18.65	811.46	388	142	30	0
4	26.89	1017.01	340	124	96	0
5	35.11	1145.99	304	116	108	30
6	39.84	1193.89	292	108	94	46
7	39.84	1193.89	292	108	94	46

Table 8: Damage description and drift limit

Description	Collapse prevention	Life safety	Immediate occupancy
Damage description	Extensive cracking and formation of hinges in ductile elements Severe damage in short columns	Extensive damage to beams. Spalling of cover and shear cracking <1/8" for ductile columns	Minor hairline cracking. Limited yielding possible at a few locations
Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

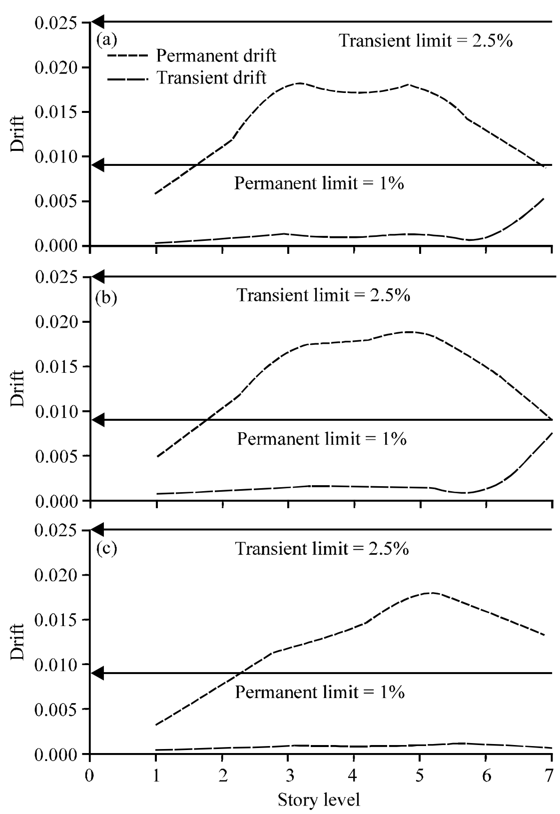


Fig. 6: The drift value versus story level for the 3 diffrent lateral load pattern: a) Tringular static pattren; b) Shape mode pattren; c) Uniform distribution patrren

Assessment of lateral drift: In order to illustrate, the overall structural response relevant to various defined performance levels, drift values treated as typical values are prescribed by FEMA (2000). For this purpose, Fig. 6 illustrate the drift value versus story level for 3 different lateral load pattern. Yet, such drift values are not treated as drift limit requirements according to the standard and cannot be replaced by the member-level analysis of which plastic rotation is assessed. Although, the drift limits recommended by FEMA (2000) are considered, as guidelines for the global analysis of a structure, it is advised to perform a more extensive member analysis to check the progress of plastic hinge development. Although, the relationship between drift and damage cannot be taken for granted, complete prediction of damage by incorporating drift alone is not possible due to the existence of complex relationships of other building characteristics. Furthermore, the creation of most established damage-drift relationships is a function of a limited amount of data. Hence for the purpose of better refining these guidelines, a more comprehensive and extensive study would be beneficial. Table 8 summarizes the structural performance levels and damage to vertical elements found in FEMA (2000). Notice that transient drift occurs when the first hinge appears in the structure during the pushover analysis. Also, permanent drift is associated with the target displacement.

CONCLUSION

The seismic performance of Special Resistant Moment Frame (SMRF) designed, according to

International Building Code 'IBC 2009' (INC, 2009) was investigated using the pushover analysis. The pushover analysis is a relatively simple method to explore the nonlinear behavior of buildings. The behavior of the properly detailed reinforced concrete frame building is adequate as indicated by the intersection of the demand and capacity curves and the distribution of hinges in the beams and the columns. Most of the hinges were developed in the beams and few in the columns but with limited damage. With an overall perspective view, the structure had a significantly high capacity to resist Basic Safety Earthquake 1 (BSE-1). In other words at target displacement for all three patterns, almost all hinges remain at immediate occupancy level. It can be inferred that the structure has the ability to resist collapse prevention around 45 cm at least two times bigger than target displacement. It is interesting to mention that the resemblance between all three lateral load distribution patterns with ultimate base shear reaction around 1000 ton was remarkable. Although, the structure demonstrated a good performance in terms of structural elements at target displacement, the lateral drift was not satisfactory at this performance level. It was concluded that the structural-drift ratio that occurred in a permanent manner was bigger than the limit of structural-drift ratio as required by FEMA (2000) for life safety which was 1.0% and the lateral drift for all three patterns was around two times bigger than a llowable amount. Consequently, non-structural elements could be damaged.

NOMENCLATURE

Cvx = The base shear of static analysis
QD = The dead-load (action)
QL = Effective live load (action) = 25% of the unreduced design live load but not less than the actual live load
ρ = The mass density or density of material
γ = The weight per unit volume
τ = The Poisson's ratio
F_y = The reinforcement yield stress
F_u = The reinforcement ultimate stress
E = The elastic modulus of the reinforcement
f≠c = The specified concert compressive strength

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