Nonlinear Behavior of Flat Slabs with Openings in Column Strip

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ملخص البحث

زلزال أكتوبر 1992 نتج عنه دمار اجتماعى واقتصادى بمصر، لذلك توجهت جهود الكثير من الباحثين لتقيم آداء الأنظمة الإنشائية للمباني الخرسانية المنفذة بمصر ودراسة مدى ملائمتها لمقاومة الزلازل. المباني المكونه من بلاطـــــات خرسانية مسطحة وأعمدة [Flat Slab – Columns Buildings] تعتبر من المباني الخاصة التي تحتاج الى تقييم خصوصا في حالة تواجد فتحات بشريحة العمـــود.

هذة الدراسة تقـدم تحليل غير مرن لمجموعـــة من هذة المباني تم تصميمها طبقا للكـــود المصري للأحمال [ECP-2012] وافترض انها منفذة بمدينة القاهرة. التحليل الغير مرن تم عمله طبقا لـــ [ATC-40] وتم تقييم آداء هذة المباني تحت تأثير زلزال متوسط الشدة باستخدام البرنامج التجاري [ETABS] وتم عمل النمذجة للبلاطات باستخدام طريقة تسمى [Grid Beam Elements]. وأخيرا تم عرض النتائج والخلاصة والتوصيات.

Abstract.

Due to the social devastation and economic impacts of recent earthquake which struck Cairo on October 1992, greater effort has been given to explicitly evaluate how reinforced concrete (RC) buildings are likely to perform during earthquakes. As being one of the special reinforced concrete structural forms flat slab-columns systems, it needs further attention about its adequacy to resist earthquake especially if constructed with large openings in column strip. This study presents through numerical simulations, a nonlinear static analysis to assess the seismic performance of 16 RC buildings. Each building is assumed to be in Cairo and designed according to the Egyptian code (ECP-2012) as a flat slab-columns system with large opening in the area common to intersecting columns strips near the internal column. The nonlinear static pushover analysis is performed following the ATC-40 procedures in assessing the performance of these buildings under moderate earthquake motions. The well-known software package ETABS is used for implementing the buildings models and performing the pushover analysis. A models slab has been designed and assembled in order to represent the inelastic behavior by using the concept of grid beam elements. The results of the study conclude that properly designed flat slab-columns buildings with opening in column strip perform well under seismic loads, and it is capable of sustaining moderate earthquake.

Keywords: flat slab-columns systems, pushover analysis, grid beam elements, moderate earthquake

1. Introduction

On October 1992, a devastating earthquake struck Cairo causing detrimental effects in reinforced concrete (RC) buildings ranging from repairable damage to total collapse. Considerable attention has been paid in order to explicitly evaluate the seismic adequacy of existing buildings. In particular, structures venerable to damage must be identified and an acceptable level of safety must be determined. To make such assessment, simplified linear-elastic methods are not adequate. Thus, the structural engineering community has developed a new generation of design and seismic procedures that incorporate performance based structures and are moving away from simplified linear elastic methods and towards a more nonlinear technique. Recent interests in the development of performance based codes for the design or rehabilitation of buildings in seismic active areas show that an inelastic procedure commonly referred to as the pushover analysis is a viable method to assess damage vulnerability of buildings.

2. Previous Studies

No available papers on assessing the seismic performance of flat slabs with openings but the different codes of practice and researches give rules, which limit the size and the location of holes in flat slabs.

El Kafrawy, A.F. and El kafrawy, M.F. [8], Ibrahim, W.W. [13], A.M. Elbehairy and M.Rabie [2], Sanger and Ahmed. S. [20], Salakawy E.F. [9] and Prawat, R. [16] studied the effects of openings on the elastic behavior of a flat plate floor under gravity loads.

3. Building Model

3.1 Geometry

In this research, 16 R.C Buildings were investigated, namely A1-3, B1-3, C1-3, D1-3, E1-3 and F. Each building is assumed to be in Cairo and designed according to the Egyptian code (ECP) [7] as a flat slab-columns system with large openings in the area common to intersecting columns strips near the internal column. Building height is assumed to be eight-story representing high-rise flat slab buildings that can be constructing without shear walls. The typical floor height is 3m. All columns are having square cross sections 0.75x0.75m and assumed to be fixed at the foundation level. The buildings are provided with 0.25 m thick floor slabs. The selected reinforcement ratios are within the range allowed by Egyptian code. This paper presents the analysis results and details of model type F.

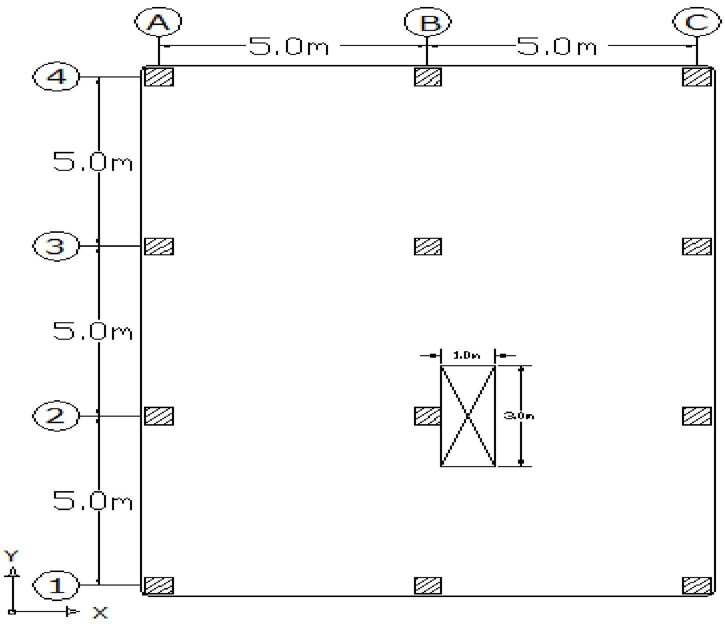

Figure 1. Plan of the Case Study F

3.2 Material Properties

Concrete having a characteristic strength after 28-days 28N/mm^2, and high-grade steel with yield strength 360N/mm^2 are used for analysis and design. The specific weight of reinforcement concrete is taken 25KN/m^3 and modulus of elasticity is determined using the formula $Ec = \sqrt{14000fcu}$. The elastic modulus of steel is taken 210KN/ mm^2. Poisson's ratios of concrete and steel are taken equal to 0.2 and 0.3 respectively.

3.3 Gravity Loads

The loads that act on the RC buildings are categorized as gravity loads, which include dead loads (DL) and live loads (LL); and lateral loads, which include earthquake loads. The dead loads include the own weight of the structural elements, the weight of flooring cover (1.5 KN/m2), and the weight of partitioning elements (2.0 KN/m2). According to EGP [9], the live load for residential RC building is 2.0 KN/m2.

3.4 Lateral Static Loads

According to ECLF [9], the seismic base shear force, Fb; for each horizontal direction in which the building is analyzed, shall be determined using the following expression:

$F_b = S_d (T1). \lambda. W / g$

Where: Sd (T1) is the ordinate of the design spectrum at period T1; T1 is the fundamental period of vibration of the building for lateral motion in the direction considered; W is the total weight of the building, above the foundation level; g is the gravity acceleration; λ is the effective modal mass correction factor, the value of which is equal to: $\lambda = 1$ for $T \geq 2TC$, and $n > 2$ stories. The value of the fundamental period of vibration, T, determined using the following expression:

$$T = C_t.H^{3/4}$$

Where C_t is a factor determined according to the structural system and building material and equal to 0.05; H is the height of the building, in m, from the foundation or from the top of a rigid basement. The ordinate of the design spectrum, Sd (T1), can be determinate from:

$$S_d(T) = \frac{2.5}{R} a_g.\gamma.S.\left[\frac{Tc}{T}\right] \geq [0.20] a_g \, \gamma$$

Where a_g is the design ground acceleration for the reference return period; Tc is the upper limit of the period of the constant spectral acceleration branch; S is the soil factor. γ is the importance factor. R is the reduction factor according the statical system of the structure. The seismic zone considered in this study is zone (3) (for Cairo city) and the shape of spectrum is type 1. Importance factor γ = 1, Soil class “C” and a soil factor S=1.5, the reduction factor (R=5). It should be noted that, ECLF 2012 recommends that in the application of the ESFM method, the building should meet the criteria for regularity in both plan and elevation, and with calculated structural period T not greater than 2 sec or 4Tc (1 sec for the selected soil class (class “C”)). The total base shear, Fb, shall be determined by applying horizontal forces Fi to each story mass mi and shall be distributed as follows: The values of base shear at

$$Fi = \left[\frac{Zi.Wi}{\sum_{j=1,n} ZjWj}\right].Fb$$

Where Fi is the horizontal force acting on story i; Fb is the seismic base shear force; Zi, Zj are the heights of the masses mi, mj above the foundation level respectively; Wi, Wj are the weights of masses mi, mj; n is the number of stories above foundation level. The loads which are considered in the seismic design of building are the full dead loads plus 25% of the live loads.

4. Nonlinear Static Analysis

Structures suffer significant inelastic deformation under a strong earthquake and dynamic characteristics of the structure change with time, so investigating the performance of a structure requires inelastic analytical procedures accounting for these features. Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic analysis procedures basically include inelastic time history analysis and inelastic static analysis which is also known as pushover analysis. The inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. However, the use of inelastic time history analysis is limited because dynamic response is very sensitive to modeling and ground motion characteristics; it requires proper modeling of cyclic load-deformation characteristics considering deterioration properties of all important components, also, it requires availability of a set of representative ground motion records that accounts for uncertainties and differences in severity, frequency and duration characteristics. Moreover, computation time, time required for input preparation and interpreting voluminous output make the use of inelastic time history analysis impractical for seismic performance evaluation. Inelastic static analysis or pushover analysis has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. Inelastic static analysis procedures include Capacity Spectrum Method, Displacement Coefficient Method and the Secant Method; Sermin Oguz. [18].

5. Pushover Methodology

Pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic nonlinear force displacement relationship can be determined.

The following steps are included in the pushover analysis.

1. Create the basic computer model (without the pushover data) in the usual manner.
2. Define properties and acceptance criteria for the pushover.

3. Locate the pushover hinges on the model by selecting the frames members and assigning them one or more hinge properties and hinge locations.

4. Define the pushover load cases. In ETABS more than one pushover load case can be run in the same analysis. Also a pushover load case can start from the final conditions of another pushover load case that was previously run in the same analysis. Typically the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Pushover load cases can be force controlled, that is, pushed to a certain defined force level, or they can be displacement controlled, that is, pushed to a specified displacement. Typically a gravity load pushover is force controlled and lateral pushover is displacement controlled. ETABS allows the distribution of lateral force used in the pushover to be based on a uniform acceleration in a specified direction, a specified mode shape, or a user-defined static load case.

5. Definition of the control node: control node is the node used to monitor displacements of the structure. Its displacement versus the base-shear forms the capacity (pushover) curve of the structure.

6. Run the static nonlinear pushover analysis.

7. Display the pushover curve.

8. Display the capacity spectrum curve. Note that you can interactively modify the magnitude of the earthquake and the damping information on this form and immediately see the new capacity spectrum plot. The performance point for a given set of values is defined by the intersection of the capacity curve (green) and the single demand spectrum curve (yellow). Also, the file menu in this display allows you to print the coordinates of the capacity curve and the demand curve as well as other information used to convert the pushover curve to Acceleration-Displacement Response Spectrum format (also known as ADRS format).

9. Review the pushover displaced shape and sequence of hinge formation on a step-by-step basis.
10. Review member forces on a step by-step.
11. Output for the pushover analysis can be printed in a tabular form for the entire model or for selected elements of the model. The main output of a pushover analysis is in terms of response demand versus capacity, values of internal actions (bending moment, shear, torsion) acting in the grid elements at each step of the analysis and the visualization of the deformed shape of the structure with in evidence the state of plastic hinges at each step. The latter permits to immediately and quite clearly understand the failure mode of the model and its damage sequence.

If the demand curve intersects the capacity envelope near the elastic range, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse.

The software shows the plastic hinges state according to different colors: fuchsia is the color for plastic hinge which are beyond the first cracking, yellow for these which are beyond the yielding (or the 80% of the capacity) and finally orange for these which have reached the capacity.

6. Flat Slab Modeling for Nonlinear Analysis

The slab is represented by a grid of beam finite elements, fixed at joints, arranged in two orthogonal directions. It has been decided to use beam elements with width equal to 100 cm and each grid beam is composed by an elastic part and by non-linear hinges. The portions of columns being in the thickness of the slab are modeled by four beam elements (core) with length equal to the column cross section, positioned along the four column semi-axes, which are infinitely rigid in the slab plain, thus creating a situation similar to reality; Merve Zorlu. [14], Guglielmo Corti. [10]. Also the elements intersecting the column have width equal to c + d (where c = side length of column and d = mean effective depth of the slab taking into account that the effective depth d is considered constant overall the slab), c + d is the width of the shear critical section according to the definition of ACI 318 and is consider also for torsion in the transverse direction. A shear plastic hinge has been placed in each one of these elements at a distance d/2 from the column face; Guglielmo Corti. [10]. Ilham Nurhuda, Han Ay Lie. [12]. The result of numerical simulation shows that the three-dimension of grid model can be used to model behavior of flat-plate structures. This approach is usually preferred in the analysis of flat slab buildings due to the ease of model definition and run time; Coronelli [6], Hueste, Browning, Lepage and Wallace [15], Merve Zorlu [14], Ying Tian, Jianwei Chen, Aly Said and Jian Zhao [4], IIham Nurhuda, Han Ay Lie. [12], G. Gugliotta, A. S. Petrolo [11] and others. The eccentrically of the loads in this type of modeling have not appreciably effect on the results; Whittle R. [21].

7. Performance Level

As shown in figure 1, three points labeled IO (Immediate Occupancy), LS (Life Safety) and CP (Collapse Prevention) are used to define the acceptance criteria for the hinge.

Immediate occupancy IO: Overall damage to the building is very light. The strength and stiffness remain nearly as those of pre-earthquake loading. Cladding and ceilings as nonstructural elements as well as the mechanical and electrical components remain secured.

Life safety level LS: Significant structural and nonstructural damage can be observed. Substantial amount building strength and stiffness are lost compared with pre-earthquake lateral strength and stiffness. But the gravity-load-bearing elements function. Nonstructural components are secured and not presenting a falling hazard.

Collapse prevention CP: The structure sustains severe damage. The lateral-force resisting system loses most of its pre-earthquake strength and stiffness. Load-bearing columns and walls function, but the building is near collapse.

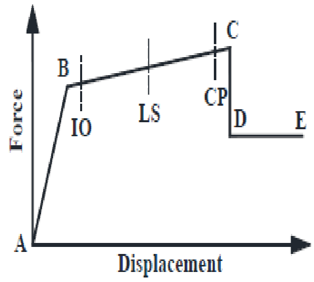

Figure 1. Different Stages of Plastic Hinge

8. The Results

The resulting pushover curve for model F is shown in Figure 3, it starts to deviate from linearity to nonlinearity due to the inelastic action of slabs. The curve linearity can be retrieved again with a small slope as the building is pushed well into the inelastic range. The pushover curve shows no decrease in the load carrying capacity of the building, at target displacement of 0.28 m, the base shear of the whole structure was 235 ton which is greater than design base shear. Figure: 4 show capacity spectrum curve and performance point which obtained by superimposing the reduced demand spectrum curve for moderate level of shaking with ATC-40 [1] capacity spectrum curve. From this performance point (figure: 4), T_{eff} (1.78) can be obtained, (see Table 1, step 4). From Table 2, it can be seen that for step 4 hinges form beyond CP (collapse prevention) is zero. For step 4 the displacement and base shear force should be considered. At performance point, where the capacity and demand meets, out of 12984 assigned hinges 12351 were in A-B stage, 446, 163, 24, and 0 hinges are in B-IO, IO-LS, LS-CP and CP-C stages respectively. As at performance point, hinges were in LS-CP range, overall performance of building acts like Life Safety to Collapse Prevention. After performing the inelastic analysis under moderate earthquake with seismic design level coefficient Ca = 0.22 and Cv = 0.32 (Zone 3 Soil type C), the base shear at performance point is found to be 198.1 ton which is greater than design base shear. Plastic hinges formation for the building mechanisms at different level have been obtained in figures 5 and 6 (a, b). Plastic hinges formation starts with columns-slabs connections of lower stories, and then propagates to upper stories. As it can be seen, in the following figures, the failure mechanism is of the desirable kind. This is consistent with the known philosophy, which suggests, “strong column and weak beam” (when their strengths are compared).

9. Conclusions

Based on the numerical study, the following conclusions can be deduced:

1. The resulting pushover curve has the linear form before it starts to deviate from linearity to nonlinearity due to the inelastic action of slabs. The curve linearity can be retrieved again with a small slope as the building is pushed well into the inelastic range
2. Pushover curve shows no decrease in the load carrying capacity of the buildings.
3. The formed hinges are not in the dangerous level according to ATC-40 categories under moderate earthquake. Hinges form beyond CP (collapse prevention level) is zero.
4. At performance point, hinges were in LS-CP range. Overall performance of the studied buildings acts like Life Safety to Collapse Prevention. In this event, plastic hinges are formed, causing decrease in the structure rigidity and original strength and followed by severe damage of some structural elements and components. However, a substantial margin remains for additional lateral deformation before collapse would occur. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical.
5. After performing the analysis under moderate earthquake with seismic design level coefficient Ca = 0.22 and Cv = 0.32 [Zone 3 (A=0.15g)-Soil type C], the base shear at performance point is found to be greater than design base shear.
6. Plastic hinges formation starts with columns-slabs connections of lower stories, and then propagates to upper stories.

7. The failure mechanism is of the desirable kind. This is consistent with the known philosophy, which suggests, “strong column and weak beam” (when their strengths are compared).
8. Most of the hinges developed in the slabs and few in the columns but with limited damage.

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Table 1. Model F, Pushover Capacity / Demand Comparison
(Moderate Acceleration X-direction, A=0.15g)

PUSHOVER CAPACITY/DEMAND COMPARISON

File

Step	Teff	ßeff	Sd(C)	Sa(C)	Sd(D)	Sa(D)	ALPHA	PF*Ø
0	1.346	0.050	0.000	0.000	0.107	0.238	1.000	1.000
1	1.346	0.050	0.000	0.000	0.107	0.238	0.641	1.512
2	1.571	0.050	0.072	0.118	0.125	0.204	0.745	1.351
3	1.581	0.055	0.077	0.124	0.123	0.197	0.746	1.351
4	1.844	0.152	0.119	0.141	0.106	0.125	0.792	1.340
5	2.158	0.220	0.176	0.152	0.108	0.094	0.813	1.327
6	2.297	0.235	0.204	0.156	0.112	0.086	0.815	1.330
7	2.349	0.240	0.215	0.157	0.114	0.083	0.814	1.334

Table 2. Model F, Pushover Result in X-Direction

PUSHOVER CURVE

File

Step	Displacement	Base Force	A-B	B-IO	IO-LS	LS-CP	CP-C	C-D	D-E	>E	TOTAL
0	-0.0045	0.0000	12978	6	0	0	0	0	0	0	12984
1	-0.0045	0.0000	12922	62	0	0	0	0	0	0	12984
2	0.0930	161.6316	12904	80	0	0	0	0	0	0	12984
3	0.0995	170.4891	12702	258	24	0	0	0	0	0	12984
4	0.1554	206.1794	12351	446	163	24	0	0	0	0	12984
5	0.2288	227.8632	12141	597	150	96	0	0	0	0	12984
6	0.2669	233.7142	12088	623	151	120	0	2	0	0	12984
7	0.2821	235.0405	12074	637	151	120	0	0	2	0	12984
8	0.2726	217.9676	12984	0	0	0	0	0	0	0	12984

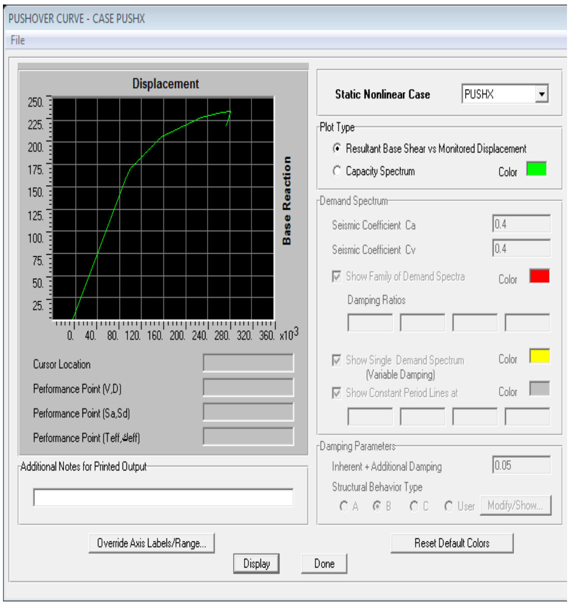


Figure 3. Pushover Curve

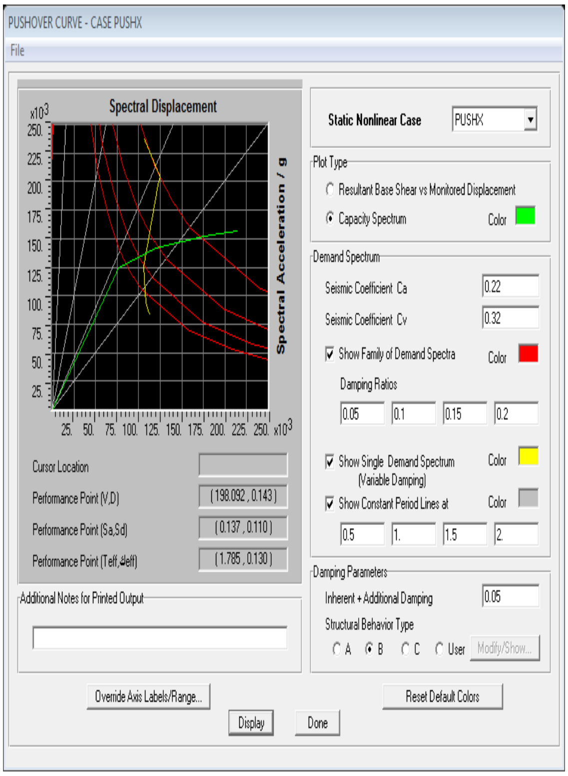


Figure 4. Capacity Curve, Demand Spectrum and Performance Point

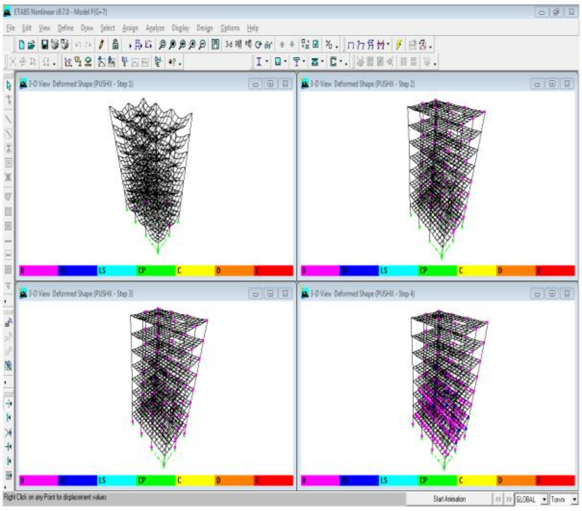
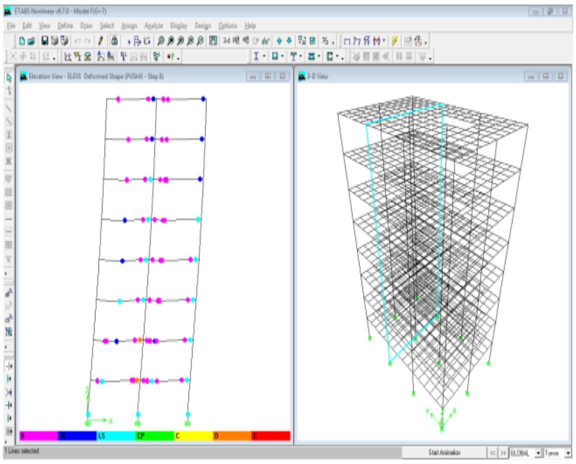
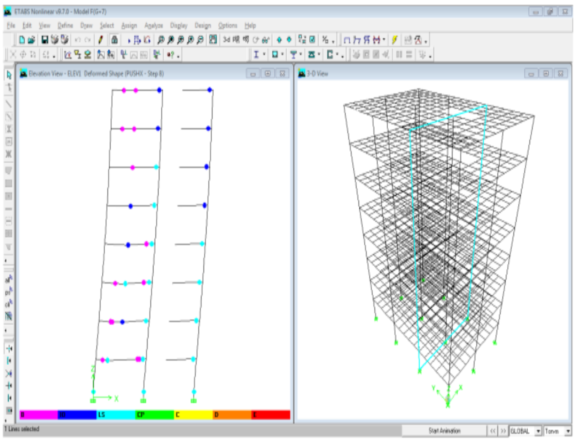


Figure 5. History of Formation of Plastic Hinges-Step (1, 2, 3 and 4)



(a)



(b)

Figure 6 (a, b). History of Formation of Plastic Hinges-Step 8