Seismic Evaluation of Reinforced Concrete Frames Using Pushover Analysis

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Abstract

Ten stories–five bays reinforced concrete frame (two dimensional beams and columns system) subjected to seismic hazard of the Mosul city/Iraq is analyzed. Plastic hinge is used to represent the failure mode in the beams and columns when the member yields. The pushover analysis is performed on the present building frame using SAP2000 software (V.14) to verify code's underlying intent of Life Safety performance under seismic effects. The principles of *Performance Based Seismic Engineering* are used to govern the present analysis, where inelastic structural analysis is combined with the seismic hazard to calculate expected seismic performance of a structure. Base shear versus tip displacement curve of the structure, called pushover curve, is an essential outcomes of pushover analysis for two actions of the plastic hinge behavior, force-controlled (brittle) and deformation-controlled (ductile) actions. Lateral deformations at the performance point proved that the building is capable of sustaining certain level of seismic load. The building clearly behaves like the strong column-weak beam mechanism, although the formed hinges are in the dangerous level according to Applied Technology Council (ATC-40) categories of structural performance and they need to be strengthened.

Keywords: Building frame, Nonlinear response spectrum, Pushover analysis, Reinforced concrete, Seismic performance.

التقييم الزلزالي للهياكل الخرسانية المسلحة باستخدام التحليل السكوني اللا خطي

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الخلاصة

في هذه الدراسة تم تحليل بناية مشيدة من الخرسانة المسلحة ومكونة من عشر طوابق (نظام أعمدة وجسور ثنائية البعد) تقع تحت تأثير المخاطر الزلزالية لمدينة الموصل / العراق. استخدم المفصل اللدن لتمثيل وضع الفشل في الجسور والأعمدة عند خضوع العضو الإنشائي للهيكل الخرساني تحت تأثير تلك الأحمال. أجري التحليل السكوني اللاخطي باستخدام برنامج (V.14) SAP2000 للتحقق من الغرض الأساسي لأداء سلامة الحياة تحت تأثير القوى الزلزالية. أستُخدِمت مبادئ الأداء الزلزالي لتحكم هذا التحليل، حيث يتم الجمع بين التحليل الإنشائي اللامرن مع المخاطر الزلزالية لحساب الأداء الزلزالي المتوقع للهيكل الإنشائي. تعطي طريقة التحليل الحالية بيانات عن قوى القص القاعدية إزاء إزاحة الطابق الأخير للهيكل وهي تعتبر من ابرز البيانات الأساسية لهذا التحليل، ويتم إجراء التحليل بافتراض سلوكين مختلفين لتصرف المفصل اللدن أثناء التحليل (فشل مطيلي عادة يكون التشوه هو المسيطر على التصرف اللامرن بينما في الفشل القصفي تكون القوة هي المسيطر على التصرف اللا مرن). أن التشوهات أو الإزاحات الجانبية عند نقطة الأداء أثبتت أن هذا المبنى قادر على الحفاظ على مستوى معين من الحمولة الزلزالية. و من الواضح أيضاً أن المنشأ الحالي يتصرف بوضوح بشكل مماثل لآلية العتب الضعيف - العمود القوي، على الرغم من أن كل المفاصل اللدنة هي في مستوى خطر وفقا للفئات المُصنّفَة للمدونة ATC -40 والذي يعتبر الحاكم للأداء الهيكلي وأنها بحاجة إلى تقوية نتيجة الأضرار.

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Introduction

Design of civil engineering structures is typically based on prescriptive methods of building codes. Normally in the static case, the loads on these structures are low and result in elastic structural behavior. However, under a strong seismic event, a structure may actually be subjected to forces beyond its elastic limit. Although building codes can provide a reliable indication of actual performance of individual structural elements, it is out of their scope to describe the expected performance of a designed structure as a whole, under large forces. With the availability of fast computers, so-called *Performance-Based Seismic Engineering* (PBSE), where inelastic structural analysis is combined with seismic hazard assessment to calculate expected seismic performance of a structure, has become increasingly feasible [1,2]. Nonlinear time history analysis is a possible method to calculate structural response under a strong seismic event. However, due to the large amount of data generated in such analysis, it is not considered practical and (PBSE) usually involves nonlinear static analysis, also known as pushover analysis. Furthermore, modern building codes such as *International Building Code* (IBC 2006) and *Federal Emergency Management Agency* (FEMA 356-2000) favor more accurate procedures (as pushover analysis) over traditional linear-elastic methods for a more thorough analysis. Recently many researchers decide how to improve, optimize and control the performance-based seismic design of structures. BAI JiuLin and OU JinPing [3] combined the failure path and the probability of occurrence for plastic hinges to strengthen the columns and beams, then considered it is a feasible way to improve the seismic capacity of the frame structure. Vijayakumar A. and Venkatesh Babu D. L. [4] analyzed three existing buildings using pushover analysis, these buildings were previously designed according to Indian standards, they concluded that these buildings were inadequate in seismic performance, and they suggested before rehabilitation work, it was necessary to check the ultimate capacity of the these buildings to determine the strengthening volume.

In the present study the presumed building is evaluated for inelastic response of the lateral static loads, equivalent to expected seismic loads, directly applied to the joints of building frame.

Seismic Loads on The Frame

1. Base shear force

The ***U**niform **B**uilding **C**ode* (UBC1997) [5] requires that the “design base shear”, ***V***, is to be evaluated from the following formula:

$$V = (ZIKCS)W \qquad (1)$$

where:

K = Inelastic behavior factor of the structure given in Table 1.
W = The total seismic weight of the structure.
S = Site coefficient for soil characteristics given in Table 2.
Z = Seismic zone factor that depends on effective peak ground accelerations in the specified area given in Table 3.
I = Importance factor. Classifying buildings according to importance:

- Special occupancy structures, standard occupancy structures (**I =1.5**).
- The building must remain functioning in a catastrophe (**I =1.25**).
- Hospitals, communication centers, fire and police stations (**I =1.0**).

C = Stiffness factor of the structure depends on the fundamental period of vibration (seconds). This factor is approximately calculated from the following relation [5,6] and not more than or equal to (**0.12**):

$$C = \frac{1}{15 \times \sqrt{T}} \leq 0.12 \tag{2}$$

where ***T*** represents the building's fundamental period of vibration in seconds. There are two relations in UBC are used to estimate ***T***, the more accurate one is:

Table (1): Inelastic behavior factor of the structure (*K*)[5].

Type of structure	***K*** factor
Special structures : Chimney, TV Towers, ….etc.	**2.0**
RC shear wall building frames.	**1.3**
RC beam-column building frames systems with or without connected shear walls according to the resistance of this system, the resistance must not be less than : 25% of the total horizontal loads applied to the structure. 50% of the total horizontal loads applied to the structure.	 **1.0** **0.8**
Elevated water storage tanks or other the same of this construction (carried on 4 columns) stiff connection in horizontal plane.	**2.5**
The structures that are not mentioned above.	**1.0**

Table (2): Site coefficient for soil characteristics (*S*)[5].

Soil Profile	**Description**	**Coefficient *S***
S1	A soil profile with either: • Rock of any characteristic, whether shady or crystalline, which has a shear wave velocity greater than 750 m/sec. • Rigid soil conditions where the soil depth is less than 60 meters, and the soil types over the rock are stable deposits of sand, gravel or stiff clay.	**1.0**
S2	A soil profile with deep non-cohesive conditions or rigid clay, where the soil depth exceeds 60 meters, and the soil types over the rock is stable deposits of sand, gravel or stiff clay.	**1.2**
S3	A soil profile containing form 6 to 12 meters of soft or medium-stiff clay with or without intermediate non-cohesive soils layer.	**1.5**
S4	A soil profile for a shear wave velocity less than 150m/sec which contains more than 12 meters of soft clay or limos.	**2.0**

Table (3): Seismic zone factor (*Z*) [5].

Zone	**0**	**1**	**2A**	**2B**	**3**	**4**
Z factor	**0**	**0.075**	**0.15**	**0.2**	**0.3**	**0.4**

$$T = \frac{0.09 \times h_n}{\sqrt{D}} \tag{3}$$

In all cases the product of coefficients (***KC***) is restricted to (**0.06-0.25**) [5].

2. Equivalent lateral static loads

The base shear force is distributed as a lateral force, which effects on the joint, at each level of the frame so that:

$$V = F_t + \sum_{i=1}^{n} F_i \tag{4}$$

The concentrated force (F_t) at the top of building frame is calculated by:

$$\begin{aligned} F_t &= 0.07\, T.V \quad if\ T > 0.7\sec \\ F_t &= 0.0 \qquad\quad if\ T \le 0.7\sec \end{aligned} \tag{5}$$

The lateral forces applied on the stories, as shown in Figure (1), are calculated from the following form:

$$F_x = (V - F_t)\frac{W_y h_y}{\sum_{i=1}^{n} W_i h_i} \tag{6}$$

where:

V = Base shear force. h_y = Height at the y level of the frame.

F_x = Lateral force applied on the y level of the frame.

W_y = The total vertical loads (dead and **25%** live loads) concentrated at the y level.

n = Number of building stories. W_i = Weight of the story i.

h_n = Total height of the frame. *and* D = Width of the frame plan.

Pushover Analysis

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is incrementally increased in accordance with a certain predefined pattern. Static pushover analysis is an attempt by the structural engineering profession to evaluate the real strength of the structure and it promises to be a useful and effective tool for performance based design.

The ATC-40 and FEMA-356 [7,8] documents have developed modeling parameters, acceptance criteria and procedures of pushover analysis. These documents also describe the actions followed to determine the yielding of frame member during the analysis. Two actions as shown in Figure (2) are used to govern the inelastic behavior of the member during the pushover analysis, that are *deformation-controlled* (ductile action) or *force-controlled* (brittle action) [7,8].

As shown in Figure (2*i*), five points labeled *A*, *B*, *C*, *D*, and *E* are used to define the force-deflection behavior of the hinge. In this figure, the deformations are expressed directly using terms such as strain, curvature, rotation, or elongation.

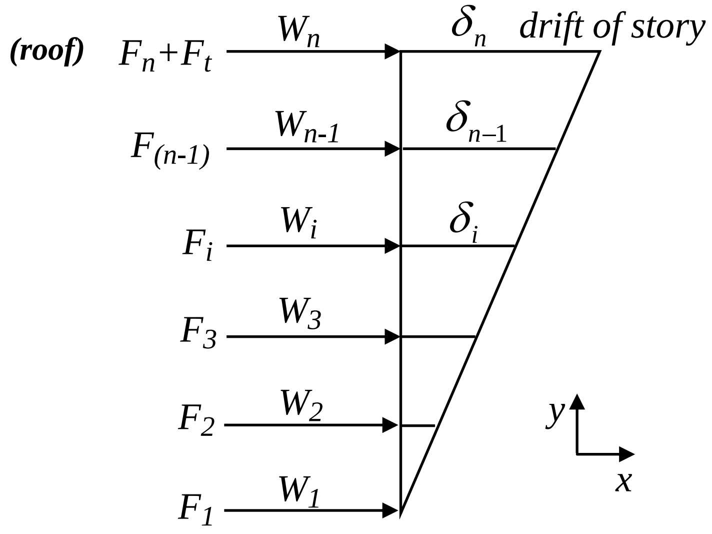
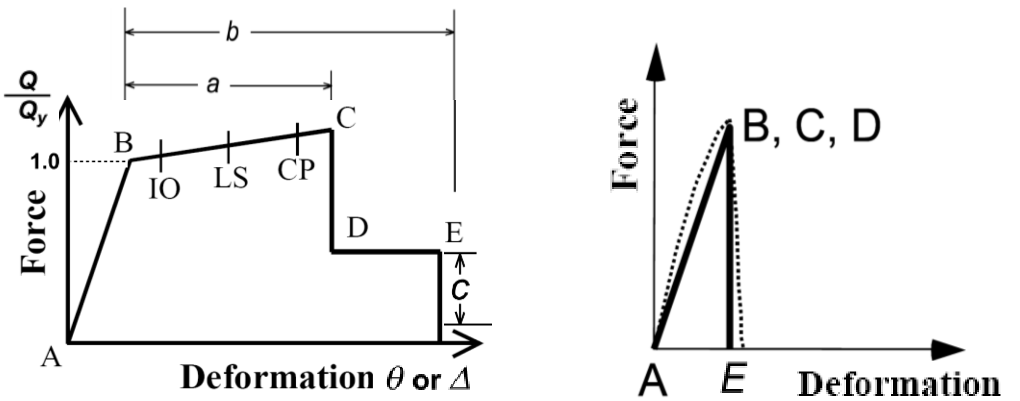


Figure (1): Distribution of lateral static forces equivalent to seismic loads.

The parameters (*a* and *b*) shall refer to those portions of the deformation that occur after yield (from *B* to *D* on the curve); that is, the plastic deformation. The parameter (*c*) is the reduced resistance after the sudden reduction from *C* to *D*. Parameters (*a*, *b*, and *c*) are defined numerically in various tables in reference [9].

Alternatively, it shall be permitted to determine the parameters *a*, *b*, and *c* directly by analytical procedures justified by experimental evidence [7,8]. The slope from point *B* to *C*, ignoring effects of gravity loads acting through lateral displacements, shall be taken between zero and 10% of the initial slope unless an alternate slope is justified by experiment or analysis.



(i)Deformation-controlled option flexural failure

(ii)Force-controlled option shear failure

Figure (2): Schematic depictions illustrating inelastic idealized force-deformation relationships.

Tables (3) and (4) show the values of parameters (*a*, *b*, and *c*) for both beams and columns. Generally these parameters depend on the section properties such as steel ratio in the tension and compression fibers, balanced steel ratio for the section, design shear strength, design axial load, compressive strength of concrete, and cross section area.

Acceptance Criteria (*Performance Level*)

Three points labeled *IO*, *LS* and *CP* as referred in Figure (2*i*) are used to define the Acceptance Criteria or performance level for the plastic hinge formed near the joints (at the ends of beams and columns). *IO*, *LS* and *CP* stand for ***I****mmediate* ***O****ccupancy*, ***L****ife* ***S****afety* and ***C****ollapse* ***P****revention*, respectively. The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-273 documents. Tables (4) and (5) show the values of Acceptance Criteria for both beams and columns**,** whereas Table (6) describes the structural performance levels of the concrete frames [7,8].

Nonlinear Hinge Property

In the present study, the nonlinear hinge properties, as assigned in SAP2000 model [10], are calculated as described in the following:

1. Axial load-bending moment (P-M) interaction surface:

P-M interaction surface determines the load at which a reinforced concrete section of the beam or column becomes inelastic and forms a hinge. For a given section geometry, material and reinforcement, *P-M* interaction surface was calculated using SAP2000 section designer module according to *ACI code* (2002) [10].

The stress-strain curve for concrete suggested by Kent and Park [11] and stored in SAP2000 software is used to complete *P-M* interaction curves for the sections in the frame.

2. Moment-plastic rotation (M- θ_p) relation:

M-θ_p relation for a member section consists of plastic rotation and corresponding moments as ratio of yield moment .This relation affects the behavior of a section once a hinge forms there. All values needed to define M-θ_p relation may be obtained using Tables (4) and (5). Plastic hinge length required for this calculation was based on FEMA guidelines.

Numerical Application And Structural Capacity

Example 1

A five bays-ten stories regular frame in reinforced concrete is considered as a numerical case. The building frame consists of structural elements as follows:

1. (*450×450 mm*) square RC columns, reinforced with (*12 Ø 25 mm*), shear stirrups of (*Ø8 mm @ 200 mm c/c*).
2. (*300×450mm*) RC beams, reinforced with (*4Ø22mm*) as tensile and compression steel with shear stirrups of (*Ø10mm @ 200mm c/c*).
3. (*125 mm*) thickness of RC slab.

The concrete strength at 28-days is (f'_c= 25.0 N/mm^2) and the reinforcing steel used is high-yield-strength deformed bars, that is (f_y = 415 N/mm^2). The building frame consists of (4 m) bay width and (4 m) story height, with no structural and geometric irregularities and assumed to be located in (***Zone II***) with soil condition as "***medium***" type. Using the expressions for axial load-bending moment (P-M) interaction and moment-rotation relationship in the modeling of hinge behavior for the beams and columns [13]. Figure (3) shows the P-M interaction details for the beam hinges to be used in the model, the P-M interaction is constructed by the source files of SAP2000 software. Figure (4) shows the moment-rotation relation of tension hinge of the beam, which is constructed using the properties of RC sections and related formulas for calculating of this relation [14].

Table (4) :Modeling parameters and numerical acceptance criteria for nonlinear procedures-reinforced concrete columns [8].

Conditions			Modeling Parameters[4]			Acceptance Criteria[4]				
						Plastic Rotation Angle, radians				
						Performance Level				
							Component Type			
			Plastic Rotation Angle, radians		Residual Strength Ratio		Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Columns controlled by flexure[1]										
$\frac{P}{A_g f'_c}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	C	≤ 3	0.02	0.03	0.2	0.005	0.015	0.02	0.02	0.03
≤ 0.1	C	≥ 6	0.016	0.024	0.2	0.005	0.012	0.016	0.016	0.024
≥ 0.4	C	≤ 3	0.015	0.025	0.2	0.003	0.012	0.015	0.018	0.025
≥ 0.4	C	≥ 6	0.012	0.02	0.2	0.003	0.01	0.012	0.013	0.02
≤ 0.1	NC	≤ 3	0.006	0.015	0.2	0.005	0.005	0.006	0.01	0.015
≤ 0.1	NC	≥ 6	0.005	0.012	0.2	0.005	0.004	0.005	0.008	0.012
≥ 0.4	NC	≤ 3	0.003	0.01	0.2	0.002	0.002	0.003	0.006	0.01
≥ 0.4	NC	≥ 6	0.002	0.008	0.2	0.002	0.002	0.002	0.005	0.008
ii. Columns controlled by shear[1, 3]										
All cases [5]			—	—	—	—	—	—	.0030	.0040
iii. Columns controlled by inadequate development or splicing along the clear height[1,3]										
Hoop spacing ≤ d/2			0.01	0.02	0.4	0.005	0.005	0.01	0.01	0.02
Hoop spacing > d/2			0.0	0.01	0.2	0.0	0.0	0.0	0.005	0.01
iv. Columns with axial loads exceeding $0.70P_o$[1, 3]										
Conforming hoops over the entire length			0.015	0.025	0.02	0.0	0.005	0.01	0.01	0.02
All other cases			0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. To qualify, columns must have transverse reinforcement consisting of hoops. Otherwise, actions shall be treated as force-controlled.
4. Linear interpolation between values listed in the table shall be permitted.

Table (5) :Modeling parameters and numerical acceptance criteria for nonlinear procedures-reinforced concrete beams [8].

Conditions			Modeling Parameters[3] Plastic Rotation Angle, radians a	Modeling Parameters[3] Plastic Rotation Angle, radians b	Residual Strength Ratio c	Acceptance Criteria[3] Plastic Rotation Angle, radians — Performance Level IO	Primary LS	Primary CP	Secondary LS	Secondary CP
i. Beams controlled by flexure[1]										
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d/2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d/2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. When more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.

Table (6): Description of performance levels of the concrete frame [12].

Elements	Type	Structural Performance Levels — Collapse Prevention	Life Safety	Immediate Occupancy
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (< 1/8" width) for ductile columns. Minor spalling in nonductile columns. Joint cracks < 1/8" wide.	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some nonductile columns. Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints < 1/16" width.
	Drift[2]	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent

Similarly, P-M interaction details and moment-rotation for column hinges are shown in Figures (5) and (6), respectively. The building frame is modeled by two nodes frame elements (three degrees of freedom in each end) through computer program SAP2000 (V.14) model construction window, using the geometric and structural details as mentioned above.

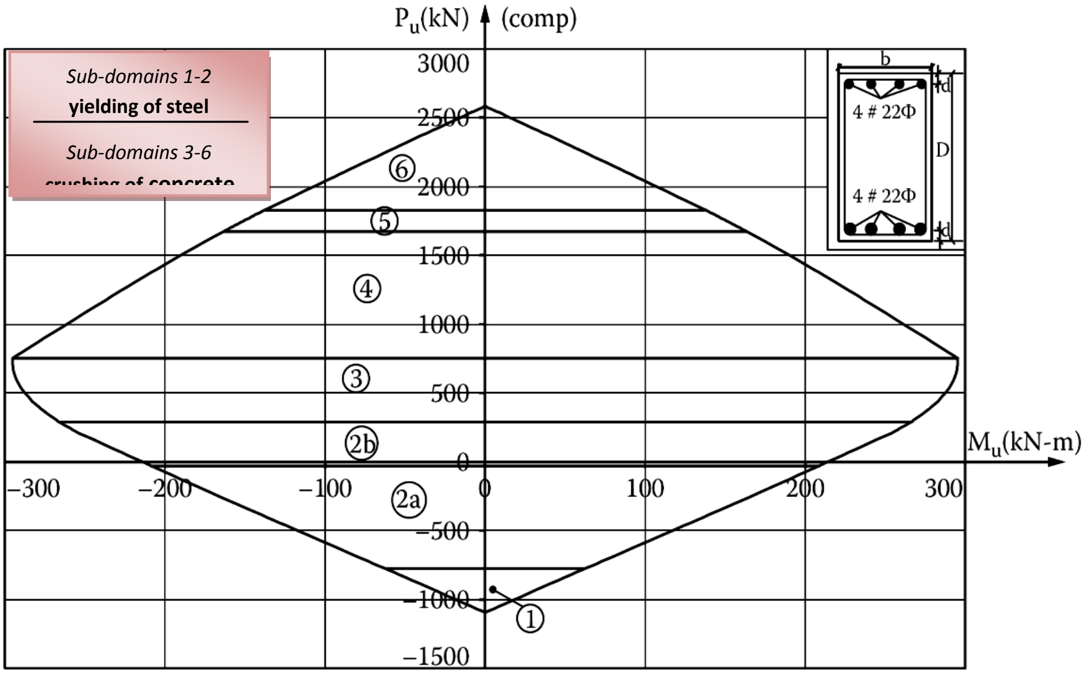


Figure (3): P-M interaction curve for beam hinges.

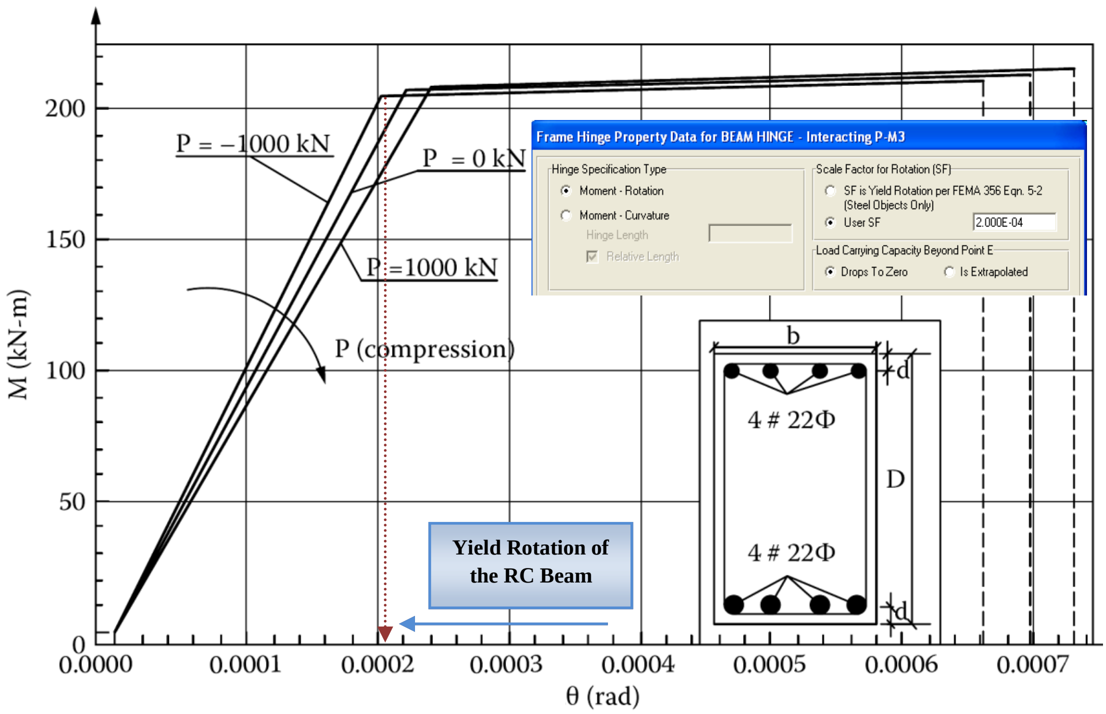


Figure (4): Moment-rotation for beam hinges.

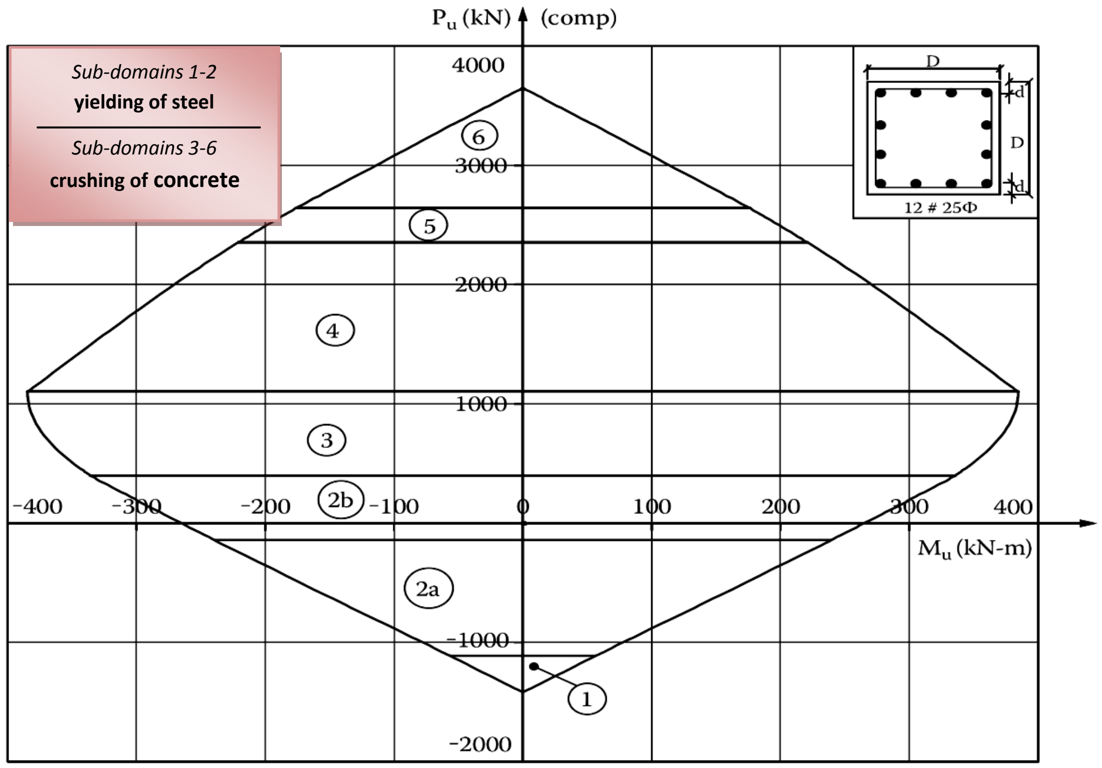


Figure (5): P-M interaction curve for column hinges.

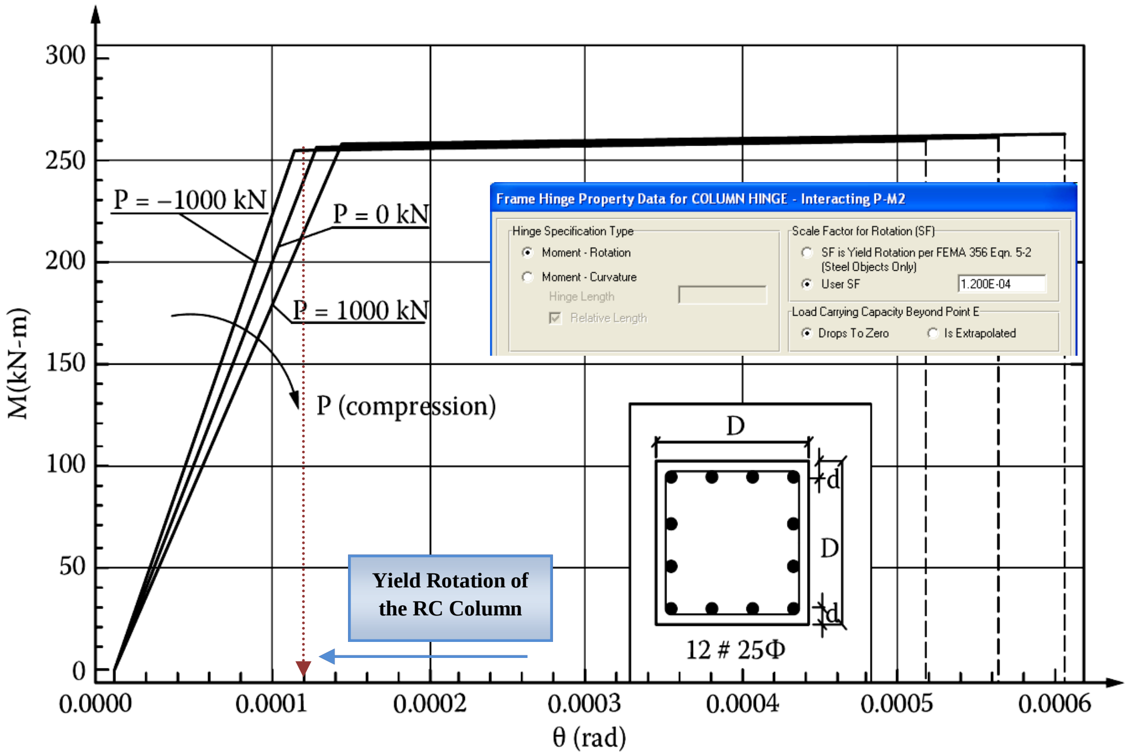


Figure (6): Moment-rotation for column hinges.

1- Lateral static loads equivalent to seismic loads

The seismic parameters to determine the base shear force of the frame, are stated depending on the seismic characteristics of the Mosul city, the base shear force of the frame is:

$$V = (0.2\times0.0725\times1.25\times1.0\times1.5)\times4587 = \underline{\mathbf{124.71}}\ kN$$

$$F_t = 0.07\times0.884\times124.71 = \underline{7.72}\,kN$$

using equation (6) with the help of Microsoft Excel, the lateral force on each story, starting from the first story to roof, is shown in Table (7).

2- Seismic demand and performance point

Two main approaches are used to evaluate the performance point (maximum inelastic displacement of the structure), *Capacity-Spectrum Method* of ATC-40 [7] and *Coefficient Method* of FEMA 356 [8]. In the present study the Capacity-Spectrum Method is more suitable for the evaluation task. Other procedures can be found in the literature.

Table (7): Lateral force on each story

Story No. (i)	h_i (m)	W_i (kN)	$W_i.h_i$	F_x (kN)
1	4.44	458.70	2036.62	2.13
2	8.88	458.70	4073.25	4.25
3	13.32	458.70	6109.88	6.38
4	17.76	458.70	8146.51	8.51
5	22.20	458.70	10183.14	10.64
6	26.64	458.70	12219.77	12.76
7	31.08	458.70	14256.40	14.89
8	35.52	458.70	16293.02	17.02
9	39.96	458.70	18329.65	19.15
10 ***roof***	44.40	458.70	20366.28	21.27
Summation	**------**	**4587**	**112014.54**	**117.00**

In the Capacity-Spectrum Method of ATC-40, the process begins with the generation of a force-deformation relationship for the structure. Then the results are plotted in Acceleration-Displacement Response Spectrum (ADRS) format as shown in Figure (7). This format is a simple conversion of the base shear versus roof displacement relationship using the dynamic properties of the system, and the result is termed a *capacity spectrum* for the structure.

The seismic ground motion specified for present study is also

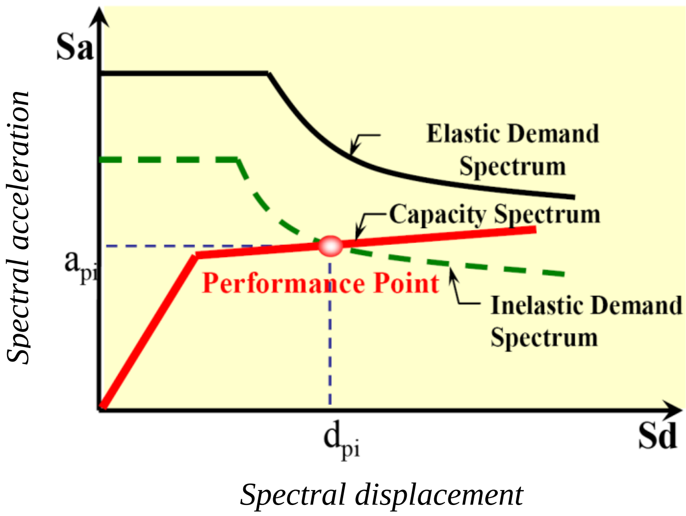


Figure (7): Capacity and demand

converted to Acceleration-Displacement Response Spectrum (ADRS) format, and the result is termed an *Elastic Demand spectrum* (usually 5% damping) of the structure.

In addition, the inelastic demand spectrum is modified from elastic demand spectrum by a procedure of effective damping to present the inelastic structural behavior under a specific ground motion. The effective damping includes the inherent damping in the structure and equivalent viscous damping taking into account for the energy dissipation of the hysteretic behavior of the structure [7] as shown in Figure (8). The intersection of capacity spectrum and inelastic demand spectrum shown in Figures (7) is named as ***performance point***, can be located through an iterative calculations as detailed in ATC-40 [7].

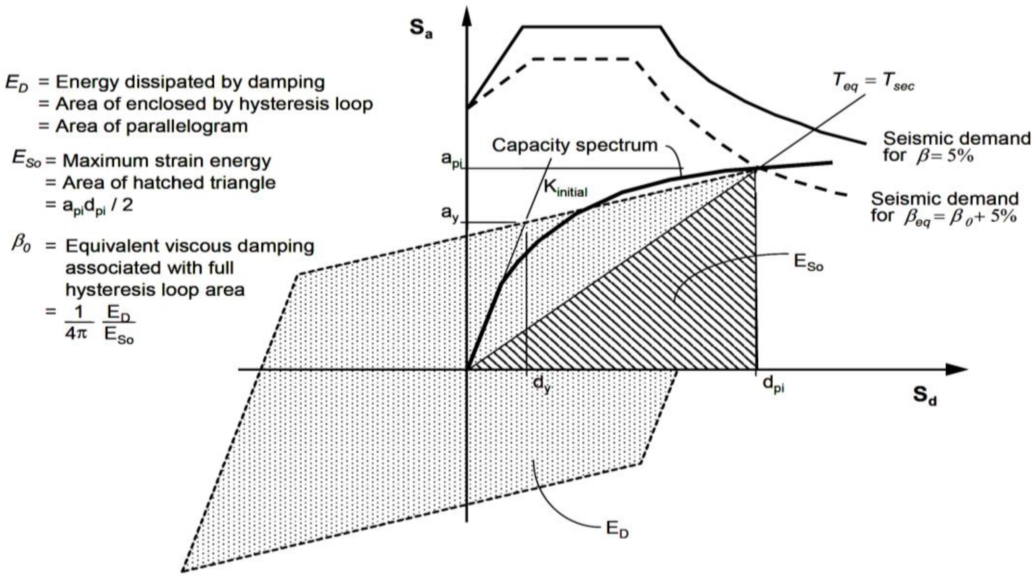


Figure (8): Graphical representation of the Capacity-Spectrum method, as present in ATC-40 [7].

The effective period is computed from the initial period of vibration of the nonlinear SDOF oscillator and from the maximum displacement ductility ratio, ($\mu=\Delta_{max}/\Delta_{yield}$). The corresponding values for performance point, which reflects the seismic performance of the present building frame, are listed in Table (8) and shown in Figure (9).

Table (8): Characteristics of performance point of the frame according to ATC-40 capacity spectrum approach.

Effective Damping (β_{eff}) Unit less	Effective Period (T_{eff}) Sec.	Spectral Acceleration (S_a)g Unit less	Spectral Displacement (S_d) cm	Base Shear (V) kN	Displacement at roof (Δ_{roof}) cm
0.075	0.879	0.408	7.91	598.41	10.427

In the present study it was aimed to assess seismic response of the ten-story building frame in a typical earthquake zone with seismic coefficients $\mathbf{C_a = C_v = 0.4}$ **(Soil Type B)** as shown in Figure (9) [6]. The static nonlinear analysis (pushover analysis) of lateral seismic forces is preferably applied after the initial pushover analysis for the dead load plus live load.

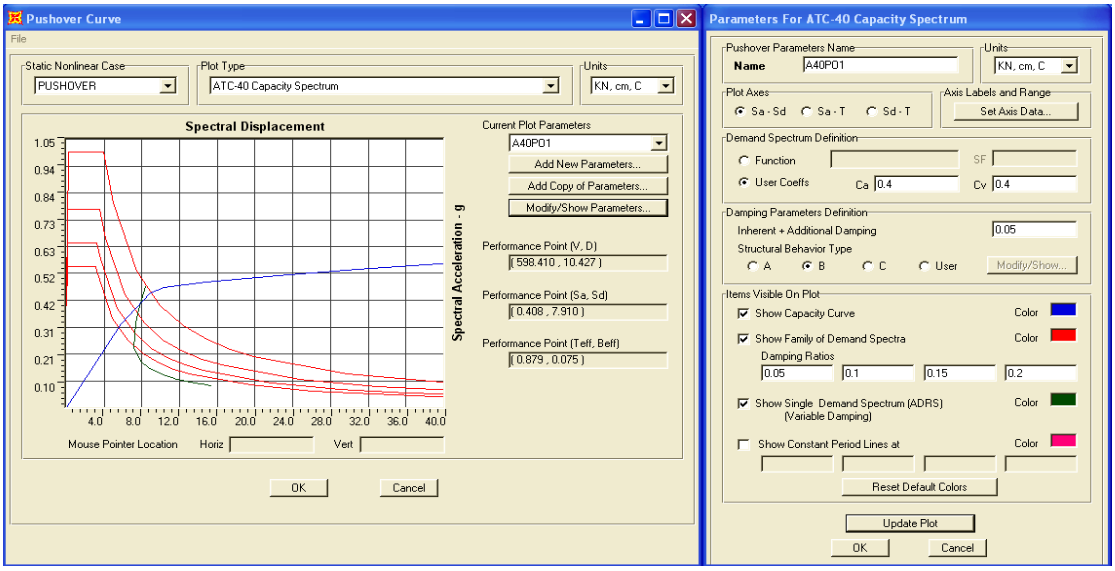


Figure (9): Demand spectrum, capacity spectrum, and parameters of ATC-40 method .

Figure (10) shows the capacity response of two actions of the plastic hinge up to failure. Once when the hinge is subjected to the shear failure and another one to flexural failure. The maximum base shear of the structure of about (996 kN) for whole analysis and the ultimate roof displacement is about (160 cm). The scaled ratio between the values of base shear deduced from the UBC code relations and the pushover analysis of the frame is (7.5) and this is acceptable according to UBC.

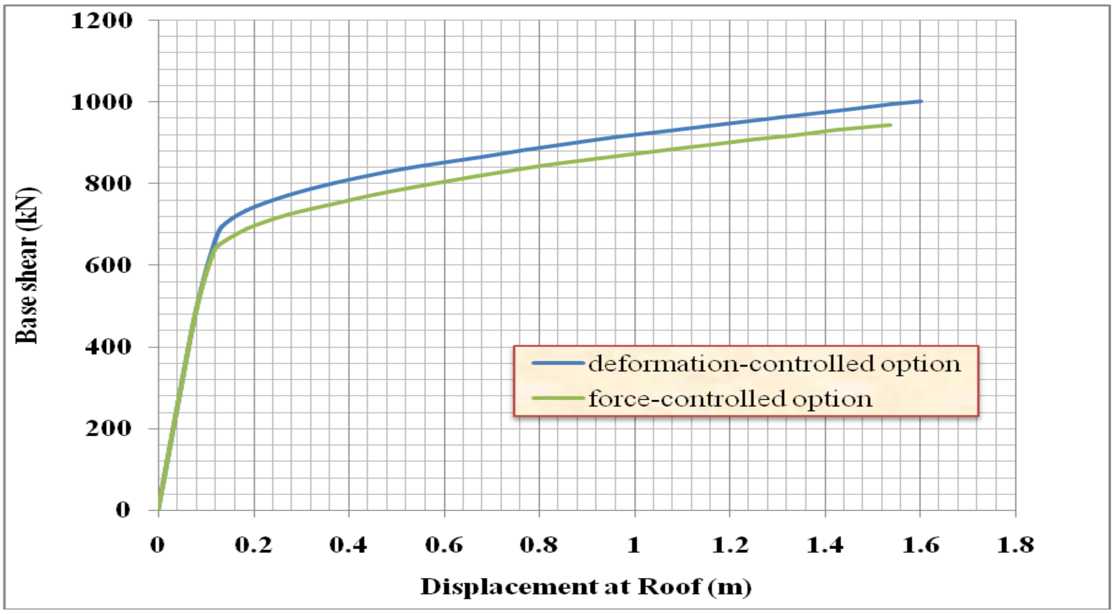
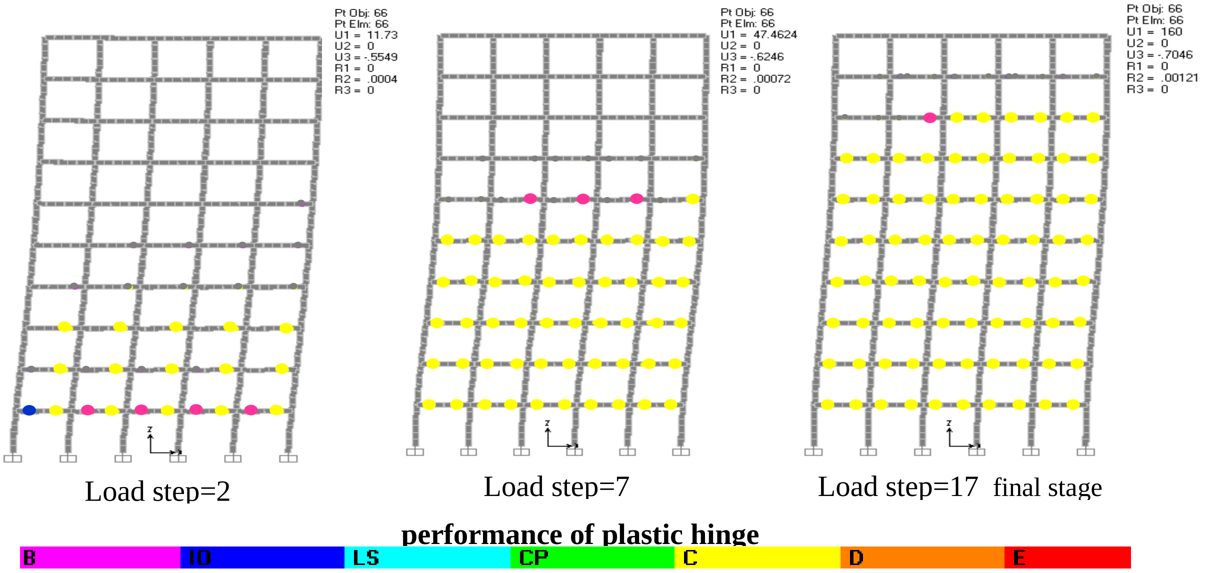


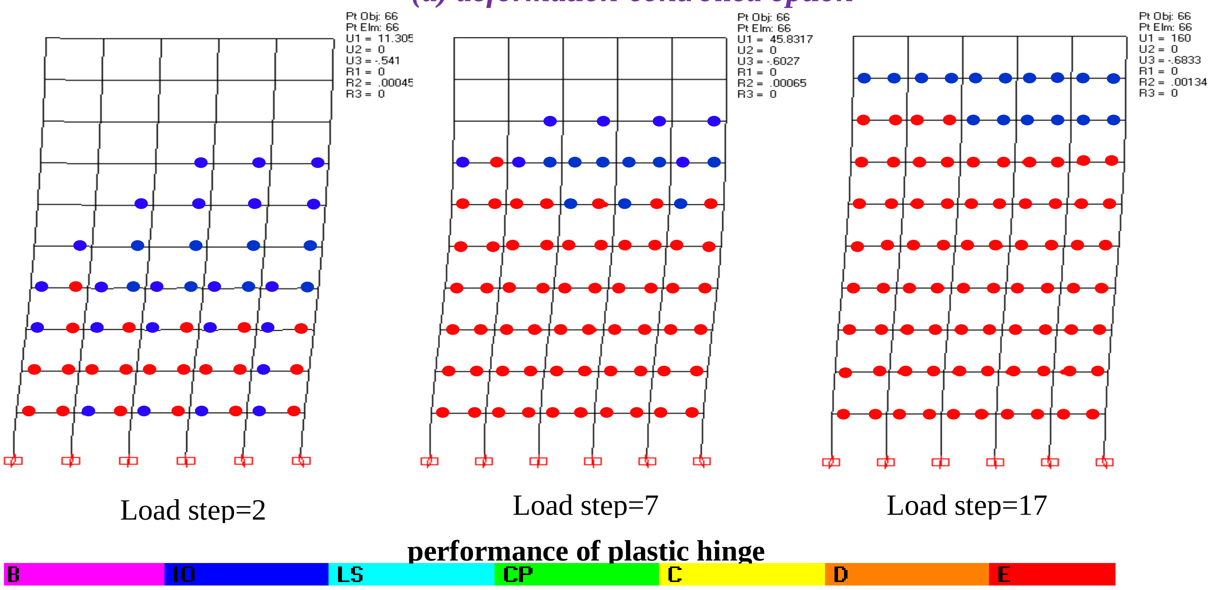
Figure (10):Capacity curve of the building frame.

Hinges were assigned at both ends of each element (beams and columns). Axial force – bending moment (P-M) interaction curves were used to govern the behavior of hinges formed in the beams and columns during the analysis. The SAP2000 default limitations were depended upon nonlinear analysis procedure.

Figure (11) shows the plastic hinge patterns at different steps of loading and different control options which govern the behavior of plastic hinge during the analysis. Also the Figure shows their state illustrated by appropriate colors. All the plastic hinges formed in the beams are positioned in the end of (*collapse prevention CP*) branch of Acceptance Criteria of plastic hinge in related to its flexural action, while the plastic hinges in the other action (Figure 11b) in damage state.



(a) deformation-controlled option



(b) force-controlled option

Figure (11): Plastic hinge patterns at different load steps-two actions of plastic hinge during the analysis.

Figure (12) shows the ductility ratio of the frame structure according to FEMA-440 Displacement Modification approach [7]. The displacement ductility gives a simple quantitative indication of the severity of the peak displacement relative to the displacement necessary to initiate yielding. The ductility ratio directly affects hysteretic behavior in reinforced concrete structures.

Lateral deformations at the performance point are to be checked against the deformation limits of ATC-40. Table (9) presents deformation limits for various performance levels [7]. Maximum total drift is defined as the story drift at the performance point displacement. Maximum inelastic drift is defined as the portion of the maximum total drift beyond the effective yield point. For *Structural Stability*, the

maximum total drift in story *i* at the performance point should not exceed the quantity of (0.33 S_i / W_i), where S_i is the total calculated lateral shear force in story *i* and W_i is the total gravity load at story *i* [7].

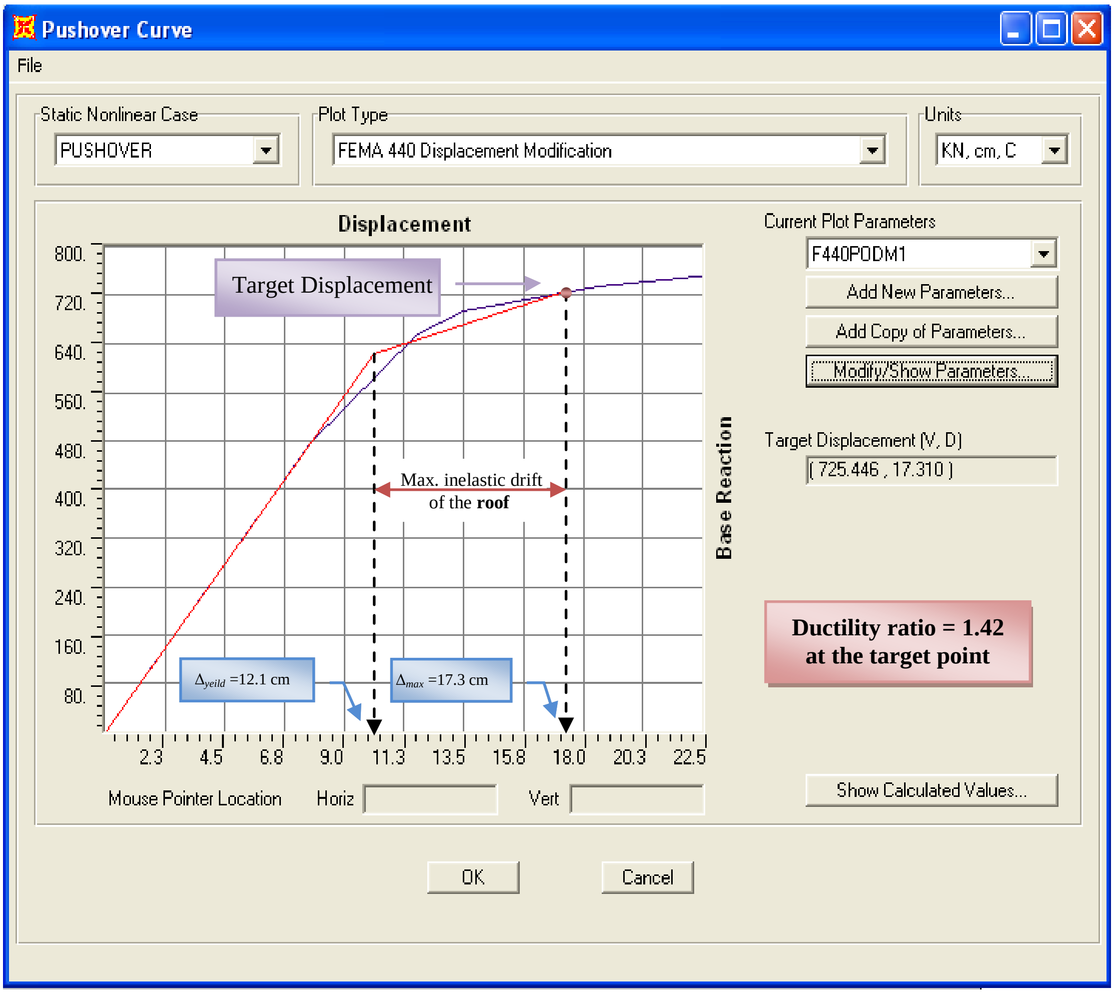


Figure (12): Ductility ratio of the frame according to FEMA-440 Displacement Modification approach.

Table (9): Story drift ratio of the present analysis and deformation limits according to ATC-40 recommendations [7].

Story drift limit	Story drift ratio after analysis	Performance Level			
		Intermediate Occupancy	Damage Control	Life Safety	Structural Stability
Maximum Total Drift	0.0039	0.01	0.01-0.02	0.02	0.33 S_i/W_i $(0.021)_{\text{at roof}}$
Maximum Inelastic Drift	0.0012	0.005	0.005-0.015	No limit	No limit

Example 2

A five bays-ten stories regular frame in reinforced concrete is considered as a second numerical case. The building frame consists of structural elements as follows:

1. (*450×450 mm*) square RC columns, reinforced with (*12 Ø25 mm*), shear stirrups of (*Ø8 mm @ 500 mm c/c*), so that the spacing of shear reinforcement does not satisfactory the ACI code and IBC code requirements.
2. (*300×450mm*) RC beams, reinforced with (*4Ø22mm*) as tensile and compression steel with shear stirrups of (*Ø10mm @ 200mm c/c*).
3. (*125 mm*) thickness of RC slab.

The same characteristics and definition of materials of example 1 are used in example 2. The expressions for axial load-bending moment (P-M) interaction and moment-rotation relationship is assumed in the modeling of hinge for the beams and columns. While the force-controlled option (brittle behavior) is only assumed for the columns during the analysis because of inadequate shear reinforcement in these columns.

Figure (13) shows the capacity response of the plastic hinge up to failure. The maximum base shear force of (1113 kN) to the end of analysis and the ultimate roof displacement is about (46 cm). It is clear from the Figure (13) that there is a large increase in base shear force scaling to roof drift, this is as a result of type of plastic hinges formed in the first story columns with assuming the force-controlled option.

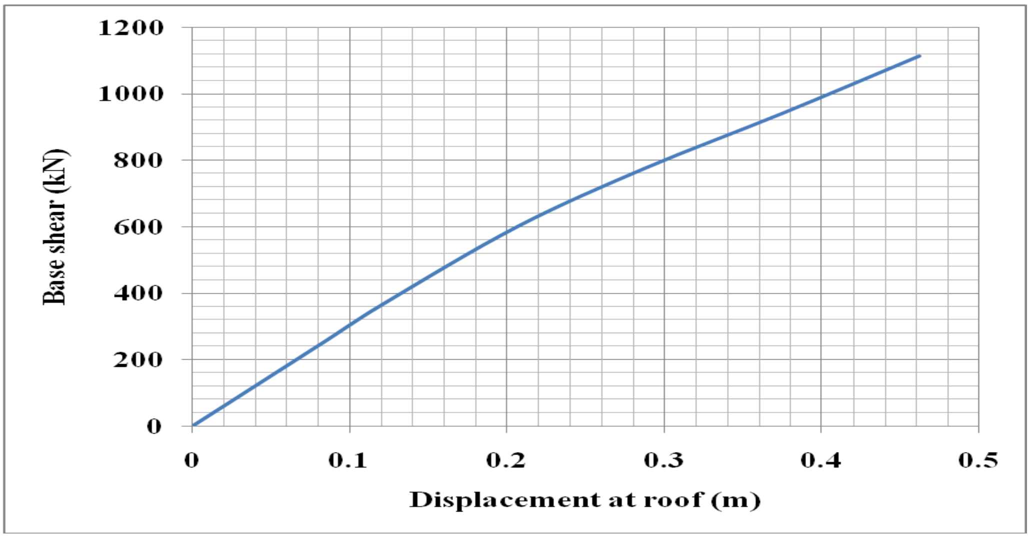


Figure (13):Capacity curve of the building frame.

Figure (14) shows the plastic hinge patterns at two steps of loading. Also the Figure shows their state illustrated by appropriate colors. All the plastic hinges formed in the beams are positioned in the safe side of elastic range (A to B) of Acceptance Criteria of plastic hinge behavior, while some of the plastic hinges formed in the columns are positioned in the risk damage state. Therefore, the building must be checked to the requirements of seismic codes to prevent such these states.

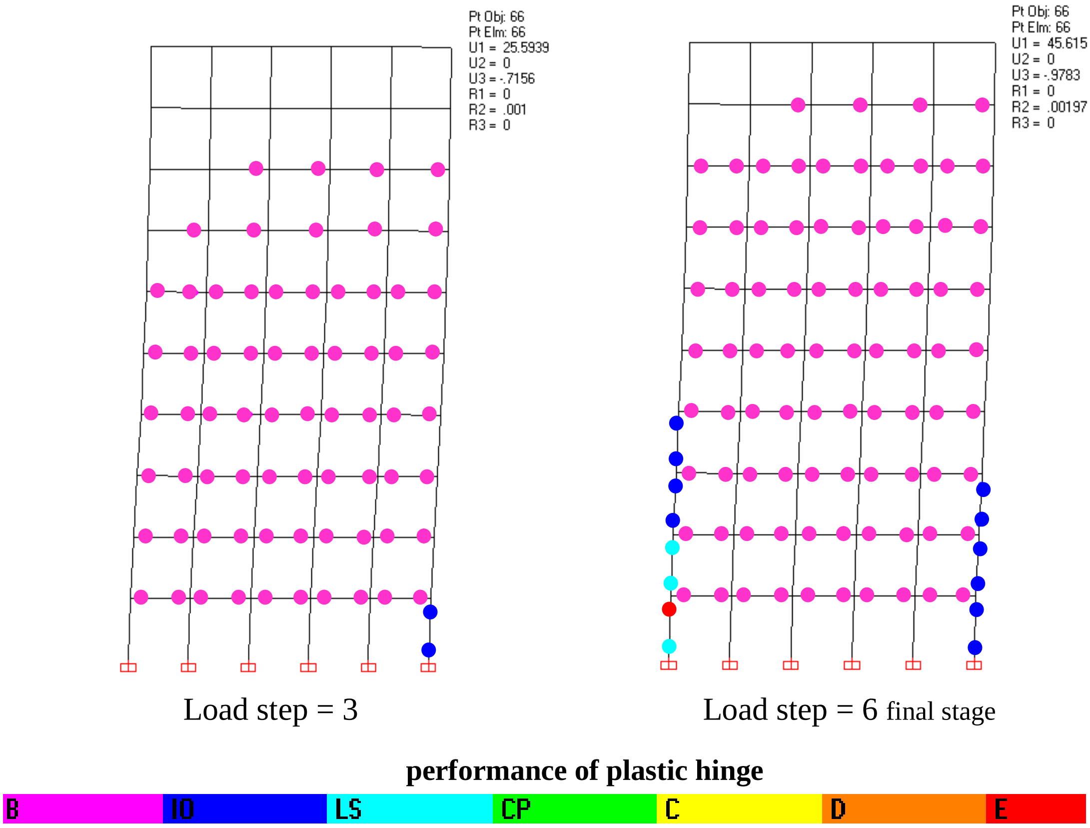


Figure (14): Plastic hinge patterns at two load steps.

Conclusions

The nonlinear static (Pushover) analysis as introduced by ATC-40 has been utilized for the evaluation of an existing reinforced concrete building frame, in order to examine its applicability. Potential structural deficiency in RC frame, when subjected to a moderate seismic loading, were estimated by the nonlinear pushover procedure. The procedure showed that the frame is capable of withstanding the presumed seismic force with some significant yielding at several beams. The main conclusions can be drawn as follows:-

1. Sequence of formation of plastic hinges (yielding) in the frame members can be clearly seen in the beams only. The building clearly behaves like the strong column-weak beam mechanism.
2. Lateral deformations at the performance point are to be checked against the deformation limits of ATC-40. Maximum total drift, maximum inelastic drift, and

structural stability do not exceed the limitations of the performance level, therefore the present building (for example 1) is considered safe for persons against seismic force.

3. All the plastic hinges formed in the beams are positioned in the dangerous branch (*collapse prevention CP*) of Acceptance Criteria of plastic hinge, this demands strengthening the beams.
4. Through the comparison between different options of the plastic hinge behavior during the pushover analysis, the plastic hinge formed due to its brittle behavior put it in the greater severity level.
5. Any missing of the international codes requirements or mistakes in the design may result in collapse of the building as shown in example 2.

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