Nonlinear Analysis Methods for Evaluating Seismic Performance of Multi-Story RC Buildings

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ABSTRACT

A major challenge in performance-based earthquake engineering is to develop simple and practical methods for estimating capacity level and seismic demand on structures by taking into account their inelastic behavior. Researchers and engineers certainly prefer to use nonlinear static methods over complicated nonlinear time-history methods. However, in Nonlinear Static procedure both predetermined target displacement and force distribution pattern are based on a false assumption that the structural behavior and its responses are dominated by the fundamental vibration modes. Therefore, over the past decades, there have been a great number of studies on considering higher mode contribution in nonlinear static results. However, their major drawback is that these approaches are inevitably more complex and time consuming compared to a single-run pushover analysis.

The primary aim of this work is to perform and compare different nonlinear analysis methods for evaluating the seismic performances of structures. For these purposes, three models are considered to represent low-rise, medium-rise and high-rise structures. This consist of a moment resisting reinforced concrete structures with no shear walls, located in a high-seismicity region of Turkey. They are designed according to Turkish Earthquake Code 2007 and TS 500-2000 codes, considering both seismic and gravity loads.

The Displacement-based Adaptive Pushover Analysis (DAP), which was proposed by Antoniou and Pinho (2004), is one of the performance assessments tool for improving the accuracy of the obtained results of nonlinear static analysis in estimating the seismic demands of the structures. In this thesis, reliability of the DAP in estimating the seismic response of 3, 8 and 12-story moment resisting reinforced concrete frames responding in the inelastic range is demonstrated. Therefore, Incremental Dynamics Analysis (IDA), by applying a large set of natural records, and FEMA 440 static pushover procedure are performed for comparison. The capacity curves of the structures, as derived by both DAP and FEMA440 pushover curves are compared with the IDA envelopes by using SeismoStruct software. Performance levels of structures are also estimated and compared by performing DAP and Incremental Dynamic Analysis using SeismoStruct software and, FEMA440 pushover analysis using SAP2000 program. Results are presented and discussed for advantage and disadvantage of procedures.

It is demonstrated that Displacement-based Adaptive Pushover Analysis is adequate for estimating seismic response of reinforce concrete frame and represents an alternative simpler procedure compared to IDA. The DAP method not only automates the pushover analysis, but also improves the accuracy of its results in estimating the seismic demands of the structure.

Keywords: Multi-Mode Pushover Analysis, Incremental Dynamic Analysis, Displacement-based Adaptive Pushover Analysis, and Nonlinear Static Analysis

ÖZ

Performansa dayalı deprem mühendisliğinin en önemli tartışmalarından biri de basit ve uygulanabilir metodların hayata geçirilip yapı deprem kapasitesinin ve talebinin belirlenmesi amacı ile doğrusal ve elastik olamayan modellerin analizene olanak sağlamaktır. Araştırmacılar ve mühendisler daha basit doğrusal olamayan statik yöntemleri daha karm aşık zaman-tanım alanındaki çözümlere tercih etmektedirler. Öte yandan doğrusal olmayan statik analiz yöntemlerinde yapıya uygulanan kuvvetler yapı periyodu ve kuvvet dağılımı sabit tutularak yapıldığında hasar gören ve elastic ötesi davranan yapıların analizinde gerçekci çözümler üretemeyebilir. Bu nedenle özellikle son dönemlerde birden fazla modal etkiyi dikkate Alana ve yük dağılımını sabiit kabul etmeyen yöntemler üzerinde çalışmalar yapılmıştır. Nevar ki bu yöntemler yinede kaçınılmaz olarak karmaşık olabilmektedir.

Bu çalışmanın ana amacı yapıların deprem performanslarının belirlenmesinde farklı yöntemlerin karşılaştırılamsıdır. Bu amaç doğrultusunda alçak katlı, orta katlı ve yüksek katlı çerçeve sistemler yapılar tasarlanıp değerlendirilmiştir. Modeller Türk Deprem Yönetmeliğinde belirtilen birinci derece deprem bölgesinde ve Türk Deprem Yönetmeliği,2007 ve TS500-2000 yönetmelikleri kullanılarak düşey ve deprem yüklerine göre tasarlanmıştır.

Deplasmana bağlı Değişken İtme Analizi (DAP) yöntemi, deprem performasının daha sağlıklı belirlenmesi için geliştirilen yöntemlerden biridir. Bu tezde DAP yönteminin güvenli bir şekilde uygulanıp uygulanmayacağının belirlenmesi amacı ile 3, 8 ve 12 katlı yapılar Analiz edilmiştir. Bununla birlikte Artımsal Dinamik Analiz (IDA) yöntemi, toplamda 12 deprem kayıdı kullanılarak çalışılmış ve DAP, FEMA 440 ve IDA yöntemleri karşılaştırılmıştır. Bu amaç için SeismoStruct yazılımı kullanılmış ve elede edilen performans eğrileri ve IDA eğrileri karşılaştırılmıştır. Ayrıca FEMA 440 yöntemine göre performans seviyelerinin belirlenmesinde SAP2000 kullanılmıştır.

Bu çalışmada gösterilmiştir ki DAP yöntemi betonarme yapıların performans değerlendirmelerinde IDA yöntemine alternative olarak daha basit bir yöntem olarak kullanılabilir. DAP yönteminin geleneksel itma analizi yöntemine göre sistematik bir yöntem olamsının yanında daha doğru sonuçlar verdiği ayrıca gözlemlenmiştir.

Anahtar Kelimeler: Çok Modlu İtme Analizini, Artımsal Dinamik Analiz, Deplasmana bağlı Değişken İtme Analizi, ve Statik İtme Analizi.

Dedicated

To my Lovely Father and Mother

To my dearest Brother

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

SDOF	Single Degree of Freedom
MDOF	Multi Degree of Freedom
IDA	Incremental Dynamic Analysis
DAP	Displacement-based Adaptive Analysis
FAP	Force-based Adaptive Analysis
NSP	Nonlinear Static Procedure
NDP	Nonlinear Dynamics Procedure
RC	Reinforced Concrete
GM	Ground Motion
FEM	Finite Element Method
THA	Time History Analysis
FEMA	Federal Emergency Management Agency
PBEE	Performance Based Earthquake Engineering
USGS	U.S. Geological Survey
NEHRP	National Earthquake Hazards Reduction Program
PGA	Peak Ground Acceleration
PGV	Peak Ground Velocity
PGD	Peak Ground Displacement
TEC	Turkish Earthquake Code
SRSS	Square root of sum of squares
CQC	Complete Quadratic Combination

Chapter 1

INTRODUCTION

1.1 Objective

Over the decades, researchers in performance-based earthquake engineering try to develop simple and precise approaches for predicting seismic capacity and demand on structures by taking into account their inelastic behavior. (Chopra A. K., August 2004). The nonlinear static procedure (NSP) has become a popular tool for design verification and performance assessment of structural systems. The usage of NSP method is certainly going to be preferred, among engineers, instead of complicated and impractical methods of nonlinear time-history analysis (NTH) (Pinho R., 2005). The NSP is limited to single-mode response; for that reason, NSP is appropriate for regular low-rise structures where higher mode contributions is not significant. The conventional NSP greatly underestimate the upper stories seismic demands of irregular-plan and high-rise structures because the procedures do not take into account higher modes contributions to the response (Poursha, 2008).

This thesis evaluates a multi-mode nonlinear static analysis method that have proposed by Antoniou and Pinho (2004), which is able to consider higher-modes contributions on structure response. The procedure, which has been named the Displacement-based Adaptive Pushover Analysis (DAP), is applied to three RC frames with different elevations. Therefore, the main objectives of the present work will be to evaluate and compare performances of proposed Adaptive Pushover Analysis (Pinho & Antoniou, 2004b). The accuracy of Adaptive Pushover Analysis methods will be assessed in predicting the global response, through a comparison of Adaptive Pushover Analysis curves with Incremental Dynamic Analysis (IDA) envelopes, as well as limit states capacity of structures. The results indicate that; Adaptive Pushover Analysis has the capacity to efficiently overcome the restrictions of conventional pushover analysis and estimate the limit state capacity and seismic demand of high-rise structures with acceptable accuracy.

1.2 Performance-Based Seismic Evaluation

Earthquake is one of the greatest natural disasters on this planet, which is capable of causing immense loss of life or property damage. Early failure has been observed in buildings that were designed according to modern principles of earthquake designs. The main reasons are because of difficulty in prediction of post-elastic seismic response of structures and lack of information regarding regional seismic hazard due to random nature of earthquakes and accordingly we do not have enough knowledge about structural behavior that subjected to dynamic loads. As a result, the adequacy of seismic codes to inhibit a total or partial collapse of buildings during earthquakes has been questioned. Eventually it has been concluded that the structural behavior and damageability of structures during strong earthquakes is essentially controlled by the inelastic deformation capacities of the ductile structural elements. Accordingly, this means that structural design and seismic assessment should be based on nonlinear deformation, not on the stresses derived by the assumed equivalent lateral loads (Aydınoğlu & Önem, 2010). Regardless of this recognition, many earthquake design codes are still governed on the force-based design. However, researchers attempt to incorporate the displacement-based evaluation and design concept into the seismic engineering practice.

Ghobarah (2001) states that Future seismic design practice will be based on explicit performance criteria that can be quantified, considering multiple performance and hazard levels. Performance-based earthquake engineering is the powerful methods aiming direct design of new buildings and dealing with the seismic performance assessment of pre-designed or existing buildings. Performance-based design has many different interpretations, and according to Applied Technology Council ATC 40 document, performance-based design refers to the methodology in which structural criteria are expressed in terms of achieving a performance objective.

1.3 Seismic Performance Assessment Methods

Seismic performance assessment methods are mainly linear or nonlinear and static or dynamic. As a result, for evaluating seismic demand and capacity of the existing structures four main kinds of analysis methods are available:

- Linear static
- Linear dynamic (Time-history and Response spectrum)
- Non-linear static (Adaptive Pushover analysis)
- Non-linear dynamic (nonlinear time-history and Incremental dynamic analysis)

1.3.1 Linear Static Procedure

Linear Static Procedure (LSP) utilized for force-based evaluation methodology and is the simplest structural analysis method. Seismic structural performance can be assessed by equivalent static analysis. However, the analysis procedures are appropriate for short, regular-plan buildings when higher mode effects are not significant. According to FEMA 356 (2000), while using this method, structures are analyzed and evaluated with linearly elastic damping and stiffness values, at or near yield level. Accordingly, the methods are restricted to 8-story structures without torsional irregularities and total height should not exceeding 25 m.

1.3.2 Linear Dynamic Procedure

When contribution of higher modes on structure response are significant, Linear Dynamic Procedures are appropriate methods and their results are more accurate than linear static procedures. According to FEMA 356 (2000), linear method should be used when buildings modeled with equivalent viscous damping values and linearly elastic stiffness, at or near yield level for this method. FEMA 356 (2000) gives the two methods of Response Spectrum and Time History for LDP but in these methods, linear elastic analysis is utilized to obtain the displacements and internal forces of the system.

1.3.3 Nonlinear Static Procedure (NSP):

Nonlinear static procedure is an approximate structural analysis method in which the structures subjected to ground shaking in a monotonically increasing pattern of lateral forces with a fixed height-wise distribution until the control node reaches to predetermine target displacement (FEMA356, 2000). NSA is a proper method for symmetric-plan low to mid-rise structures for which higher modes contributions are likely to be minimal.

Based on FEMA 356 (FEMA356, 2000), NSP is not just an appropriate method for low-rise structures and is not limited with a single mode response. In fact, FEMA 356 indicates that NSP is an appropriate estimator of capacity demand of high-rise buildings. This is not possible, because the invariant or adaptive lateral load patterns, except SRSS of modal story loads pattern, specified in FEMA 356 are all associated with a single-mode response.

The major drawbacks in this method are (Aydinoglu, 2003),

- Defining the seismic loads through elastic spectral accelerations has no theoretical basis, as they are not consistent with the inelastic deformation of the structure during the pushover process.
- The peak response quantities corresponded with the multi-mode contributions are not able to be properly estimated with a conversion technique based on a single-mode response.

1.3.4 Advantages and Disadvantages of Nonlinear Static Procedures

According to FEMA 440 (2005), Nonlinear static procedures are a reliable assessments tool for estimating of maximum floor and roof displacements but their capability to predict the maximum inter-story drifts of structures are unreliable, particularly for structures that higher modes effects are more significant on them. However, results of inter-story drifts obtained by multi-mode pushover analyses, such as DAP, are more accurate and reliable particularly in high-rise structures. NSPs are also very poor estimators of other response quantities, includes overturning moments and shear forces in structures that higher modes control the structural response.

Mwafey and Elnashai (2000), state that the main usage of the NSP is to predict the seismic capacity of structures and less applicable for prediction of seismic demands particularly when a special ground motion is applied to the structure.

1.3.5 Multi-Mode Pushover Analysis

Single-mode nonlinear static analysis is based on the assumption that the structural response is dominated by fundamental mode and changes of the mode shapes are not considered after structure yields. According to this, the structure subjected to monotonically increasing lateral loads with a fixed height-wise distribution until the control node reaches to predetermine target displacement. But, it has been proved that after yielding occurs; the results of this procedure are not reliable (Rovithakis, Pinho, & Antoniou, 2002). Therefore, the fixed lateral load patterns cannot consider higher modes effect and are not able to redistribute inertia loads due to yielding of the structure. Thus, various multi-mode pushover analysis approaches have been proposed by many researchers to take into account the structural responses in several modes.

In multi-mode pushover analyses, which are described by various investigators, the results are obtained for each mode independently, by applying invariant lateral load distributions that represent structural response in each modes. In each modal pushover analysis, predetermined target displacement is computed from obtained structural response values. Then for each modal pushover analysis, modal structural response quantities are determined separately. Finally, they are combined by applying statistical combination methods (SRSS or CQC). This approach is based on assumption that the lateral load distributions and mode shapes are fixed, although structural response in each mode should be consider as nonlinear. Target displacement values can be calculated by using equivalent linearization approaches (FEMA440, 2005). There have been a great number of studies on this issue and only the ones with the greatest importance are chosen to be referred to in this study.

1.3.5.1 Adaptive Pushover Procedures

Adaptive pushover analysis, proposed by Gupta and Kunnath (2000), is based on an elastic demand spectrum. Accordingly in the proposed method, equivalent seismic loads are computed at each pushover step using the instantaneous mode shapes. The associated elastic spectral accelerations are used for scaling of the seismic loads. Lateral loads are applied to the structure in each mode independently and after scaling the incremental modal responses, finally they combined with statistical rules (SRSS). The two major drawbacks regarding multi-mode adaptive pushover analysis proposed by Gupta and Kunnath (2000) are:

- Loading characteristics based on elastic instantaneous spectral accelerations, associated with the instantaneous free vibration periods, are not compatible with the structural inelastic behavior.
- Conventional pushover curve, obtained by combining multi-mode pushover analyses results, is not able to estimate the peak response quantities properly. (Aydinoglu, 2003).

Alternative pushover procedures [(Elnashai,2001), (Rovithakis, Pinho, & Antoniou, 2002)] have been proposed based on adaptive load patterns after recognizing the fact that conventional pushover methods suffer from some limitations due to implementation of fixed load distributions, that are not consistent with the progressive structural yielding of the elements, and neglecting effects of higher and torsional modes. In adaptive pushover procedure, the story forces are obtained at each modal pushover step applying the interested mode shapes according to the instantaneous stiffness matrix and corresponding elastic spectral Pseudo accelerations. Then lateral force distribution is computed by combing the story forces with a modal combination statistical rule and applied through a single-run pushover analysis. Two critical conclusions can be drawn in this procedure (Aydinoglu, 2003),

- Adaptive load pattern represents the inelastic behavior in a more reliable way compared to fixed load pattern; however it suffers from the same problems that elastic spectral accelerations are not consistent with the instantaneous inelastic response.
- The application of the modal combination in defining the equivalent seismic loads instead of combining the response quantities induced by those loads in individual modes.

Displacement-based Adaptive Pushover (DAP) technique has been proposed by Antoniou and Pinho (2004a, b), in which a set of lateral displacements (rather than forces) is monotonically applied to the structure. The displacement pattern is updated at each step of the analysis, based on the current dynamic characteristics of the structure. The DAP approach represents better performance than the force-based adaptive pushover and has been further investigated that the performance of the DAP method shows significantly improvement in estimation of seismic demand of the structure. (Pinho & Antoniou, 2004b). In chapter 3, represents detailed Displacement-based Adaptive Pushover and its fundamental methodology.

1.3.5.2 Modal Pushover Analysis (MPA)

MPA method proposed by Chopra and Goel (2001) to take to the account higher modes effects in NSA and achieve a notable contribution to the multi-mode pushover analysis. MPA method for estimating peak inelastic structural response to earthquake excitation, summarized in sequence of steps:

(1) Run pushover analysis and plot pushover curves independently for each mode with invariant lateral load patterns associated with the linear (initial) mode shapes,

(2) Convert the pushover curve in each mode to a capacity spectrum of the corresponding equivalent SDOF system using the modal conversion parameters based on the same linear (initial) mode shapes,

(3) Calculate peak inelastic displacement of the equivalent SDOF system in each mode for a given earthquake using the bilinear form of the capacity diagram as a backbone curve (alternatively calculate inelastic spectral displacement using smooth response spectrum – FEMA, 2000),

(5) Calculate peak inelastic response quantities of interest, include story drifts independently in each mode,

(6) Apply SSRS rule to estimate the combined peak response quantities.

The weakest point of MPA procedure is performing the pushover analysis independently in each mode without considering the effect of other modes in the plastic hinge formation. In fact in a frame analysis, different sets of plastic hinges are developed at different locations independently in each mode and generally linear behavior governs even in high-rise structures except in the first few modes (Aydinoglu, 2003). For this reason, MPA provide unacceptable accuracy in plastic hinge rotations, however errors are found relatively smaller in story drifts, because of the participation of the elastic higher modes. This finding led to a questionable suggestion that story drifts could be considered in lieu of the plastic hinge rotations as the representative demand parameter in the acceptance criteria of NSP (Chopra & Goel, 2001). Since the inelastic behavior in higher modes is poorly estimated, a modified version of MPA developed in 2004 in which structural seismic demands were determined by combination of the inelastic response of first-mode pushover analysis with the elastic response of higher modes (Chopra, Goel, & Chintanapakdee, 2004)

1.3.5.3 Incremental Response Spectrum Analysis (IRSA)

Aydinoglu developed a multi-modal IRSA method in 2003. He describes the procedure, in which by performing an incremental pushover analysis, effects of multiple modes are taking into the account. The incremental nature of the analysis allows the effects of softening due to inelasticity in one mode to be reflected in the properties of the other modes. Aydinoglu applied the analysis to a generic 9-story frame model of the SAC building to estimate seismic demand of structure while the gravity loads and P-Δ effects were neglected. After comparing the results with modal pushover analysis (four modes) and nonlinear dynamic analysis, there was a very good agreement for inter-story drift, story shear, floor displacement, floor overturning moment and beam plastic hinge rotation. Despite the good results obtained by this method FEMA 440 (2005) states that Further study is required to establish the generality of the finding and potential limitations of the approach.

The basic steps to be performed at each IRSA stage are described below:

(1) Condense out massless degrees of freedom from the instantaneous (tangent) stiffness matrix modified at the end of the previous pushover step.

(2) Perform free vibration analysis by using Jacobi method (matrix transformation method). Calculate instantaneous eigenvalues with the corresponding eigenvectors and compute the participation factors for the number of modes considered.

$$\Gamma_{xn}^{(i)} = \frac{L_{xn}^{(i)}}{\Psi_n^{(i)T} M \Psi_n^{(i)}}; \quad L_{xn}^{(i)} = \Psi_n^{(i)T} M I_X^g \tag{1.1}$$

Where,

$\Gamma_{xn}^{(i)}$: Instantaneous participation factor for an earthquake in x direction.

$\Psi_n^{(i)}$: Instantaneous n'th mode shape vector

M: Mass Matrix , I_X^g: Kinematic Vector

(3) In each mode, obtain all unit modal response quantities of interest, $\bar{r}_n^{(i)}$, such as bending moments of the potential plastic hinges, $\bar{M}_{jn}^{(i)}$, induced by $\bar{u}_n^{(i)}$ in which,

$$\bar{u}_n^{(i)} = \Psi_n^{(i)} \Gamma_{xn}^{(i)} \tag{1.2}$$

Where,

$\bar{u}_n^{(i)}$: Displacement vector.

(4) Convert each modal capacity diagram to a bilinear diagram according to Figure below and calculate the initial effective period. Skip this stage in the first and second pushover steps (in the first step modal capacity diagrams are linear while in the second they are already bilinear). For each mode calculate spectral displacement either from the solution of Equation 1.3 for a given earthquake using the bilinear modal capacity diagram or from the specified smooth elastic response spectrum using the initial effective period obtained at Stage (4).

$$\Delta\ddot{d}_n^{(i)} + 2\xi_n^{(i)}\omega_n^{(i)}\Delta\dot{d}_n^{(i)} + (\omega_n^{(i)})^2\Delta d_n^{(i)} = -\Delta\ddot{u}_x^{g(i)} \tag{1.3}$$

Where,

$\Delta\ddot{u}_x^{g(i)}$: Ground acceleration increment.

$\xi_n^{(i)}$: Instantaneous modal damping ratio.

$\omega_n^{(i)}$: Instantaneous natural frequency

$\Delta d_n^{(i)}$: Modal displacement increment at the (i)'th incremental step is expressed as,

$d_n^{(i)} = d_n(t) - d_n(t_i - 1)$

In which, $d_n(t)$: Modal displacement in the n'th mode.

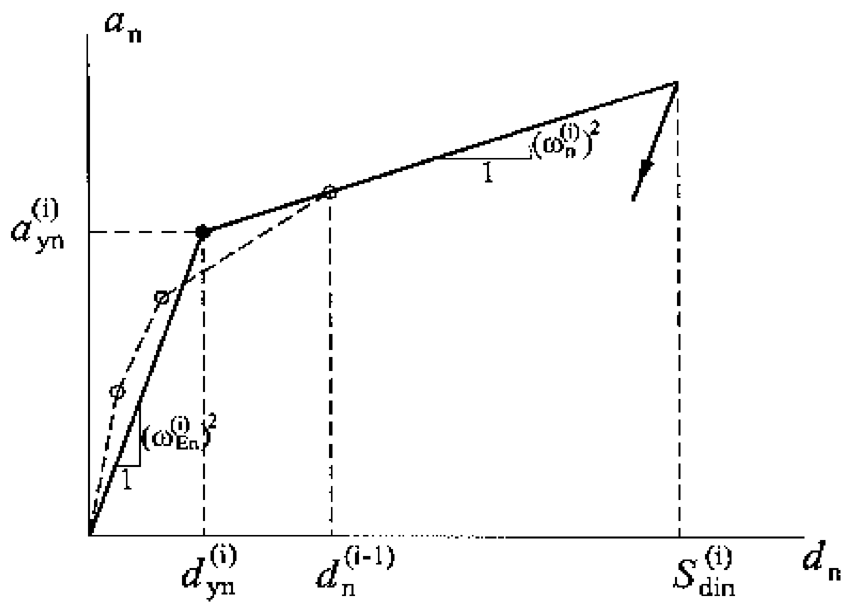


Figure 1.1. Modal capacity diagram Bi-linearization (Aydinoglu, 2003).

(5) Calculate inter-modal scale factors for all modes considered from Equations 1.4 as appropriate. In the practical version of IRSA with initial elastic periods, inter-modal scale factors are calculated from Equation 1.5 only once at the first pushover step and thereafter constantly used in all steps.

$$\lambda_n^{(i)} = \frac{S_{din}^{(i)} - d_n^{(i-1)}}{S_{dil}^{(i)} - d_1^{(i-1)}} \tag{1.4}$$

Where,

$\lambda_n^{(i)}$: Intermodal scale factor

$S_{din}^{(i)}$: Peak inelastic modal displacement

$$\lambda_n^{(i)} = \frac{S_{den}^{(i)} - d_n^{(i-1)}}{S_{del}^{(i)} - d_1^{(i-1)}} \tag{1.5}$$

Where,

$S_{den}^{(i)}$: Elastic spectral displacement of the n'th mode.

$$\lambda_n^{(i)} = \lambda_n^{(1)} = \frac{S_{den}^{(i)}}{S_{del}^{(i)}} \; (i = 2,3,\ldots) \tag{1.6}$$

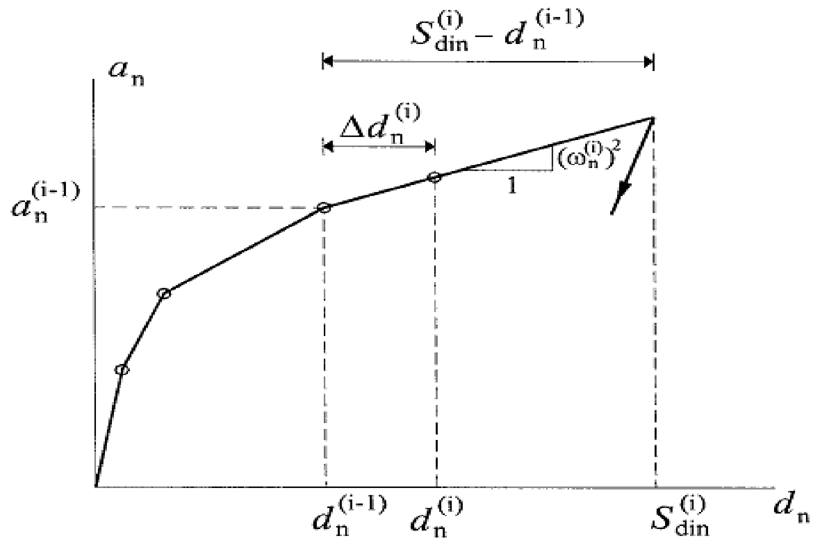


Figure 1.2. Scaling procedure for a modal displacement increment (Aydinoglu, 2003).

(6) Using the information obtained at Stage (3) and (6) calculate combined unit response quantities of interest,$\bar{r}^{(i)}$, such as bending moments of the potential plastic hinges $\bar{M}_j^{(i)}$ from Equations1.7-9, respectively, or from the specified smooth elastic response spectrum using the initial effective period obtained at Stage (4).

$$\bar{r}^{(i)} = \sqrt{\sum_{n=1}^{N_s}(\bar{r}_n^{(i)}\lambda_n^{(i)})^2} \quad (1.7)$$

$$\bar{M}_j^{(i)} = \sqrt{\sum_{n=1}^{N_s}(\bar{M}_{jn}^{(i)}\lambda_n^{(i)})^2} \quad (1.8)$$

$$\bar{r}^{(i)} = \sqrt{\sum_{m=1}^{N_s}\sum_{n=1}^{N_s}(\bar{r}_m^{(i)}\rho_{mn}^{(i)}\bar{r}_n^{(i)})} \quad (1.9)$$

(7) Calculate the first modal displacement increment from Equation 1.10 and locate the plastic hinge yielded at the end of this pushover step. Then, obtain the response quantities of interest from Equation 1.11 and the new coordinates of modal capacity diagram(s) from Equation 1.12-13,

$$\Delta d_1^{(i)} = \frac{M_j^{(y)} - M_j^{(i-1)}}{\bar{M}_j^{(i)}} \quad (1.10)$$

$$r^{(i)} = r^{(i-1)} + \bar{r}^{(i)}\Delta d_1^{(i)} \quad (1.11)$$

$$d_n^{(i)} = d_n^{(i-1)} + \Delta d_n^{(i)} = d_n^{(i-1)} + \lambda_n^{(i)}\Delta d_1^{(i)} \quad (1.12)$$

$$a_n^{(i)} = a_n^{(i-1)} + \Delta a_n^{(i)} = a_n^{(i-1)} + \lambda_n^{(i)}(\omega_n^{(i)})^2\Delta d_1^{(i)} \quad (1.13)$$

(8) Check if the first modal displacement exceeded the first-mode spectral displacement obtained at Stage (5). If exceeded, calculate the peak response quantities from Equations 1.15-16 and terminate the analysis. If not, continue with the next step.

$$\Delta d_n^{(p)} = S_{din}^{(p)} - d_n^{(p-1)} \quad (1.15)$$

$$r^{(i)} = r^{(i-1)} + \bar{r}^{(i)}\Delta d_1^{(i)} \quad (1.16)$$

(9) Considering the last yielded hinge determined at Stage (8), modify the current stiffness matrix and return to Stage (1) for the next pushover step.

1.3.6 Nonlinear Dynamic Analysis

Nonlinear dynamic analyses are able to generating results with high accuracy and also relatively low uncertainty by using the combination of ground motion acceleration (FEMA440, 2005). When the NDP is applied for seismic performance assessment of the structure, a mathematical model directly incorporating the nonlinear load deformation characteristics of individual components and then ground motion time histories, in which represent the severity of earthquake, are applied to the elements of the structure. (FEMA356, 2000). NDA is the most accurate and reliable approach of seismic analysis which in practice it takes too much time and computational efforts. The method has usually been applied only by researchers in the past and the obtained result cannot be used easily in the design practice.

1.3.6.1 Nonlinear Time History Dynamic Analysis Method

This method evaluates structural seismic performance by applying series of ground motions acceleration to structure. In this procedure, ground motion acceleration apply to structure for evaluating the displacement of each frame to estimate the performance limit states possibility for each frame. Three steps are required for selecting the Earthquake records. Firstly, earthquake design response spectrum should be specified according to seismic code related to the location of construction building, then several earthquake ground motion records are selected corresponding to site characteristics and seismic design spectrum. Finally, selected earthquake records are uploaded and then by considering a load case, selected acceleration series are applied to the structure to assess the Seismic Performance of structure (Mezgeen, 2014).

1.3.6.2 Incremental Dynamic Analysis

IDA is a method that estimates the seismic behavior of structure by specifying performance limit-states for a specified structure at a selected site. It fundamentally takes the old concept of scaling accelerogram records and use it in such a way that estimate precisely the full range of structural behavior, from elasticity to collapse. In IDA procedure, a set of chosen ground motion records is applied to a structural model, each of those scaled to multiple levels of intensity (Vamvatsikos & Cornell, 2002). Finally, by summarizing IDA envelops, defining limit-states on them and obtaining the results with fragility curves of probabilistic structural damage, the aims of performance-based earthquake engineering can be reached. In chapter 3, represents detailed incremental dynamic analysis and its fundamental methodology. The main objectives of IDA method are summarized below,

- Better understanding of the structural behavior under strong ground motion levels.
- Predicting the seismic structural capacity level of the structure.
- Comprehensive understanding the range of response or demands against the range of potential levels of a ground motion record.
- Illustrate the dispersion of the structural response nature within increasing of seismic ground motion intensity.
- Derive a multi-record IDA curve to demonstrate stability and variability of different seismic ground motion records.

1.3.6.3 Advantages and Disadvantages of Dynamic Procedures

According to FEMA 440 (2005), great dispersion in engineering demand parameters is resulted by the ground motion variability. FEMA440 illustrate this problem by showing Figure 1.3. The figure demonstrates the results obtained from the research work done by Vamvatsikos & Cornell (2002) in which a series of nonlinear time history analyses perform by setting selected ground shaking that scaled to multiple levels of intensity.

According to FEMA 356, Calculated response can be highly sensitive to characteristics of individual ground motions for nonlinear dynamic procedures.

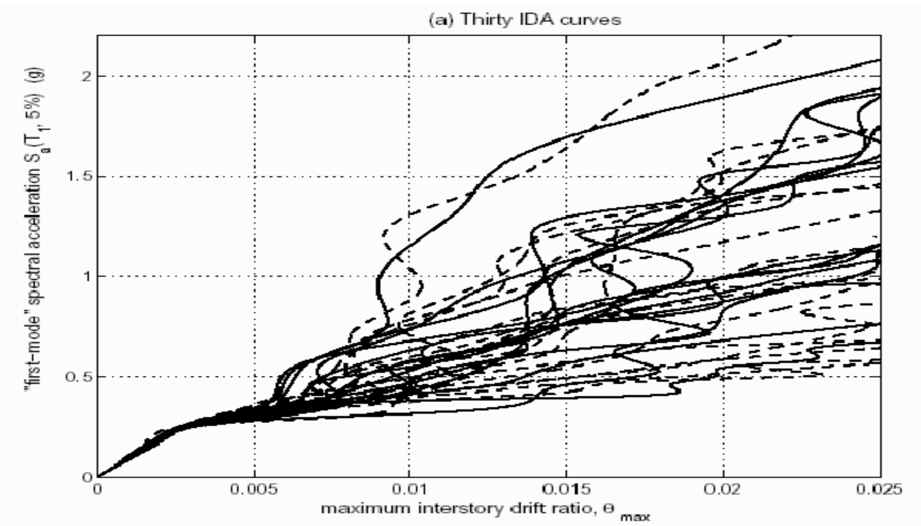


Figure 1.3. IDA Envelope curves study done by Vamvatsikos & Cornell (2002) used thirty ground motion records.

1.3.7 Why nonlinear dynamics analysis cannot be in codes?

It is undeniable that NDA is the most accurate and reliable technique in evaluating structural seismic performance. Instead, static analyses always have major drawbacks such as absence of time-dependent effects. The main reasons that nonlinear dynamics analysis cannot be in codes are described below (Pietra, Antoniou, & Pinho, 2006),

- Earthquake design codes are not a proper guidance on approaches to simulate a set of site-specific acceleration time-series consistent with given code standard response spectrum.
- Despite the improvements in computer processing power, NDA still takes too much time and computational efforts for 3D irregular models with thousands of elements. There are some modelling errors faced by the researcher while evaluation process evolves therefore designers should consider that the dynamics analysis will require to be repeated several times.
- In dynamic analysis output, It is not easy to detect errors in finite elements model, instead errors in pushover analysis tend to be relatively apparent, that is the reason for a first model checking, preliminary simpler analysis, such as pushover analysis, are run.

Therefore, it is concluded that development of nonlinear static analysis approaches need to be continue, so that these analysis methods will become as reliable and accurate as nonlinear dynamics analysis.

1.4 Thesis Outline

This study consists of five chapters and is organized according to the following outline,

Chapter 2 presents an overall summary of the modeling assumptions. Modeling considerations includes structures geometry, sections properties and material characteristics ae also described.

In chapter 3, nonlinear analyses includes Pushover, DAP and IDA has been discussed in detail. The reasons for choosing of the selected Software and codes are also explained. This chapter describes the ways of selecting Ground Motion records and how to generate pushover, IDA and fragility curves for seismic performance assessment of structures.

Chapter 4 concentrates on the analytical results of nonlinear analyses based on methodology described in Chapter 3. Pushover curves obtained by each nonlinear static method are compared with IDA envelopes and their seismic performance assessed.

Chapter 5 presents summary, conclusions and recommendations of the present study.

Chapter 2

DESIGN OF SAMPLE STRUCTURES

2.1 Modeling

2.1.1 Description of structures

Three RC structures, with different elevation, are considered to represent high-rise, medium-rise and low-rise RC structures for this work. The structures have a moment resisting RC elements without any shear walls and are supposed to be located in a high-seismicity region of Turkey. Structures are designed according to TS 500-2000 and Turkish Earthruake Code (2007), taking into account seismic and gravity loads. The sample structures considered in this study are discribed as follows,

- All the floors are the same height of 3 m in elevation.
- The dimensions of structures (width/elevation) used in this study are the same ratio.
- A typical RC structures with high ductility level is considered.
- In order to design structures, Equivalent static analysis, defined by a TEC2007 response spectrum, and fully rigid design method are used.
- The Response Modification Factor for Systems of high Ductility level according to TEC2007 is 8 (Table 5.2, (TEC, 2007)).

  R_x=R_y=8

- Seismic evaluation has been applied according to the Turkish Seismic Code (2007) with Ground Motion Acceleration of 0.4 in zone 1 and soil type Z4 has been used.
- Purpose of occupancy Considered Residential, so Importance Factors is equal to 1 **(**I=1) according to table 2.3 of TSC2007.
- Consider the limitation of relative story drift according to TSC2007.

$$\frac{(\delta_i)_{max}}{h_i} \leq 0.02 \tag{2.1}$$

- The participating live load (30% of live load) and dead load on the structure are 2 $\frac{KN}{m^2}$ and 5$\frac{KN}{m^2}$, respectively.
- Yield strength of the both longitudinal and transverse reinforcements was assumed to be 420 MPa and the characteristic compressive strength of concrete is equal to 25 MPa. In the potential plastic hinge regions, Three layouts with 0.1m, 0.15m, and 0.2m spacings are used for transverse reinforcement. Mechanical properties of used steel in the analyses were selected according to Turkish standard Code (TS500, 2000).

Figure 2.1 represents 3D of the sample models and, plan view of 3-, 8- and 12-story structures are shown in Figure 2.2 .

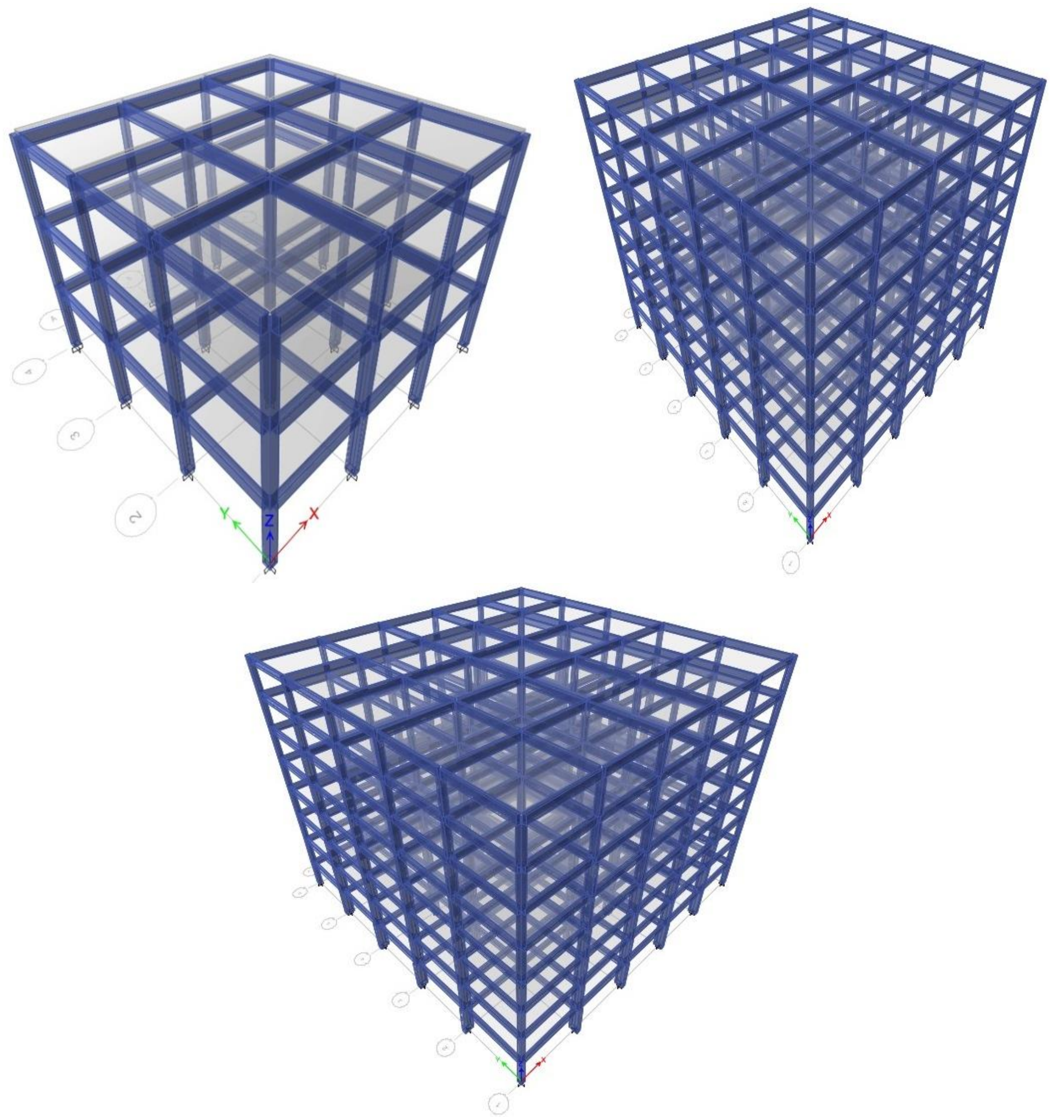


Figure 2.1. 3D models of symmetric-plan 3-story, 8-story and 12-story structure.

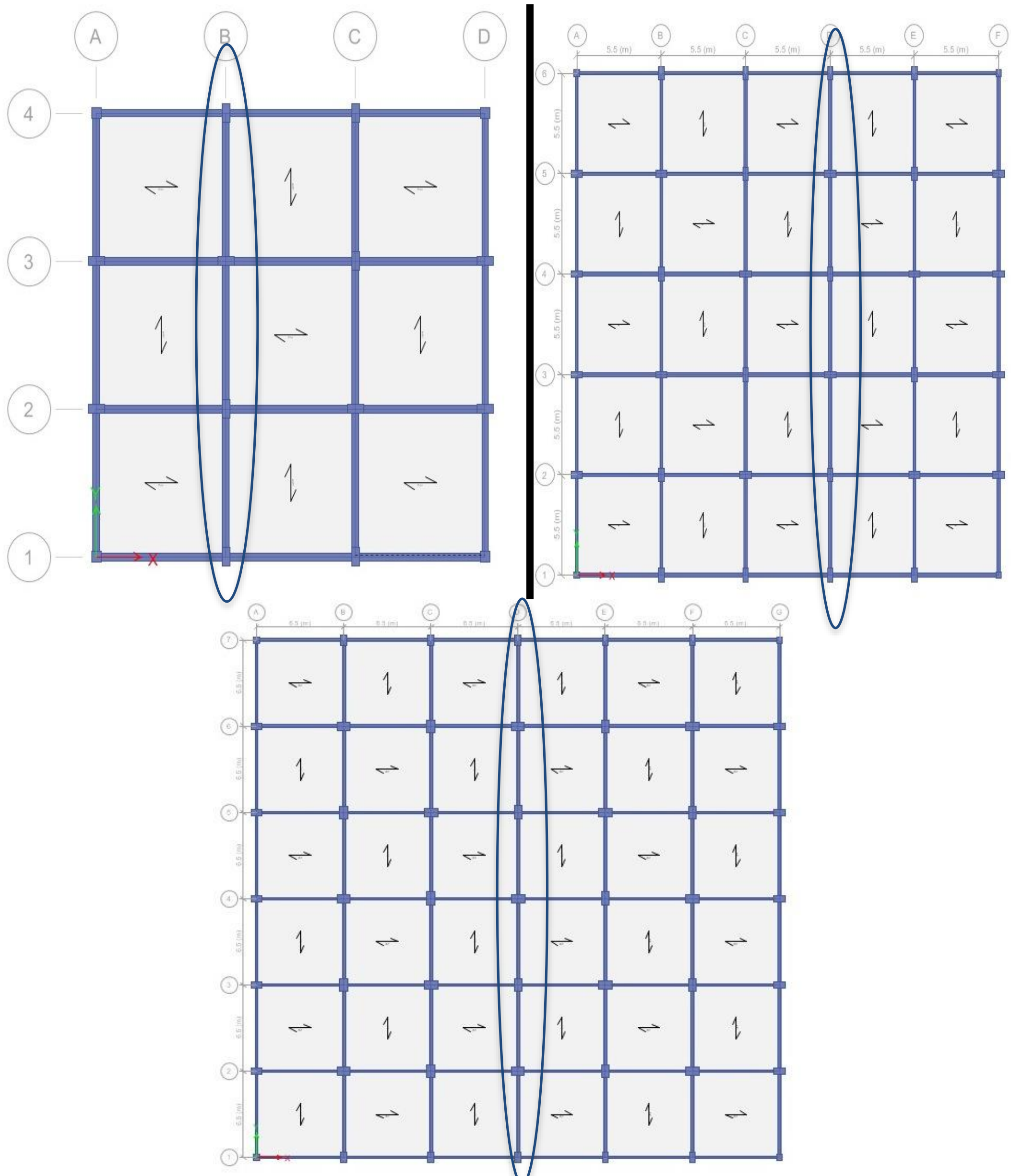


Figure 2.2. 3-story RC structure are 12 m by 12m in plan, 8-story RC structure are 27.5 m by 27.5m in plan and 12-story RC structure are 39 m by 39m in plan.

2.2 Sections Design

2.2.1 3-story structure

The 3-story RC frame is 9 m in elevation and all the floors are the same height of 3 meter. The frame has three bays with 4 meter span length. Longitudinal beam and column reinforcement amount and, column dimensions are demonstrated in Figure 2.3 and Table 2.1, respectively. The section area of all beams are 0.2m×0.5m. The amounts of top and bottom reinforcement (in cm^2) and beam section characterizations are displayed in the elevation view in Figure 2.3 and Table 2.2, respectively.

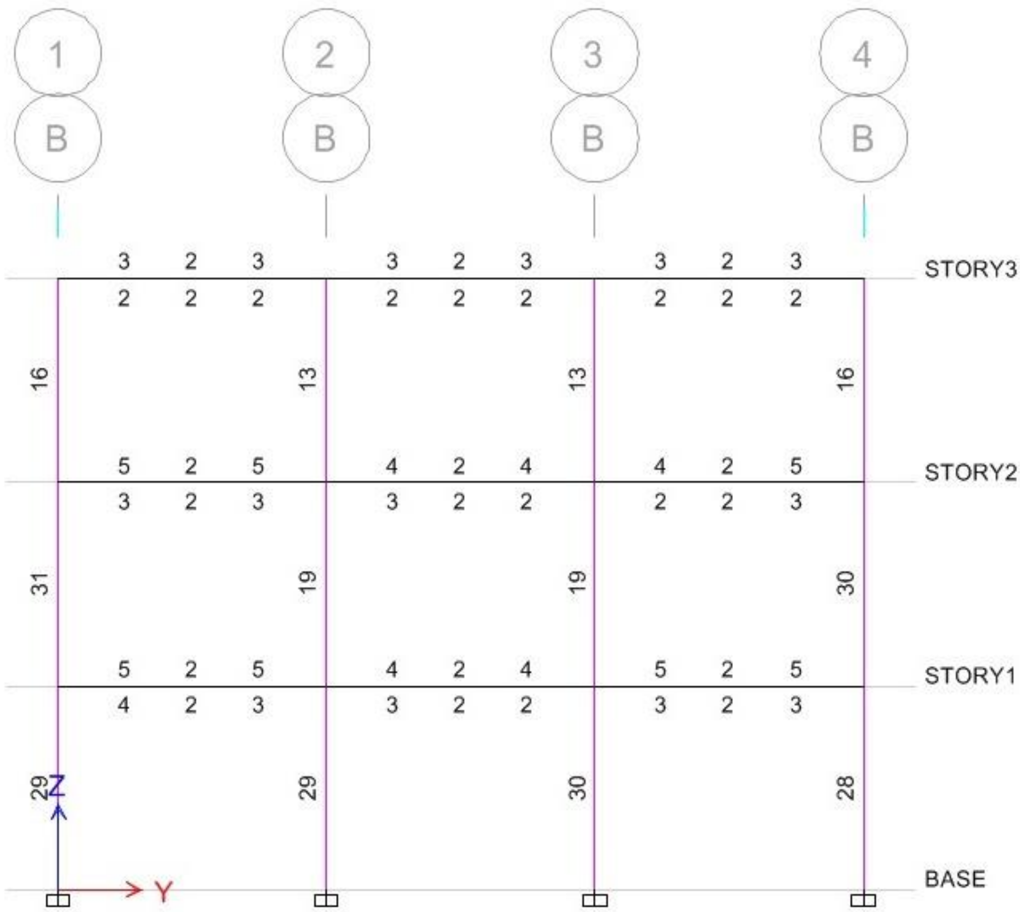


Figure 2.3. Longitudinal beam and column reinforcement amount (cm^2).

Table 2.1. Columns section characterizations.

	Dimension(cm)	REINFORCEMENT	Stirrup
Story1	50x25	10Ø20	Ø8/20
Story2	50x25,40x25	10Ø20, 8Ø18	Ø8/20
Story3	40x25	8Ø18, 8Ø16	Ø8/10

Table 2.2. Beam section characterizations.

	Dimension(cm)	Top	Bottom	Stirrup
3-story	50x20	3Ø18	3Ø18	Ø8/15

2.2.2 8-story structure

The 8-story RC frame is 24 meter in elevation and all the floors are the same height of 3 meter. The frame has five bays with 5.5 meter span length. Longitudinal beam and column reinforcement amount and, column dimensions are demonstrated in Figure 2.4 and Table 2.3, respectively. The section area of all beams are 0.2m×0.6m. The amounts of top and bottom reinforcement (in cm^2) and beam section characterizations are displayed in the elevation view in Figure 2.4 and Table 2.4, respectively.

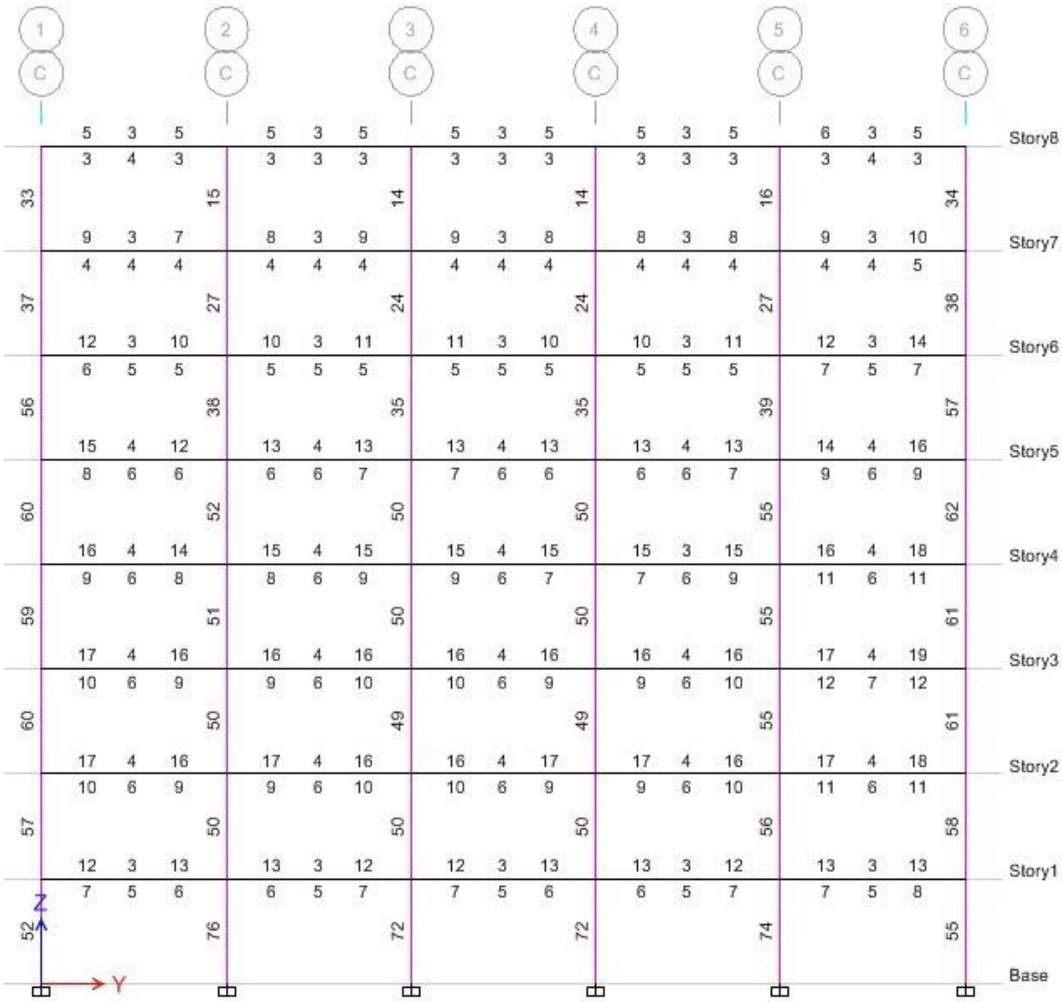


Figure 2.4. Longitudinal beam and column reinforcement amount (cm^2).

Table 2.3. Columns section characterizations.

	Dimension(cm)	REINFORCEMENT	Stirrup
Story1	75x35,80x40	10Ø28, 14Ø28	Ø8/15
Story2	70x35	10Ø28	Ø8/15
Story3	70x35	10Ø28	Ø8/15
Story4	70x30	10Ø28	Ø8/15
Story5	70x30	10Ø28	Ø8/15
Story6	70x30, 50x30	10Ø28, 10Ø25	Ø8/15
Story7	50x25	10Ø22	Ø8/12.5
Story8	50x25, 40x20	10Ø25,8Ø18	Ø8/12.5

Table 2.4. Beam section characterizations.

	Dimension(cm)	Top	Bottom	Stirrup
8-story	60x20	3Ø18+2Ø20	6Ø18	Ø8/15

2.2.3 12-story structure

The 12-story RC frame is 36 meter in elevation and all the floors are the same height of 3 meter. The frame has six bays with 6.5 meter span length. Longitudinal beam and column reinforcement amount and, column dimensions are demonstrated in Figure 2.5 and Table 2.5, respectively. The section area of all beams are 0.2m×0.7m. The amounts of top and bottom reinforcement (in cm^2) and beam section characterizations are displayed in the elevation view in Figure 2.5 and Table 2.6, respectively.

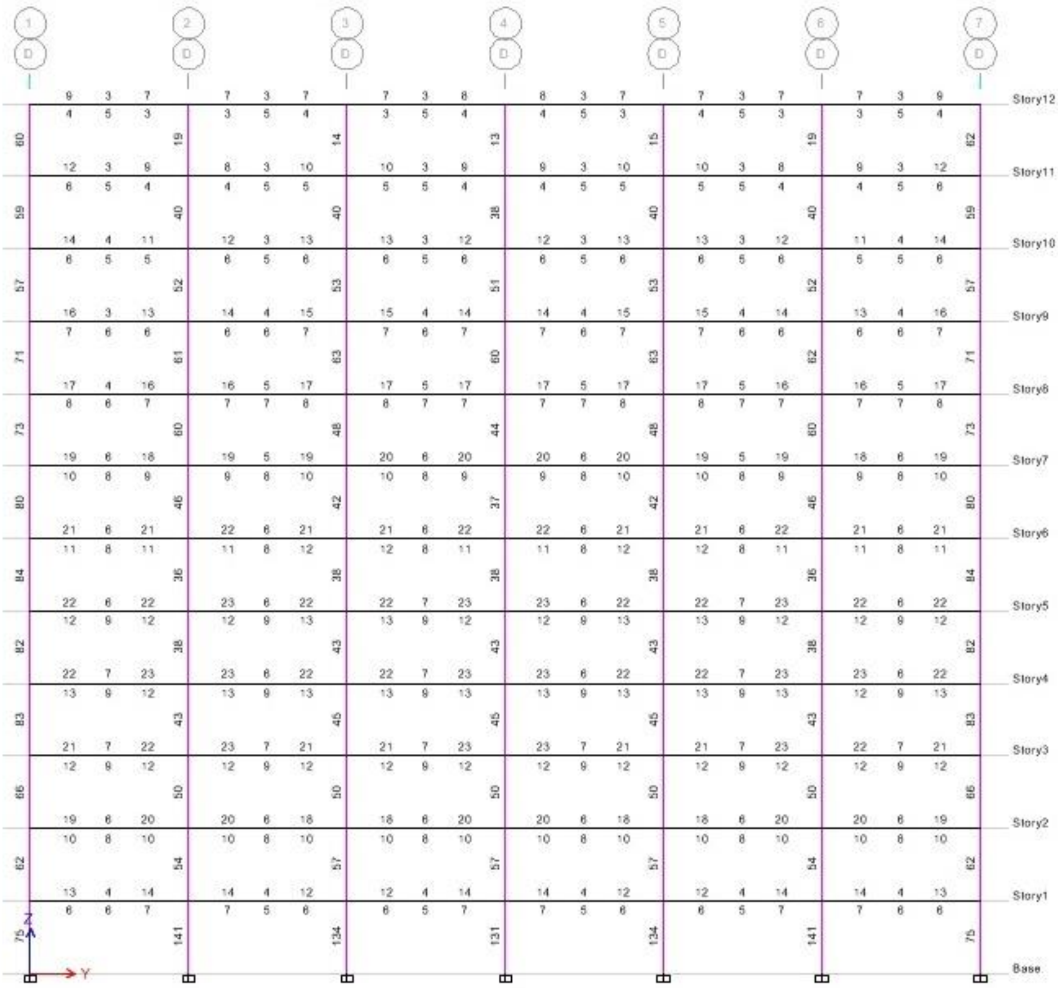


Figure 2.5. Longitudinal beam and column reinforcement amount (cm^2).

Table 2.5. Columns section characterizations.

	Dimension(cm)	REINFORCEMENT	Stirrup
Story1	80x40,95x60, 100x60	14Ø28,18Ø32, 18Ø32	Ø8/15
Story2	80x45, 90x60, 95x60	14Ø25,12Ø25, 12Ø25	Ø8/15
Story3	80x40, 90x55, 90x55	14Ø25,12Ø25, 12Ø25	Ø8/15
Story4	75x40, 85x50, 90x50	14Ø28,12Ø22, 12Ø22	Ø8/15
Story5	75x40, 80x45, 90x45	14Ø28,12Ø22, 12Ø22	Ø8/15
Story6	75x35, 80x45, 85x45	14Ø28,12Ø22, 12Ø22	Ø8/15
Story7	75x35, 70x40, 70x45	12Ø28,12Ø25, 12Ø22	Ø8/15
Story8	75x35, 70x35, 70x35	12Ø28,12Ø28, 12Ø25	Ø8/15
Story9	75x35, 65x30, 70x30	12Ø28,12Ø28, 12Ø28	Ø8/15
Story10	70x30, 60x30, 60x30	10Ø28,10Ø28, 12Ø25	Ø8/15
Story11	70x30, 45x30, 45x30	10Ø28, 8Ø28, 10Ø22	Ø8/15
Story12	70x30, 40x20, 35x20	10Ø28, 8Ø20, 8Ø18	Ø8/15

Table 2.6. Beam section characterizations.

	Dimension(cm)	Top	Bottom	Stirrup
12-story	70x20	8φ20	2Ø20	Ø8/15

Chapter 3

METHODOLOGY

3.1 Target Displacement

According to FEMA356, The target displacement is intended to represent the maximum displacement likely to be experienced during the design earthquake. The appropriate estimation of target displacement point is a very important task in the seismic performance assessment of structures. Equation 3.1 represents a basic relation that is used to calculate the target displacement, δ_t , at each story level (FEMA440, 2005),

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} \tag{3.1}$$

Where,

$\boldsymbol{C_0}$ is Modification factor used to relate spectral displacement of an equivalent SDOF system to the roof displacement of the structure MDOF system obtained applying one of the following methods,

- The easiest way is using Table 3-2 of FEMA 356 to determine the appropriate value of the modification factor.
- The first modal participation factor obtained at the level of the control node. A shape vector consistent to the deflected shape of the structure at the target displacement can be used.

According to FEMA 356, $\boldsymbol{C_1}$ is modification factor to relate expected maximum inelastic displacements to displacements obtained for linear elastic response. FEMA440 states that the modification factor estimates maximum displacements with unacceptable accuracy. FEMA 440 improved the simplified equation 3.2 that can be used for most structures.

$$C_1 = 1 + \frac{R-1}{aT_e^2} \tag{3.2}$$

Where;

T_e: Effective fundamental period of the structure.

a: Constant parameter for different site classes, for instant, a is equal to 60 in site class D.

R: Ratio of elastic strength demand.

$$R = \frac{S_a}{\frac{V_y}{W}} \tag{3.3}$$

Where,

S_a: Response spectrum acceleration, g,

V_y: Yield strength obtained by performing NSP

W: Effective seismic weight of the structure

C_m: Effective mass factor using Table 3-1 in FEMA356.

According to FEMA 356, $\boldsymbol{C_2}$ is modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displace-ment response. FEMA 440 states that this modification factor has limitation in considering the effects of strength degradation and recommend that the C_2 coefficient be as equation 3.4.

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \qquad (3.4)$$

$$\begin{cases} T < 0.2s \rightarrow C_2 = 0.2 \\ T > 0.7s \rightarrow C_2 = 1 \end{cases} \qquad (3.5)$$

According to FEMA 356, $\boldsymbol{C_3}$ is modification factor to represent increased displacements due to dynamic P-Δ effects. Values of C_3 shall be calculated using Equation 3.6-7.

$C_3 = 1 + \frac{|\alpha|(R-1)}{T_e}$, For structures with negative post yield stiffness (3.6)

α: Ratio of post-yield stiffness to effective elastic stiffness, as shown in Figure 3.4.

$\mathrm{C}_3 = 1$

For structures with positive post yield stiffness (3.7)

FEMA 440 eliminate the FEMA 356 coefficient $\boldsymbol{C_3}$. Ratio of elastic strength demand (R), instead of $\boldsymbol{C_3}$, has been suggested by FEMA 440 code for avoiding dynamic instability.

3.2 Computing and defining inelastic frame elements

In this study, for performing nonlinear analysis two different finite element software, SeismoStruct and SAP2000, are selected. Modeling of structure and, defining geometric and material nonlinearities are the main differences between the two selected programs.

3.2.1 Defining inelastic frame elements in SeismoStruct Software

SeismoStruct software is capable of considering both global and local sources of geometric and material nonlinearities. The current unknown deformation of the elements have been described by attaching local chord system to each finite element as shown in Figure 3.1 and it rotates and translates with the element. (SeismoStruct, 2007). Geometric nonlinearities are also available in SAP2000 software for nonlinear time-history analysis and P-delta plus large displacements effects are taken into account by the program.

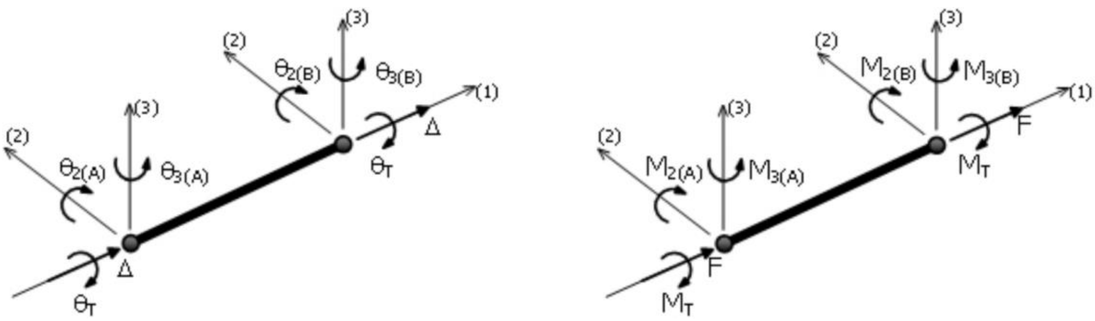


Figure 3.1. Global and local sources of geometric nonlinearities (SeismoStruct, 2007)

For characterizing of the nonlinear response of the system, defining of material elasticity to elements is very important task and it can be defined in elements through distributed and concentrated plasticity. In SeismoStruct software, material inelasticity of the elements is made of so called fiber modeling approach in which the element has been subdivided into many segments. The section is discretized in sufficient quantity of fibres and the response of sections are obtained through the integration single fiber's response of individual fibres (typically 100-150) (SeismoStruct, 2007). SAP2000 software do not take into account the material nonlinearity of the elements.

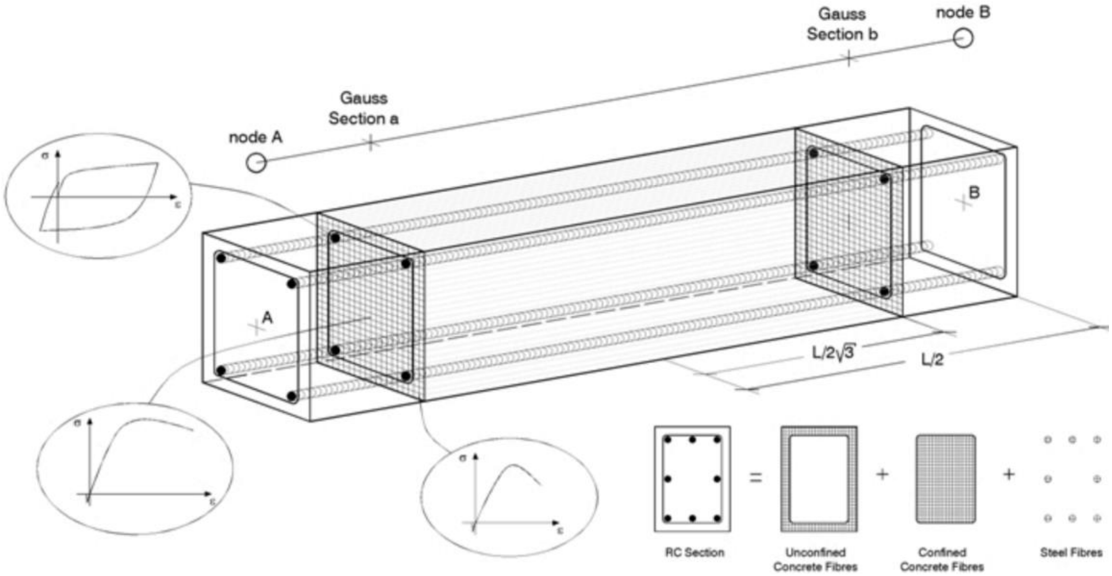


Figure 3.2. Material inelasticity (SeismoStruct, 2007)

In SeismoStruct software, many various material types are available while in SAP2000 program, only elastic material such as orthotropic and isotropic can be used.

3.2.2 Defining inelastic frame elements in SAP2000

In nonlinear analysis, to consider nonlinear structural behavior of each element, plastic hinges are assigned at the both ends of beam–column elements. FEMA (2000c) has given information about nonlinear plastic hinge properties of all of the structural elements in its Tables 6-7 through 6-9 which researchers widely use them.

SAP2000 implements the plastic hinge properties and computes it automatically from section and material properties based on given criteria in ATC-40 or FEMA-356. In SAP2000, three kinds of hinge properties are available. They are User-Defined hinge properties, Auto Hinge Properties, and Program Generated Hinge properties. Inverse of User-Defined hinge properties, Auto hinge properties cannot be modified and viewed because the default properties are section dependent. In user-defined hinge properties, it is required to define moment curvature data for each element and moment rotation relationship for each section.

In program Generated Hinge Properties, the software combines its built-in criteria with the defined section properties for each object to generate the final hinge properties which means that you do considerably less work defining the hinge properties because you do not need to define every hinge. CSI SAP2000 program is able to displays the plastic hinges behavior at each step of the change process (SAP2000, 2006).

Inel and Ozmen (2006) state that for RC buildings, the difference between the results of pushover analysis by using Auto hinge and User-Defined properties is very little for new buildings and more for old ones (more appropriate for rehabilitation objectives). Since the aim of this study is assessing the nonlinear behavior of RC buildings that designed according new design code. M3 and P-M2-M3 interaction hinges were used for defining hinges at the beginning and ending points of the beams and columns, respectively. Therefore, Auto-hinge properties are assigned to the frame elements based on FEMA-356 generalized force-deformation relation model as shown in Figure 3.3.

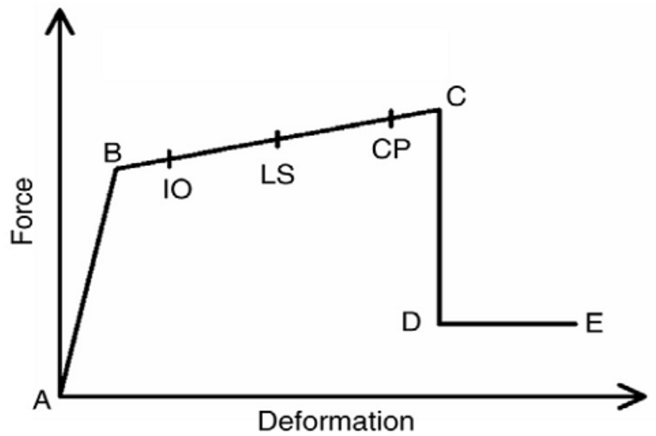


Figure 3.3. The relationship of Force-deformation of a typical plastic hinge (FEMA356, 2000).

3.3 Structural Performance limits states

According to FEMA 356, the discrete Structural Performance Levels, that is going to be studied, are Collapse Prevention (CP), Life Safety (LS) and Immediate Occupancy (IO).

3.3.1 Immediate occupancy (IO)

In this level, the post-earthquake damage should be in the level that the structure remains safe to occupy and stays harmless to inhabit. The basic seismic and vertical force-resisting systems of the structure retain the pre-earthquake design stiffness and strength of the construction. Therefore, Slight damage to the structure has occurred, that can be easily repaired, is observed.

3.3.2 Life Safety (LS)

In this level, the post-earthquake damage should be in the level that the structure has suffered considerable damage, nevertheless retains a margin opposing onset of partial or total collapse. Some elements and components of structure are severely damaged and there is risk of injury to life. It should be possible to repair the structure and repairing may be less economical when compared to complete reconstruction.

3.3.3 Collapse Prevention (CP)

In this level, the post-earthquake damage should be in the level that the building is on the verge of experiencing partial or total collapse. The structure has suffered Comprehensive damage, as well as encompassing momentous degradation in the strength and stiffness of the seismic load resisting system. The construction is not safe for re-occupancy and could not be technically practical to repair.

3.3.4 Collapse (C)

In this level, the post-earthquake damage should be in the level that the structure is at the total collapse and fails to satisfy any criteria that mentioned above, thus this means that re-occupancy of the structure should not permitted because the structure is at the collapse level.

3.3.5 Drift Levels

Inter-story drift profiles were used to achieve valuable results data on the failure mechanism and illustrate influence of yielding derived from the inelastic procedures that is directly correlated to non-structural and structural damage (FEMA440, 2005).

In this work, according to FEMA 356, three limit states including collapse prevention (CP), life safety (LS) and immediate occupancy (IQ) will be defined. For a RC frame without any shear walls, the IQ is defined when inter-story drift ratio reaches 1% of the floor height. Similarly for LS is defined at θ_{max}= 0.02 and finally CP is considered for θ_{max} = 0.04, as shown in Table 3.1 (FEMA356, 2000).

Table 3.1. Structural Performance Levels (FEMA356, 2000).

Elements	**Type**	**CP**	**LS**	**IQ**
Concrete Frames	Drift	4% transient	2% transient	1% transient

3.4 Nonlinear Static Procedures

3.4.1 Introduction

The Nonlinear Static Procedures (NSP) is nowadays generally recognized as appropriate seismic performance and the assessment of existing buildings. The aim of NSP is to estimate seismic demand and capacity of a structure with acceptable accuracy. In the procedure, the structure subjected to ground shaking in a monotonically increasing pattern of lateral forces with a fixed height-wise distribution until a target displacement is reached (FEMA440, 2005). The stages of performing NSA method was summarized by following steps,

1. Model the structures includes of the elements established by computer.
2. Define section material of the elements and their characteristics.
3. Assign beams and columns of the frame, then identify cross sections of elements.
4. Define and assign live and dead forces as a vertical load pattern according to TS500 code. Different types of lateral load pattern distribution, according to Fema356, are applied to the frames.
5. Define plastic hinge properties. Auto-hinge properties are assigned to the frame elements based on FEMA-356.
6. Define the pushover load cases and identify nonlinear static analysis (P-Δ effects are considered by the program).
7. Run the analysis, then the results information is derived and the capacity curve is achieved to assess the seismic performance of the building.

After running traditional pushover analysis, the capacity curve achieved and then the determined target displacement for choses performance level has to be obtain based on FEMA 440.

3.4.2 Methodology

3.4.2.1 Nonlinear Static Analysis Procedures

The procedures could be an appropriate method for performance assessment of buildings that does not have significant higher mode effects, such as short symmetric buildings. Therefore, first a modal response spectrum analysis is executed to capture 90% mass participation by using sufficient modes for evaluating the efficiency of higher modes. A second RSA should also be executed and only the first mode participation shall be considered. If the maximum shear story, obtained by modal RSA, exceeds 130% of the story shear, obtained by RSA in which taking into account only the first mode response, the effects of higher modes in that structure will be significant.

3.4.2.2 Conventional Nonlinear Static Analysis Based on FEMA 356

In **FEMA 356**, two separate NSA are required to be applied. Different load vectors should be applied in each analysis. The larger value of response quantity of the two analyses will be selected to determine acceptability criteria (FEMA356, 2000).

1. First load vector shall be chosen from the below options,
   - First mode: Limited to the structures that their mass participates in first mode are not less than 75 percent.
   - Code distribution: Limited to the structures that their mass participates in first mode are not less than 75 percent, and it is suggested that uniform distribution should be the second load vector.
   - SRSS of modal story loads: If $T_e > 1s$, this option should be applied.
2. A second load vector shall be chosen from the below lists,
   - Adaptive load distribution
   - Uniform distribution

3.4.2.3 Idealizing force-displacement curve

In NSA, the calculated relationship between displacement of a control node and base shear should be idealized to the bilinear curve in order to calculate effective yield strength,V_y the effective lateral stiffness, K_e, and the yield slop, α, , of the structure as illustrated in Figure 3.4. At the origin is starting point of initial linear portion of the idealized force-displacement curve and maximum base shear is the last point of the second linear portion. The effective lateral stiffness, K_e should be considered as the secant stiffness obtained a base shear force equal to 60% of the effective yield strength of the building. The post-yield slope, α, should be identified by a line segment which passes through the substantial curve at the obtained target displacement (FEMA440, 2005).

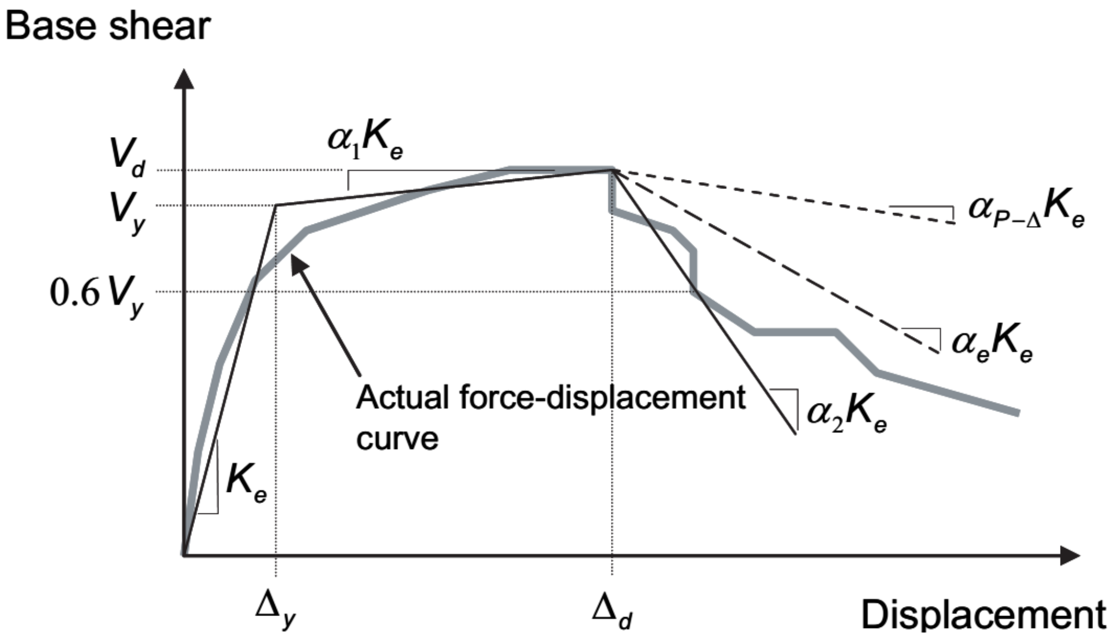


Figure 3.4. Idealized force-displacement curve for NSA (FEMA440, 2005)

3.5 Displacement-based Adaptive Pushover

3.5.1 Introduction

Antoniou and Pinho have proposed a displacement-based adaptive pushover analysis (DAP) in 2004 to take into account the updated loading vector at each analysis step according to current dynamic characteristics of the building. The aim of adaptive pushover analysis is to evaluate the seismic performance of the structure by predicting seismic demands and capacity of a building and considering its dynamic response characteristics includes the effect of the frequency content and deformation of input motion. (Pinho & Antoniou, 2004b).

The lateral load distribution in the adaptive pushover method, continuously updated during the analysis, depending to modal shapes and participation factors obtained by performing eigenvalue analysis at each step of analysis. DAP is fully multi-modal method that take into account the modification of the inertia forces, the structural stiffness softening, and its period elongation due to spectral amplification (Antoniou & Pinho, 2004a).

DAP has the capability to update and change the horizontal load distributions based on the constantly changing modal properties of the structure and solves the drawback of fixed-load pattern of the pushover analysis, providing a more accurate tool for assessing structural performance and better response estimator than conventional pushover methods, particularly, in structures that the effects of higher modes play a major role in its dynamic structural response.

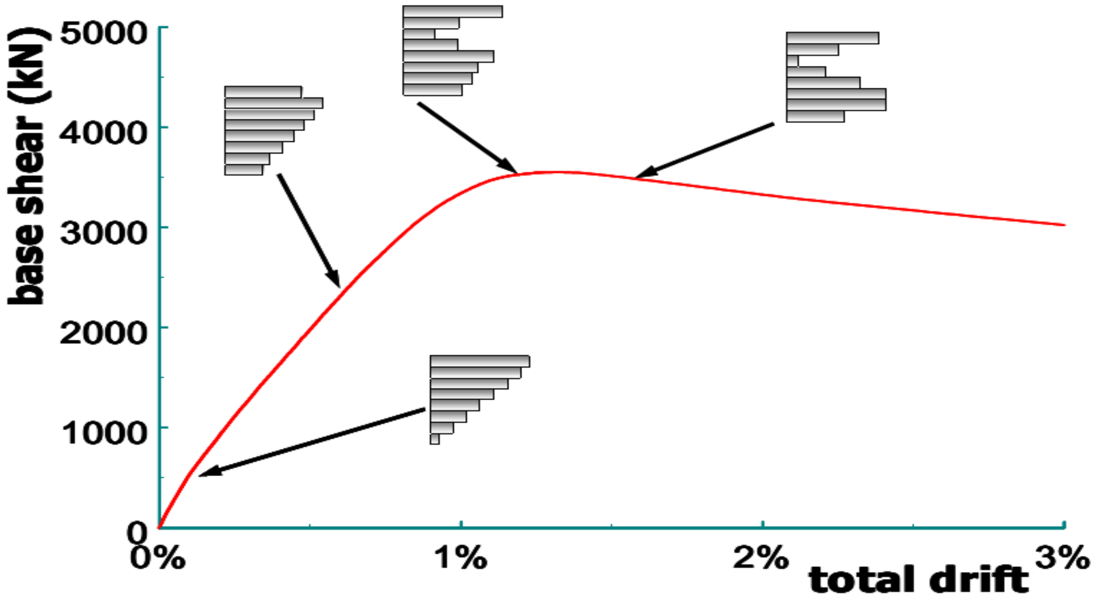


Figure 3.5. Shape of updated loading vector at each analysis step in Adaptive pushover. (Pietra, Pinho, & Antonio, 2006).

3.5.2 Methodology

The analysis stages of the Displacement-base Adaptive Pushover method, described in greater detail in the study of Pinho and Antoniou [2004], are as follows,

1. Perform eigenvalue analysis taking into account the structural stiffness softening at the end of the last load step without applying any additional force. Then, compute eigenvectors and periods of the system. Preferably use Lanczos method for this purpose (Hughes, 1987).
2. According to the participation factors and the modal shapes of the Eigensolution, the patterns of the displacements are obtained for each mode separately. For considering corresponding value for each vibration mode in the calculation of the load pattern, a particular spectral shape should be defined.
3. Using statistical rules method (SRSS or CQC) to combine the lateral load profiles of the modes. The absolute values of story forces are obtained by the nominal loads and the load factor λ, because only the relative story loads values

are interested. Horizontal loads are normalized according to the maximum value for DAP.

4. Increase the load factor λ to scaled-up story loads. The story forces are obtained by the nominal load at that story, updated load factor and the displacement pattern calculated above (typically, the nominal loads are equivalent at all stories). Incremental scaling can also be employed, whereby only the load increment is updated and added to the load already applied to structure throughout the last increments.
5. Apply the new obtained loads to the structure and then determine the response of the structure by solving the system of equations at the new equilibrium state.
6. Update and compute the matrix of tangent stiffness of the system and return to the first stage of the algorithm, for the next increment of the DAP.

3.5.3 Choice of the Software for Computer Analysis (SeismoStruct software)

SeismoStruct software is a fiber-based Finite Elements program for seismic analysis of the buildings. The software is able to evaluate structural performance under dynamic or static loading, taking into the account geometric nonlinearities as well as material inelasticity (SeismoStruct, 2007). The accuracy of the program in assessment structural performance has been proved by comparing its results with experimental results obtained by pseudo-dynamic tests [(Casarotti & Pinho, 2006), (Elnashai & Pinho, 2000)]. Moreover, the algorithm in section 3.5.2 is available in the software and has been implemented effectively that will be used to perform the DAP method. Antoniou and Pinho (2002), state that The main advantage of the proposed algorithm is that it permits the application of the exact force profiles calculated by modal analysis at every step, without stability or performance problems.

3.6 Incremental Dynamic Analysis (IDA)

3.6.1 Introduction

Incremental Dynamic Analysis (IDA) is a parametric structural analysis approach that has proposed to predict seismic behavior of structures under strong ground motion. IDA is able to estimate limit-state capacity and seismic demand by executing a series of nonlinear time history analyses under a suite of multiple scaled ground motion records. Selected ground motion intensity, for evaluating seismic capacity, is incrementally increased until structural capacity reach to the global collapse. Vamvatsikos (2002) states that IDA has significant potential and is not just a solution for performance based earthquake engineering. In other words, it has the capability to extend far beyond that and give more accurate prediction about structural behavior under seismic load to researchers. IDA method basically takes the old concept of scaling ground motion records and develops it into a way to accurately describe the full range of structural behavior, from elasticity to collapse. IDA is widely applicable method and a multi-purpose tool for assessing structural performance which can accurately predict the responses of structures under a wide range of intensities.

IDA is one of the well-known approaches to evaluate the structural performance level under a suite of seismic ground motions. IDA is able to estimate limit-state capacity and seismic demand by performing a series of nonlinear time history analyses under a suite of multiple scaled accelerogram records of earthquake ground motion acceleration. In IDA method, the intensity of selected ground motion is incrementally increased until the intended limit state seismic capacity of the global structural system is achieved. Besides, it contains plotting an intensity measure (i.e. first mode spectral acceleration,S_a) versus a damage measure (maximum inter-story drift ratio).

Moreover, fragility curves have been also derived by IDA method in which demonstrate expected damage in terms of Collapse Prevention, Life Safety and immediate occupancy as a function of the chosen ground motion intensity. The multi-record IDA curves have been summarized into their 16%, median and 84% fragility curves and then limit states capacity at each performance capacity level have been obtained. Based on obtained result, probabilistic structural damage (fragility curves) are estimated in terms of maximum inter-story drift ratio and PGA for predefined limit states at each performance capacity level. SeismoStruct software program was used in order to run IDA.

3.6.2 Methodology

In this study, IDA method has been performed on 3, 8 and 12-story RC frame building which is designed according to the TS500-2000 design code. Then, twenty earthquake ground motion records have been selected and subjected to RC frames. Accordingly, Nonlinear Dynamic Time History Analysis has been applied under a set of ground motions scaled to a specific level of intensity and then IDA curves for each dynamic analysis have been derived. Consequently, by summarizing the multi record IDA into their 16%, median and 84% percentiles, fractile values are obtained. In this work, the maximum inter-story drift ratio of the structure has been considered as a damage measure (DM) and 5% damped first mode spectral acceleration as an intensity measure (IM).

3.6.3 Selected Ground Motions

For performing IDA, a series of nonlinear dynamic time history analysis has applied to structure under a suite of multiple scaled ground motion records. The task of selecting and scaling a proper real set of ground motion is very important for seismic design and analysis and also this is a complex task because each of them has differences in their characteristics and accordingly their effects on structural response will be different.

Moreover, the accuracy of IDA results are depends on the number of chosen accelerogram records. According to research performed by Shome and Cornell (1999), it is usually enough to select ten to twenty accelerogram records to estimate limit-state capacity and seismic demand of structures with sufficient accuracy. All the twenty selected ground motion records data were taken from NGA STRONG MOTION RECORD database of Pacific Earthquake Engineering Research (PEER) center, (PEER, 2010).

Some limitations in selecting of ground motion records were imposed to balance selection of large motions and to insure that they are strong enough to cause structural damage and collapse. The limitations in selecting of ground motion records are described as follows,

- Lowest moment magnitude of earthquake should be 6.5.
- AGA and PGV should be greater than 0.2g and15 cm/sec, respectively.
- Source to site Distance should be at least 10 km.

- The recommended lowest usable frequency for the record should be less than 0.25Hz, to ensure that the low frequency content was not removed by the ground motion filtering process
- All NEHRP soil type of selected ground motion records happened to be on C and D sites categories. Soil shear wave velocity should be greater than 180m/s in upper 30m of soil.
- Spectral shape is not considered in selecting of records
- Ground motion records were chosen to be free-field without any consideration of station housing.
- Selected Fault Mechanism in all records are Strike-slip to be in consistent with Turkey.

Twenty ground motions records were selected by considering the restrictions described above. The next step is to apply these records to the RC Frames in order to determine maximum inter-story drift ratio of the systems and finally drawing IDA envelopes.

3.6.4 SeismoStruct Software

One type of analysis that can be run directly in Seismostruct program is incremental Dynamics analysis. For performing IDA, the user is asked to enter the Incremental Scaling Factors in the first step and then requires to define the time history curve (usually a natural or artificial accelerogram) and corresponding curve multiplier (scaling factor) (SeismoStruct, 2007).

Table 3.2. List of earthquake ground motions (PEER, 2010)

		Even Information				Site Information				Record Information		
EQ Index	**Peer NGA Rec. Num.**	**Event**	**Year**	**Fault Type**	**Mag.**	**Station Name**	**Vs-30 (m/s)**	**Fault Distance**	**NEHRP Soil Type**	**Lowest Usable Freq. (Hz)**	**PGV**	**PGA**
1	1602	Duzce, Turkey	1999	Strike-Slip	7.14	Bolu	326	12.2	D	0.06	56.4	0.728
2	1787	Hector Mine	1999	Strike-Slip	7.13	Hector	684.9	11.2	C	0.04	28.6	0.266
3	169	Imperial Valley	1979	Strike-Slip	6.53	Delta	274.5	22.285	D	0.06	26.0	0.238
4	174	Imperial Valley	1979	Strike-Slip	6.53	El Centro Array #11	196.2	13	D	0.25	34.5	0.364
5	177	Imperial Valley	1979	Strike-Slip	6.53	El Centro Array #2	188.8	14.33	D	0.125	31.5	0.315
6	189	Imperial Valley	1979	Strike-Slip	6.53	SAHOP Casa Flores	338.6	10.215	C	0.25	19.6	0.287
7	162	Imperial Valley	1979	Strike-Slip	6.53	Calexico Fire Statio	231.2	11	D	0.25	21.2	0.275
8	1111	Kobe, Japan	1995	Strike-Slip	6.9	Nishi-Akashi	609	16.15	C	0.12	37.3	0.509
9	1116	Kobe, Japan	1995	Strike-Slip	6.9	Shin-Osaka	256	23.8	D	0.12	37.8	0.243
10	1158	Kocaeli, Turkey	1999	Strike-Slip	7.51	Duzce	276	14.5	D	0.24	58.8	0.312
11	1148	Kocaeli, Turkey	1999	Strike-Slip	7.51	Arcelik	523	12.05	C	0.09	17.7	0.218
12	900	Landers	1992	Strike-Slip	7.28	Yermo Fire Station	353.6	23.7	D	0.07	51.5	0.245
13	848	Landers	1992	Strike-Slip	7.3	Coolwater	271	19.85	D	0.13	25.6	0.283
14	752	Loma Prieta	1989	Strike-Slip	6.93	Capitola	288.6	22.1	D	0.25	36.5	0.529
15	767	Loma Prieta	1989	Strike-Slip	6.9	Gilroy Array #3	349.9	12.5	D	0.12	35.7	0.555
16	778	Loma Prieta	1989	Strike-Slip	6.9	Hollister Diff. Array	215.5	24.67	D	0.125	43.9	0.269
17	1634	Manjil, Iran	1990	Strike-Slip	7.37	Abbar	724.5	12.8	C	0.25	55.3	0.209
18	721	Superstition Hills	1987	Strike-Slip	6.54	El Centro Imp. Co.	192.1	18.35	D	0.125	40.9	0.258
19	721	Superstition Hills	1987	Strike-Slip	6.54	El Centro Imp. Co.	192.1	18.35	D	0.125	46.4	0.358
20	725	Superstition Hills	1987	Strike-Slip	6.54	Poe Road (temp)	207.5	11.45	D	0.25	35.7	0.446

3.6.5 Summarizing the IDA Curves

It is essential to summarize generated IDA Curves for defining limit states on them since each IDA curve contains large variability of records, a huge amount of data and a wide range of behavior. Therefore, it is needed to utilize a proper summarization method to reduce this huge amount of data. IDA envelope curve can be summarized, by many easy techniques, a measure of dispersion (e.g., the difference between two fractiles, or the standard deviation) and into central value (median). Vamvatsikos (2002) has chosen the cross sectional fractiles technique to summarize IDA curves in his study. Based on above discussion, I have summarized IDA curves based on calculation the 84%, median and 16% fractile values of IM (IM84% , IM50% ,IM16% respectively) and DM (DM84%, DM50%,DM16% , respectively) for each limit-state, as shown in Table 4.10.

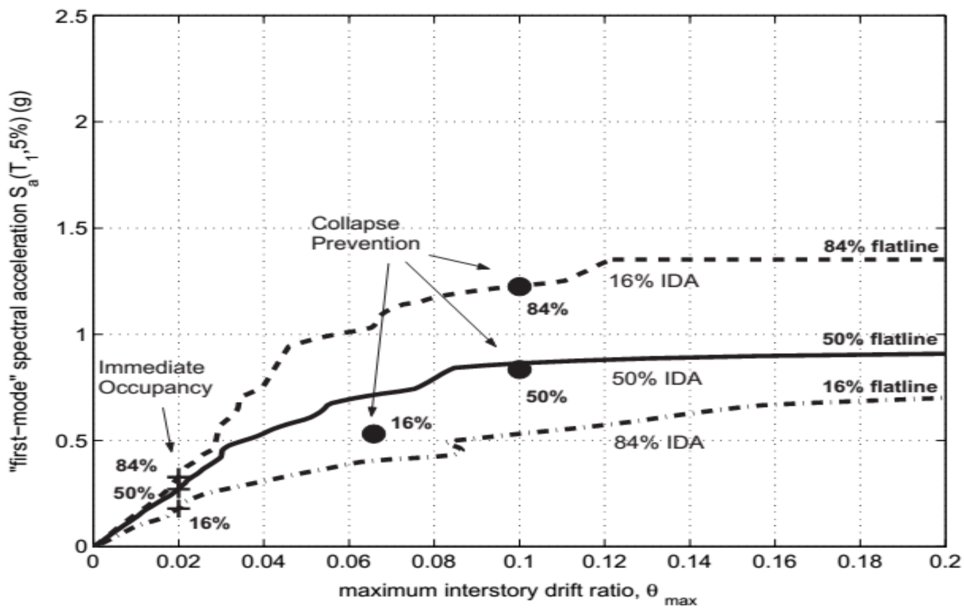


Figure 3.6. IDA envelopes has been summarized into their 16%, median and 84% fractiles (Vamvatsikos & Cornell, 2002).

Chapter 4

RESULTS AND DISCUSSIONS

4.1 3-Story Frame

4.1.1 Target displacement of 3-Story Frame

Table 4.1. FEMA440 parameters for 3-story frame model.

Item	Value	Item	Value	Item	Value
C_0	1.298	T_i	0.412	V_y	145.118
C_1	1.202	K_i	6660.229	D_y	0.023
C_2	1.033	K_e	6308.367	Weight	460.066
S_a	1.000	Alpha	0.224	C_m	1.000
T_e	0.423	R	3.171		

The target displacement,δ_t, of 3-story RC frame are calculated in accordance with Equation 3.1 and obtained parameter shown in Table 4.1, as specified in section 3.1.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} = 0.071\ m$$

4.1.2 Performance limit states of 3-Story Frame

Table 4.2. Pushover steps for 3-story frame model.

Step	**Displacement** **m**	**Base Force** **kN**	**Drift Ratio** **%**
1	0.012	77.638	0.13
2	0.028	147.614	0.315
3	0.047	188.359	0.53
4	0.071	215.708	0.79
5	0.106	249.517	1.176
6	0.115	255.212	1.281
7	0.151	263.41	1.681
8	0.173	268.299	1.92
9	0.173	268.299	1.92

Pushover analysis steps are shown in Table 4.2 applying 1st mode lateral load for the 3-story frame model via CSI SAP2000. The obtained results demonstrate that the 3-story RC frame starts to yield at 0.023 m top displacement. At step 4, Plastic hinges for beams change their performance level to immediate occupancy and at this level the structure has 0.071 m top displacement. According to FEMA 356, the post-earthquake damage should be in the level that the structure remains safe to occupy and stays harmless to inhabit, and slight damage to the structure has occurred, that can be easily repaired. In the next step, top displacement is reached to 0.151m and the performance level of beams raise and change to life safety. In this level, some structural components and elements has suffered comprehensive damage and there is risk of injury to life. Structure collapse at step 8 where top displacement is equal to 0.172m. Table 4.3

shows limit states capacities of 3-story RC frame. Figure 4.1(a) shows Base shear versus Top Displacement (Pushover Curves) of 3-story RC frame and its Idealized force-displacement curve.

Table 4.3. The performance levels of 3-Story Frame

Performance Level	Start (m)	End (m)
Yield	0.023	0.071
IQ	0.071	0.115
LS	0.115	0.172
CP	0.172	-

According to FEMA 440 procedure, the target displacement is equal to 0.071 m in 3-story RC frame. The frame yields at 0.023 m and the obtained top drift ratio of the RC frame is 0.79%. Based on the target displacement, the largest plastic hinge is at immediate occupancy level. Thus, 3-story RC frame under 1st mode lateral load expected to have slight damage that can be easily repaired

4.2 8-Story Frame

4.2.1 Target displacement of 8-Story Frame

Table 4.4. FEMA440 parameters for 8-story frame model.

Item	Value	Item	Value	Item	Value
C_0	1.332	T_i	0.993	V_y	743.092
C_1	1.060	K_i	8506.897	D_y	0.087
C_2	1.000	K_e	8506.897	Weight	3639.137
S_a	0.926	Alpha	0.116	C_m	1.000
T_e	0.993	R	4.533		

The target displacement, δ_t, 8-Story Frame are calculated in accordance with Equation 3.1 and obtained parameter shown in Table 4.4, as specified in section 3.1.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} = 0.318\ m$$

4.2.2 Performance limit states of 8-Story Frame

Table 4.5. Pushover steps for 8-story frame model

Step	Displacement m	Base Force kN	Drift Ratio %
1	0.057	485.194	0
2	0.082	654.187	0.238
3	0.115	769.869	0.341
4	0.216	887.837	0.481
5	0.318	970.292	0.9
6	0.32	971.753	1.325
7	0.404	1024.907	1.332
8	0.449	1040.195	1.684
9	0.449	1040.197	1.872

Pushover analysis steps are shown in Table 4.5 applying uniform lateral load for the 8-story frame model via CSI SAP2000. The obtained results demonstrate that the 8-story RC frame starts to yield at 0.087 m top displacement. At step 4, Plastic hinges for beams change their performance level to immediate occupancy and at this level the structure has 0.26 m top displacement. According to FEMA 356, the post-earthquake damage should be in the level that the structure remains safe to occupy and stays harmless to inhabit, and slight damage to the structure has occurred, that can be easily repaired. In the next step, top displacement is reached to 0.404m and the performance level of beams raise and change to life safety. In this level, some structural components and elements has suffered comprehensive damage and there is risk of injury to life.

Structure collapse at step 8 where top displacement is equal to 0.44 m. Table 4.6 shows limit states capacities of 3-story RC frame. Figure 4.1(b) shows Base shear versus Top Displacement (Pushover Curves) of 3-story RC frame and its Idealized force-displacement curve.

Table 4.6. The performance levels of 8-Story Frame

Performance Level	Start (m)	End (m)
Yield	0.087	0.264
IQ	0.264	0.404
LS	0.404	0.438
CP	0.438	-

According to FEMA 440 procedure, the target displacement is equal to 0.318 m in 8-story RC frame. The frame yields at 0.0874 m and the obtained top drift ratio of the RC frame is 0.9%. Based on the target displacement, the largest plastic hinge is at immediate occupancy level. Thus, 8-story RC frame under uniform lateral load expected to have slight damage that can be easily repaired.

4.3 12-Story Frame

4.3.1 Target displacement of 12-Story Frame

Table 4.7. FEMA440 parameters for 12-story frame model.

Item	**Value**	**Item**	**Value**	**Item**	**Value**
C_0	1.296	T_i	1.450	V_y	872.411
C_1	1.000	K_i	9889.537	D_y	0.091
C_2	1.000	K_e	9609.151	Weight	9306.740
S_a	0.680	Alpha	0.094	C_m	1.000
T_e	1.471	R	7.254		

The target displacement, δ_t, 12-Story Frame are calculated in accordance with Equation 3.1 and obtained parameter shown in Table 4.7, as specified in section 3.1.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} = 0.47$$

4.3.2 Performance limit states of 12-Story Frame

Table 4.8. Pushover steps for 12-story frame model

Step	Displacement m	Base Force kN	Drift Ratio %
0	0	0	0
1	0.0429	424.175	0.119
2	0.0683	640.993	0.189
3	0.123	866.542	0.341
4	0.241	1038.191	0.668
5	0.440	1198.216	1.222
6	0.529	1242.221	1.470

Pushover analysis steps are shown in Table 4.8 applying uniform lateral load for the 12-story frame model via CSI SAP2000. The obtained results demonstrate that the 12-story RC frame starts to yield at 0.091 m top displacement. At step 4, Plastic hinges for columns change their performance level to immediate occupancy and at this level the structure has 0.24 m top displacement. According to FEMA 356, the post-earthquake damage should be in the level that the structure remains safe to occupy and stays harmless to inhabit, and slight damage to the structure has occurred, that can be easily repaired. In the next step, top displacement is reached to 0.44m and the performance level of columns raise and change to life safety. In this level, some structural components and elements has suffered comprehensive damage and there is risk of injury to life. Structure collapse at step 6 where top displacement is equal to 0.53m. Table 4.9 shows limit states capacities of 3-story RC frame. Figure 4.1(c) shows Base shear versus Top Displacement (Pushover Curves) of 3-story RC frame and its Idealized force-displacement curve.

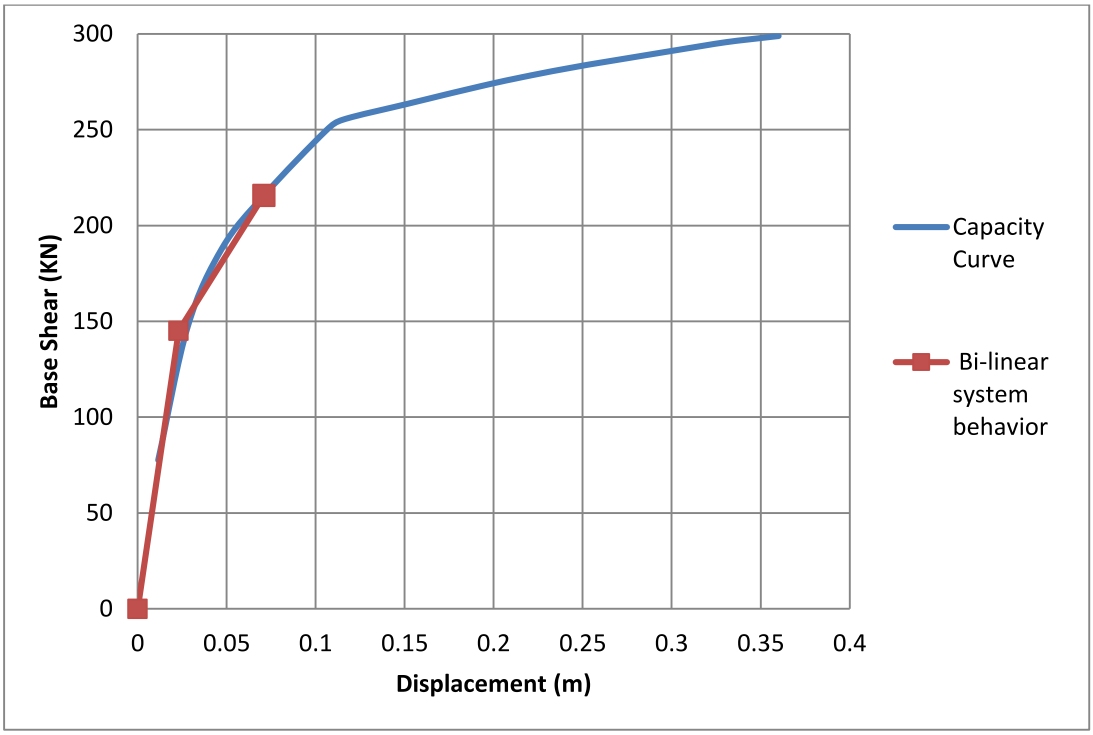
Table 4.9. The performance levels of 12-Story Frame

Performance Level	**Start (m)**	**End (m)**
Yield	0.091	0.241
IQ	0.241	0.440
LS	0.440	0.529
CP	0.529	-

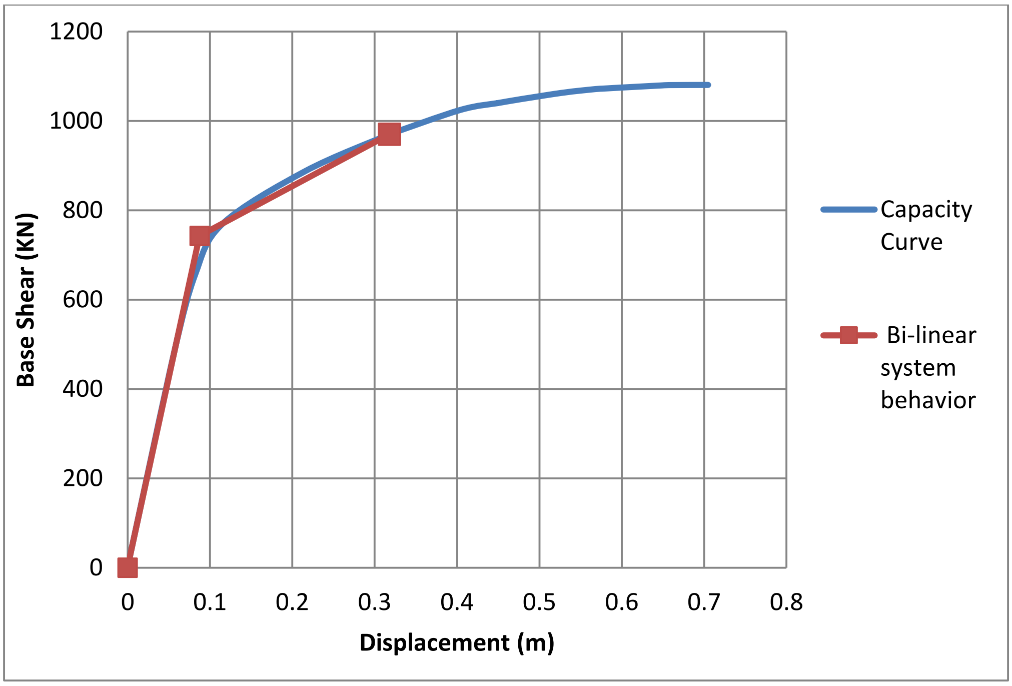
According to FEMA 440 procedure, the target displacement is equal to 0.47 m in 8-story RC frame. The frame yields at 0.091 m and the obtained top drift ratio of the RC frame is 0.9%. Based on the target displacement, the largest plastic hinge is at life safety level. Thus, 8-story RC frame under uniform lateral load expected to have comprehensive damage and there is risk of injury to life.

4.4 Capacity curve of conventional pushover analysis

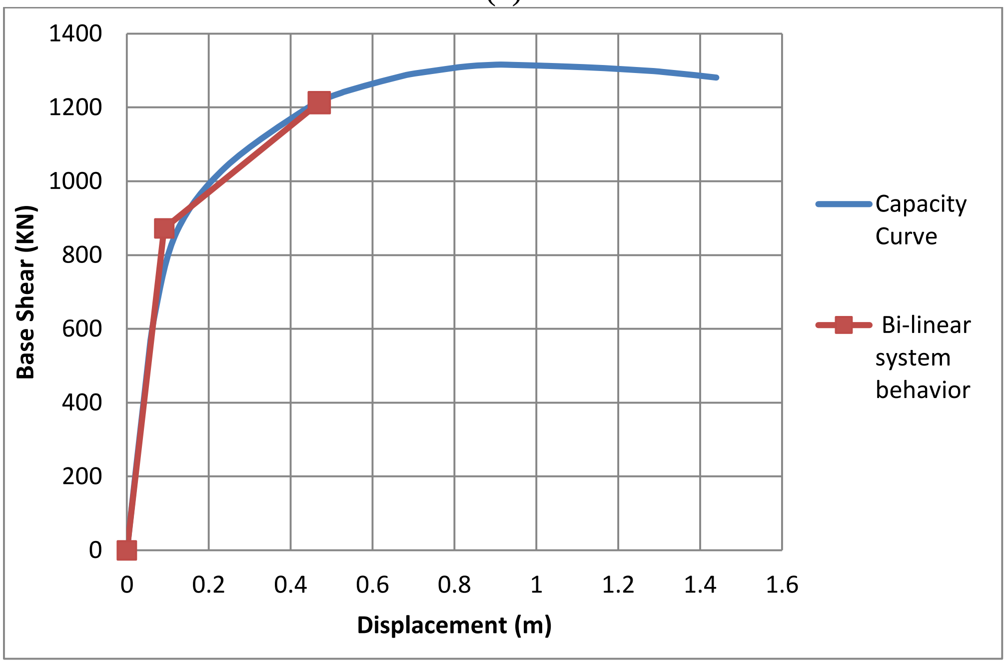
Figure 4.3(a-c) shows structure's pushover capacity curve with bilinear system behavior derived by performing pushover analysis, in accordance with FEMA 440, using SAP2000 program.



(a)



(b)



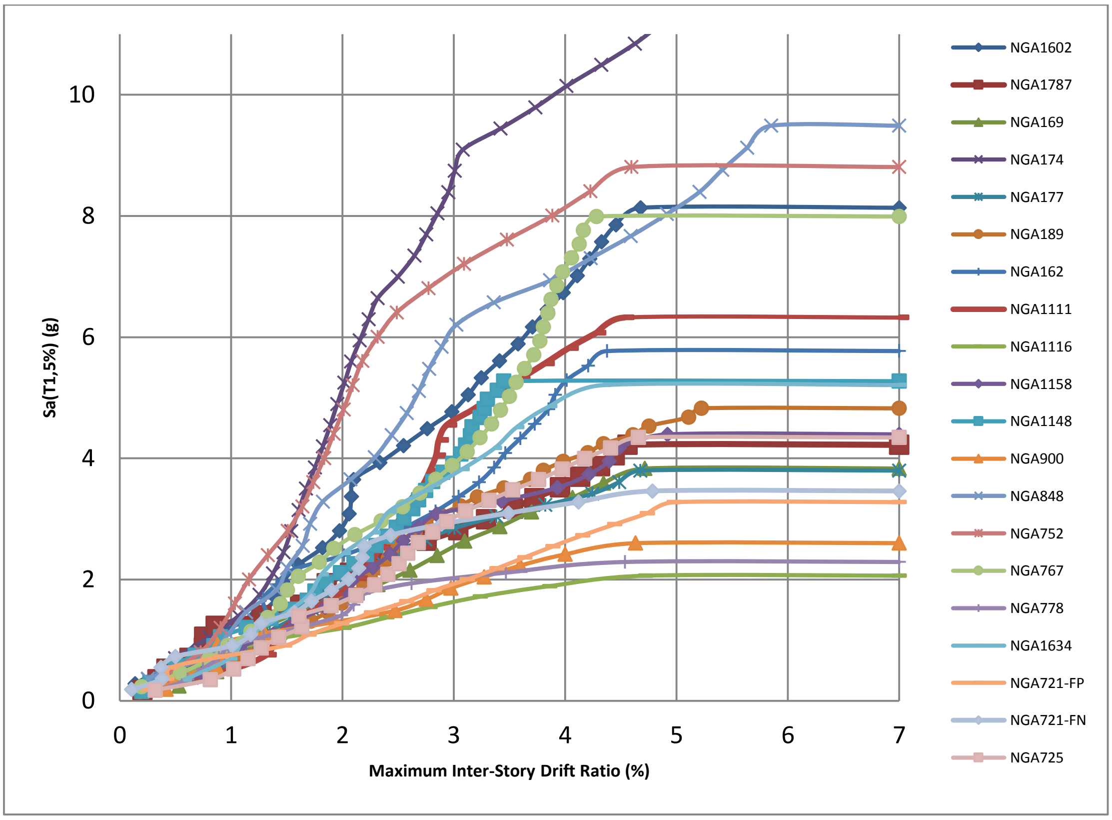
(c)

Figure 4.1. Static Pushover Curve and its Idealized force-displacement for (a) 3-story (b) 8-story (c) 12-story RC frame

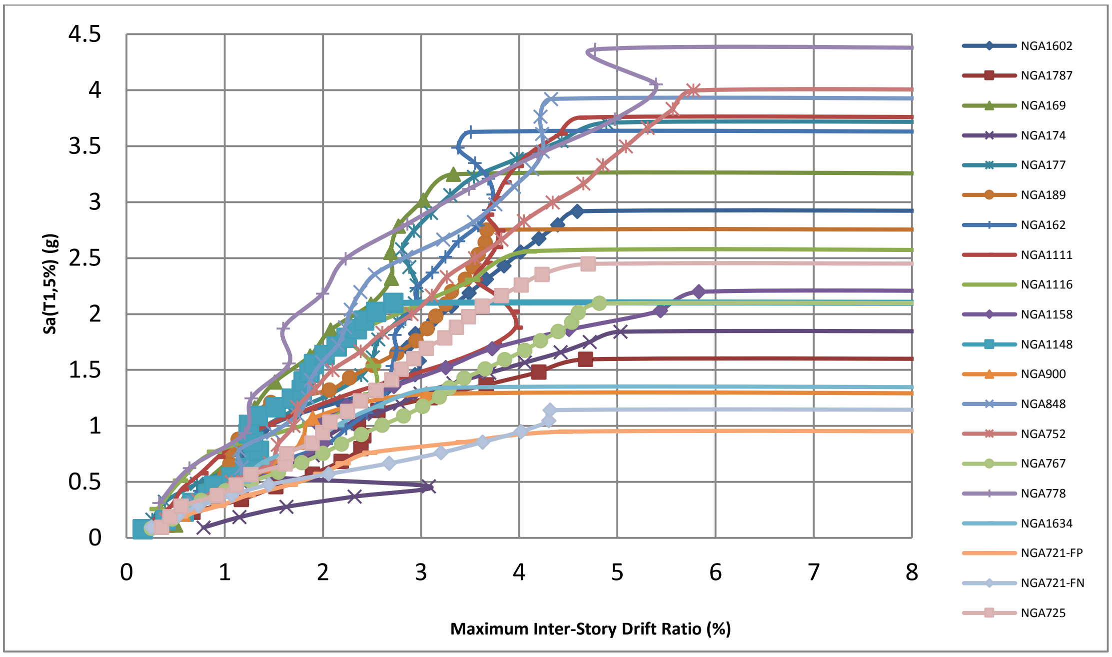
4.5 Incremental Dynamic analysis (IDA)

4.5.1 Post-processing and Generating IDA Curves

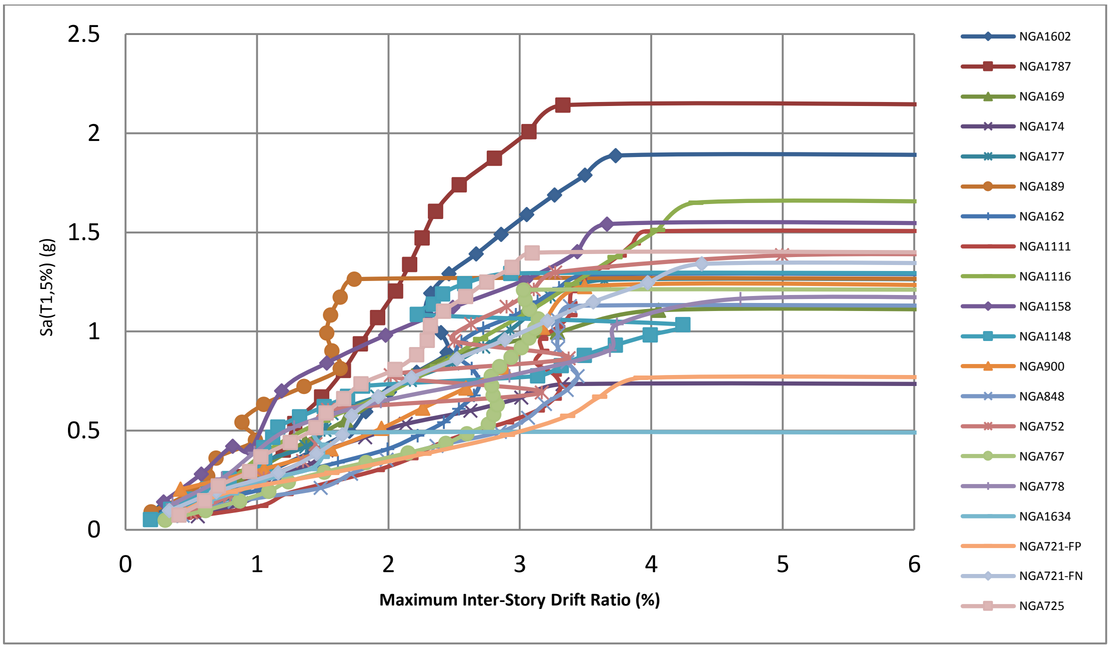
It is important to choose a proper IM and DM to obtain appropriate post-processing results. It has been proven by Luco and Cornell (2004) that first mode pseudospectral acceleration for damping equal to 5%, $S_a(T_1, 5\%)$ is a sufficient and efficient IM choice for the medium-rise structures since it provides a proper estimate of seismic demand and capacity of the building, and minimizes the scatter in the results.



(a)



(b)

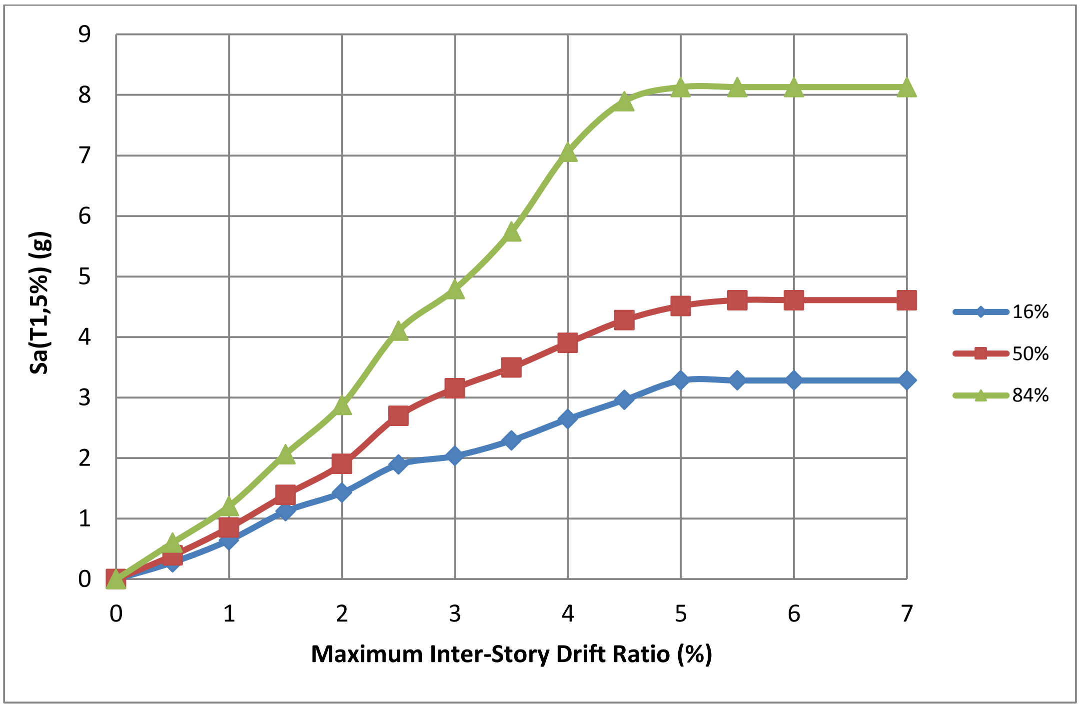


(c)

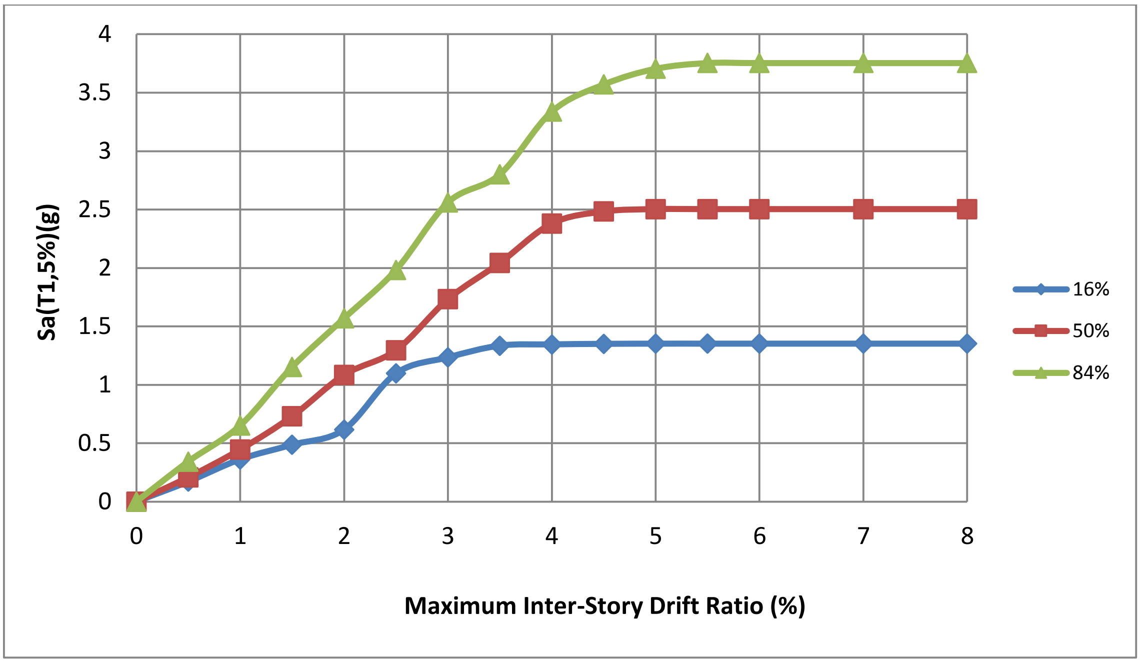
Figure 4.2. IDA curves of twenty earthquakes for (a) 3-story (b) 8-story (c) 12-story RC Frame.

4.5.2 Summarizing the IDA Curves

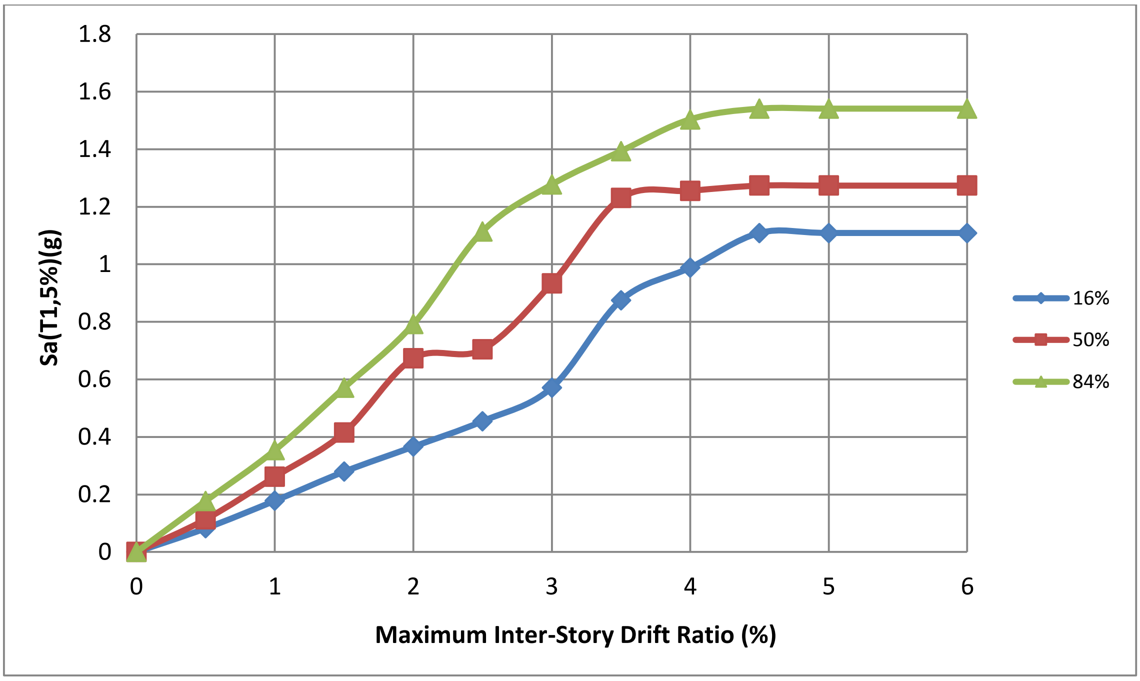
16%, median and 84% IDA curves were achieved by summarizing the multi-record IDA envelopes for each of the three case studies as explained in detail in section 3.6.5. Then, limit states have been defined at each performance level, as shown in Table 4.10(a-c). Figures 4.3(a-c) show multi-record IDA envelope (84%, median and 16% fragility curves) of each time period.



(a)



(b)



(c)

Figure 4.3. The Summary of the IDA Curve for (a) 3-story (b) 8-story (c) 12-story RC Frame.

4.5.2.1 Defining of Limit States

Table 4.10. Summarized capacities for each limit-states for (a) 3-story (b) 8-story (c) 12-story RC Frame.

(a)

	$S_a(T_1,5\%)$(g)			θ_{max}		
	16%	50%	84%	16%	50%	84%
Immediate Occupancy	0.644	0.851	1.210	0.01	0.01	0.01
Life Safety	1.430	1.903	2.880	0.02	0.02	0.02
Collapse Prevention	2.648	3.920	7.060	0.04	0.04	0.04

(b)

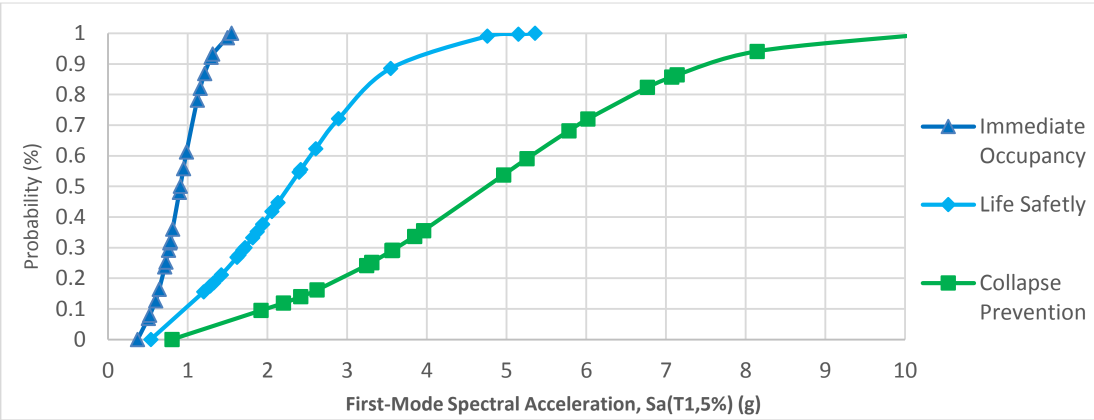
	S_a $(T_1,5\%)$(g)			θ_{max}		
	16%	50%	84%	16%	50%	84%
Immediate Occupancy	0.361	0.446	0.651	0.01	0.01	0.01
Life Safety	0.616	1.084	1.569	0.02	0.02	0.02
Collapse Prevention	1.345	2.380	3.338	0.04	0.04	0.04

(c)

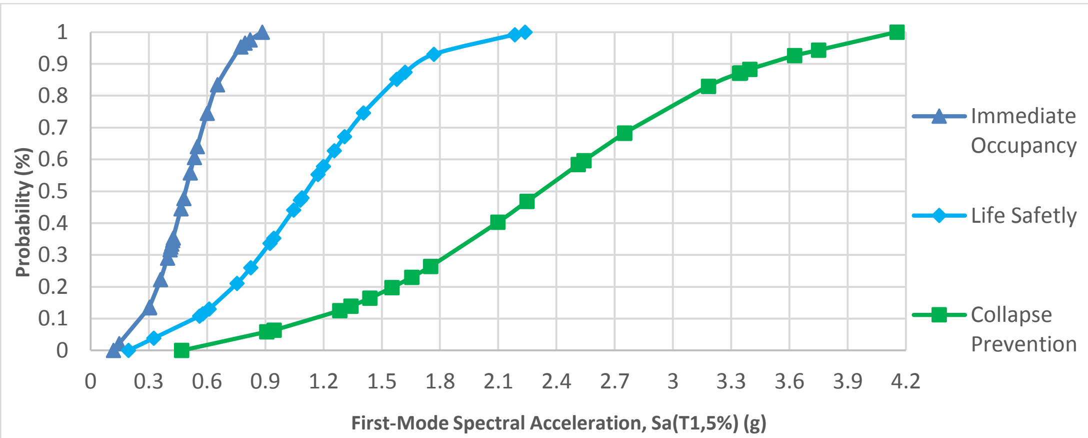
	S_a $(T_1,5\%)$(g)			θ_{max}		
	16%	50%	84%	16%	50%	84%
Immediate Occupancy	0.178	0.261	0.353	0.01	0.01	0.01
Life Safety	0.366	0.673	0.791	0.02	0.02	0.02
Collapse Prevention	0.988	1.255	1.5039	0.04	0.04	0.04

4.5.3 Probabilistic Fragility Curves

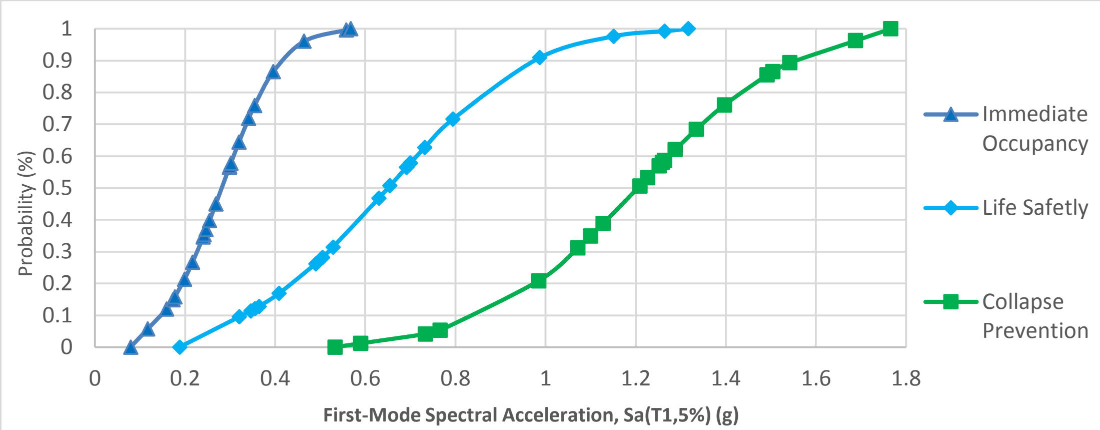
Fragility curves are useful tools for estimating potential and probability of structural damage due to earthquake as a function of peak ground acceleration and top displacement. Fragility curve can be determined based on standard probability distribution (assume to be lognormal). The normal distribution fragility curves in terms of first-mode spectral acceleration and top drift (m) at the predefined limit states are displayed in Figure 4.4(a-c) and Figure 4.4(a-c), respectively. Probability of each performance level for different earthquake zones are provided in Table 4.11.



(a)



(b)

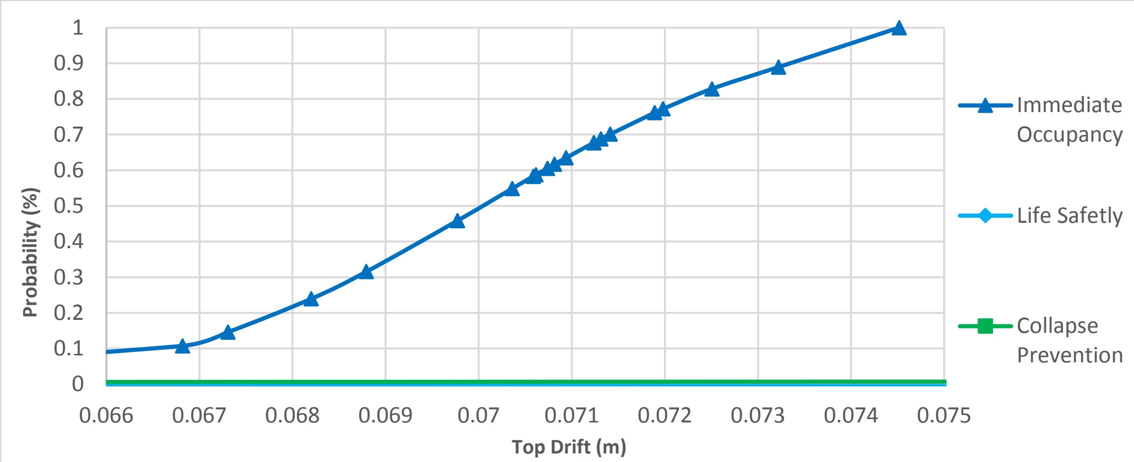


(c)

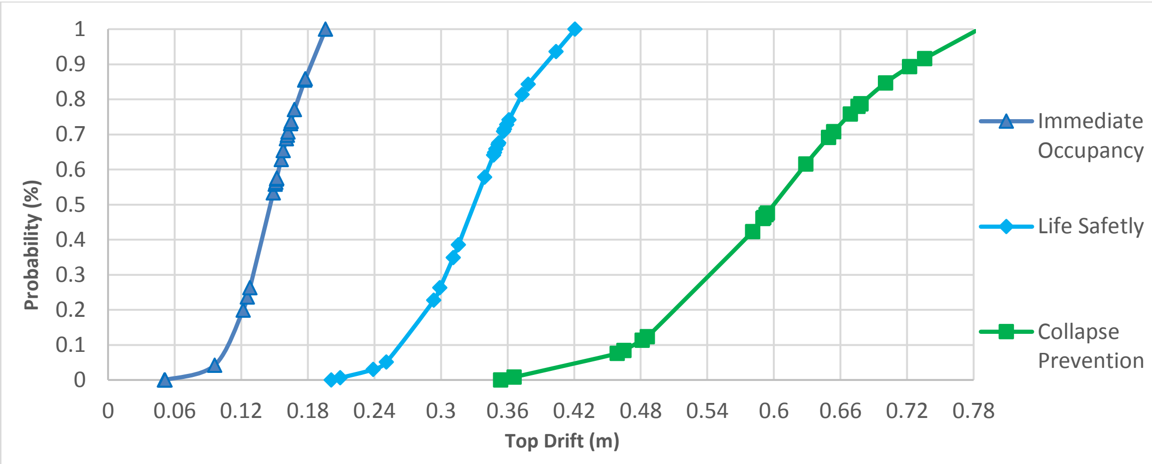
Figure 4.4. The normal distribution probabilistic fragility curves in terms of PGA (a) 3-story (b) 8-story (c) 12-story RC Frame.

Table 4.11. Seismic performance levels of Structures by performing Incremental Dynamics Analysis using SeismoStruct software.

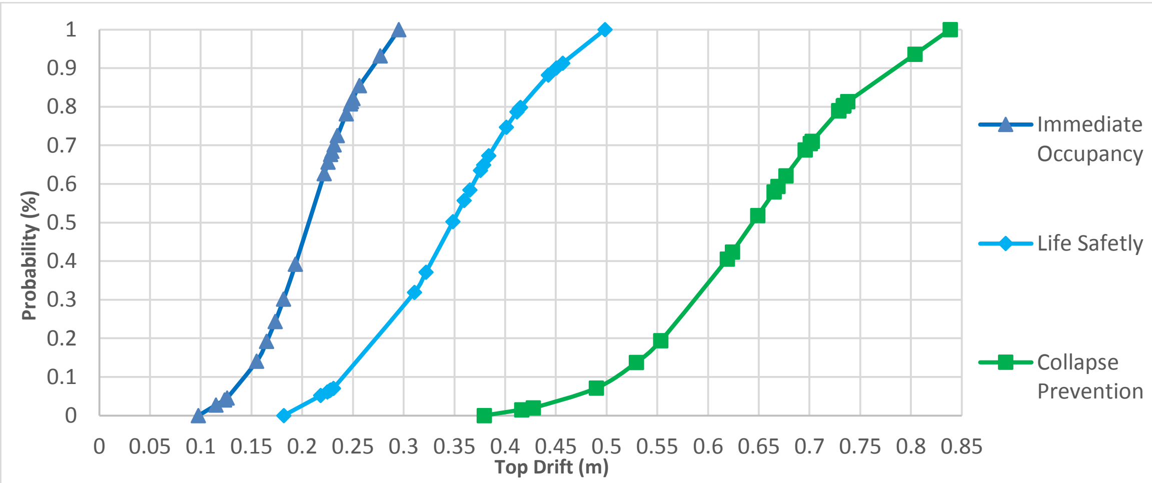
Structure Frame	**Limit state**	**Seismic Zone- First-Mode Spectral Accelaration (g)**		
		S_a=0.7	S_a=0.9	S_a=1.1
		Probability of Exceeding %		
3-Story	IQ	22	49	76
	LS	0	0	13
	CP	0	0	0
8-Story	IQ	88	100	100
	LS	18	32	49
	CP	0	0	9
12-Story	IQ	100	100	100
	LS	58	82	95
	CP	0	15	35



(a)



(b)



(c)

Figure 4.5. The normal distribution probabilistic fragility curves in terms of top drift (m) (a) 3-story (b) 8-story (c) 12-story RC Frame.

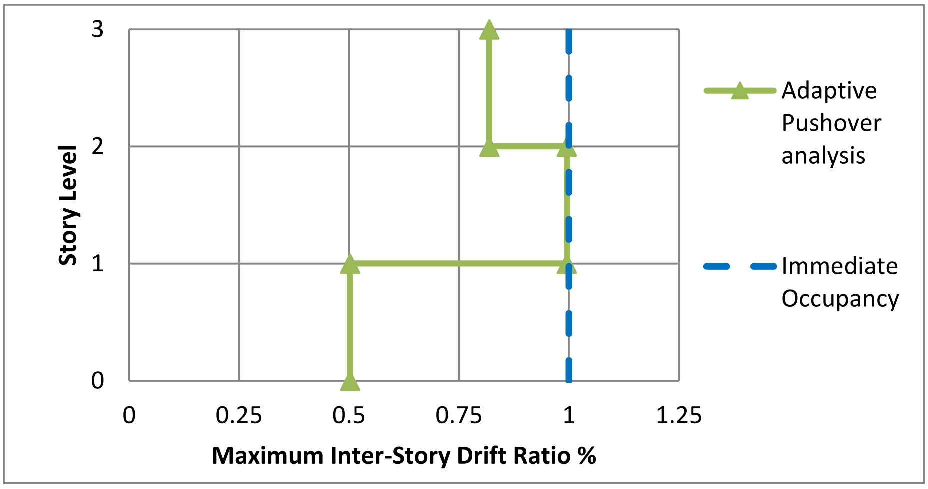
Target displacement for 3-story RC frame is equal to 0.071 meter, as calculated in section 4.1.1. At this point, the frame has the probability of 64% to be in IO level and no probability exist that the performance level of structure be in LS level and CP level, as derived from the Figure 4.5(a), according to IDA results using SeismoStruct software.

Target displacement for 8-story RC frame is equal to 0.318 meter, as calculated in section 4.2.1. At this point, the frame has the probability of 100% and 40% to be in IO and LS level, respectively and, no probability exist that the performance level of structure be in CP level, as derived from the Figure 4.5(b).

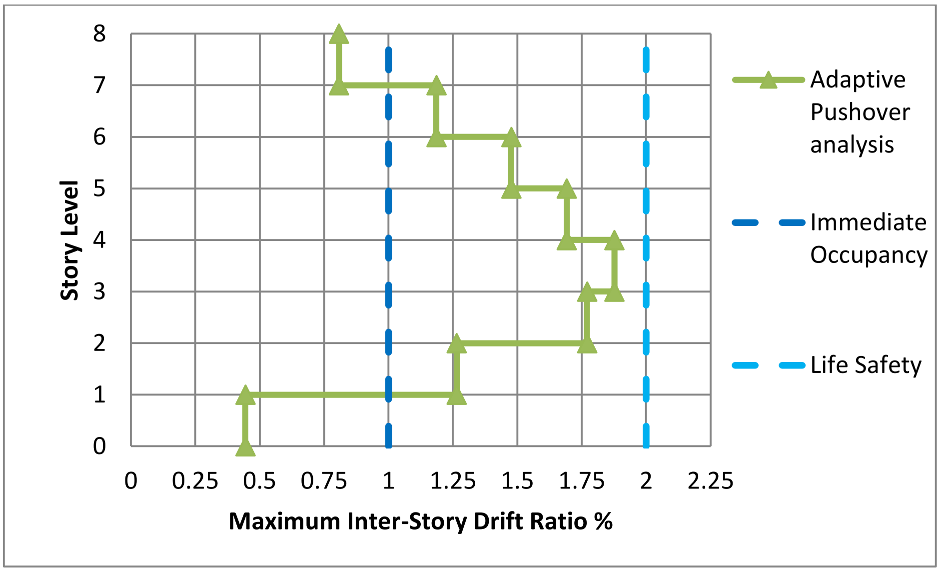
Target displacement for 12-story RC frame is equal to 0.47 meter, as calculated in section 4.3.1. At this point, the frame has the probability of 100% and 94% to be in IO and LS level, respectively and, no probability exist that the performance level of structure be in CP level, as derived from the Figure 4.5(c).

4.6 Inter-Story Drift Profiles derived by DAP Method

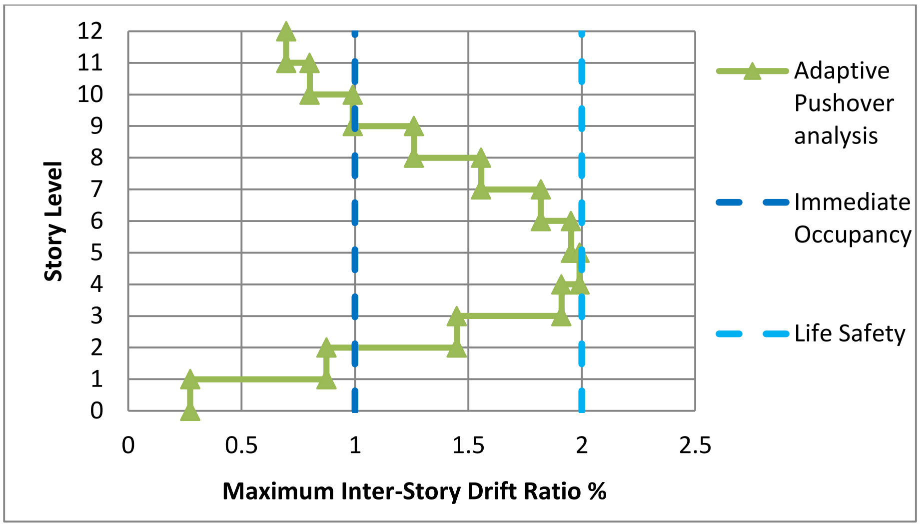
Dario (2008) states that the DAP characteristic is positive and encouraging in the estimating of the drift profiles shape of high-rise RC structures. In Figure 4.6(a-c), the obtained inter-story drift profiles of the three RC frames are demonstrated by performing DAP method using SeismoStruct software.



(a)



(b)



(c)

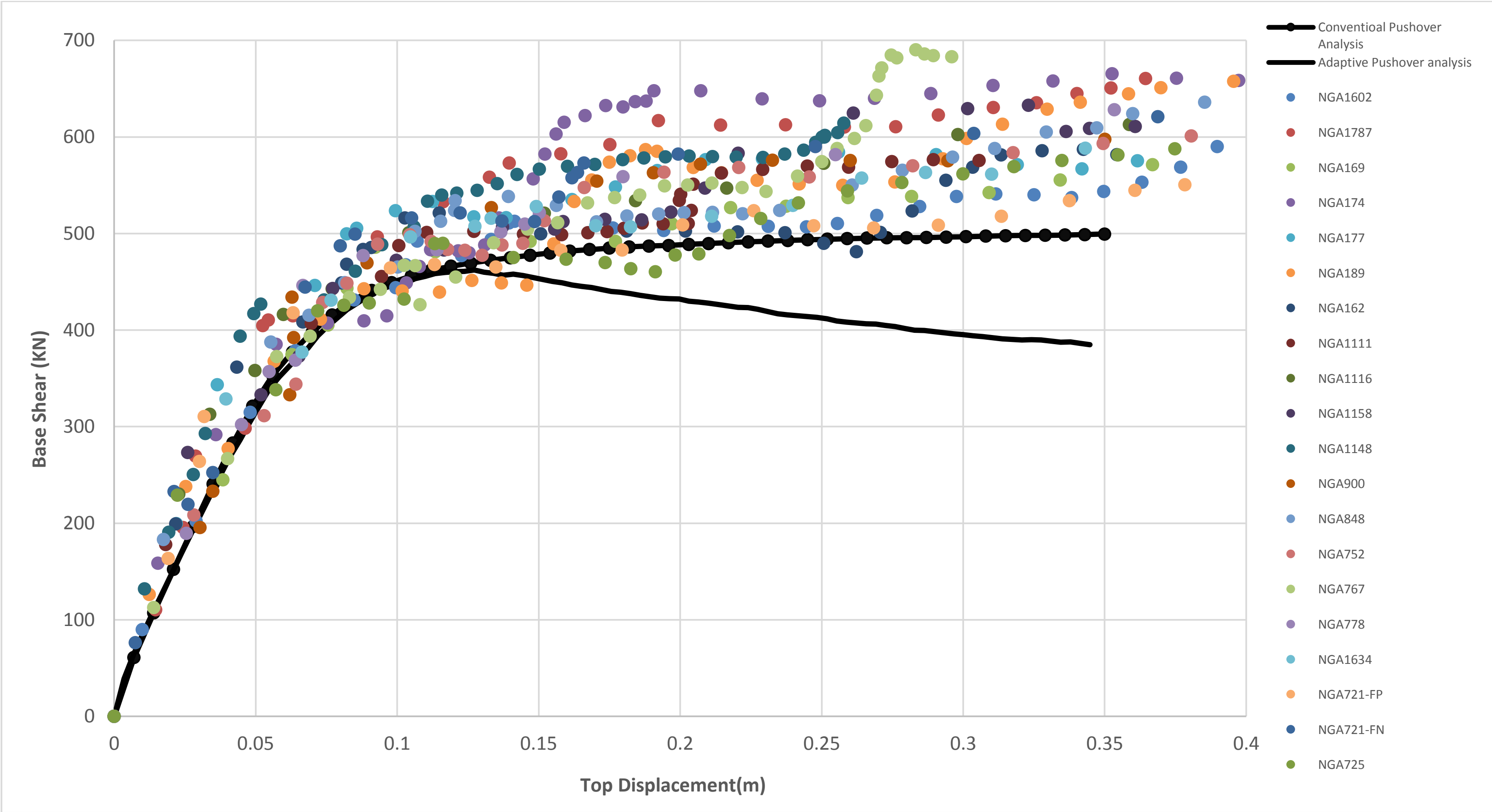
Figure 4.6. Inter-story drift profiles of (a) 3-Story (b) 8-Story (c) 12-Story RC Frame.

As shown in Figures 4.6(a-c), the maximum inter-story drift ratio of 3-story frame has reached to 1% and the limit state criteria at this level is immediate occupancy according to FEMA 356. In 8-story frame, the maximum inter-story drift ratio has not exceeded 2% and it still remain in immediate occupancy level and at this level, structure expected to have slight damage that can be easily repaired. The maximum inter-story drift ratio in 12-story frame has reached to 2% and the limit state criteria at this level is life safety the building is expected to have comprehensive damage and there is risk of injury to life.

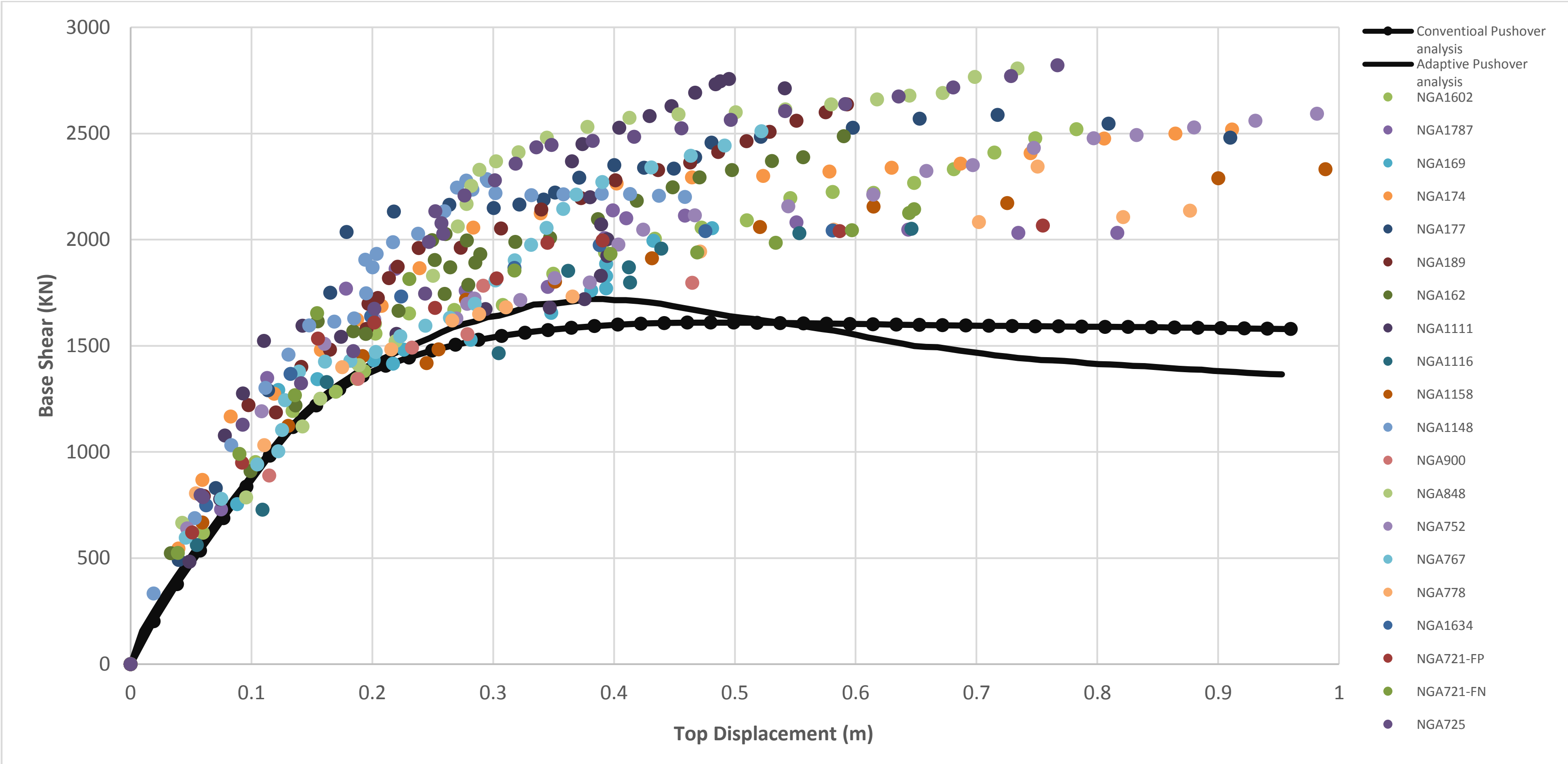
4.7 Comparison between Nonlinear Dynamic and Static Analyses

4.7.1 Base Shear vs. Top Displacement Curves (Capacity Curve)

One of the most important steps in post-processing of nonlinear structural analysis is to achieve the capacity curve (base shear versus top displacement). The capacity curve can reveal important features of structural response, includes yield displacement, total strength and initial stiffness estimation of the structure. Thus, it is important and crucial to compare the new approaches of pushover analysis to nonlinear dynamic analysis envelopes in terms of the base shear vs. top displacement curve. The aim of comparing different analyses method is to identify and understand the differences in the results achieved by different approaches, and verify their accuracy compared to dynamic and static analyses, applying variable or fixed load distributions (Rovithakis, Pinho, & Antoniou, 2002). The results of capacity curves obtained by different nonlinear analyses are shown in Figure 4.7(a-c). All the nonlinear analysis performed by using SeimoStruct software for reducing errors.

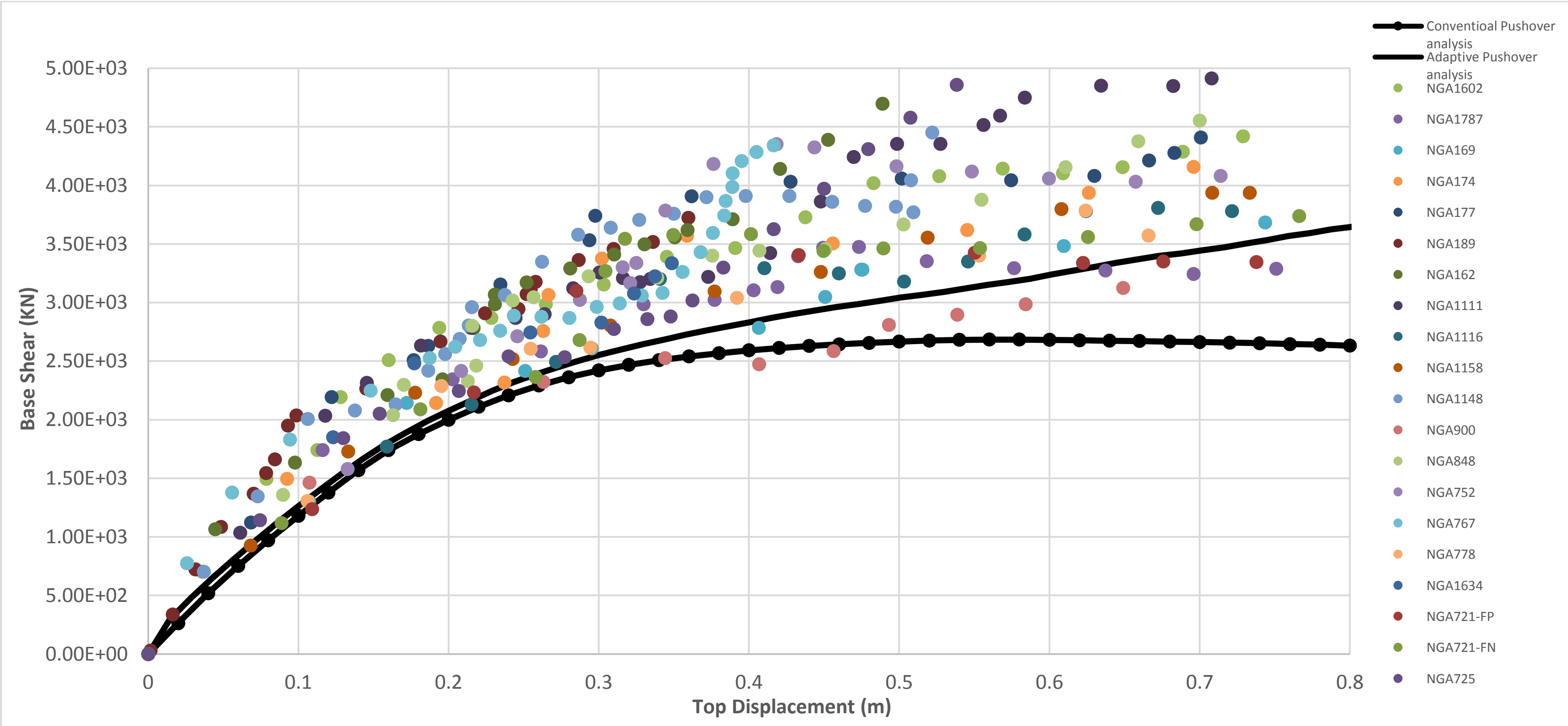

Base Shear (KN)
700
600
500
400
300
200
100
0
0
0.05
0.1
0.15
0.2
0.25
0.3
0.35
0.4
Top Displacement(m)
Conventioal Pushover Analysis
Adaptive Pushover analysis
NGA1602
NGA1787
NGA169
NGA174
NGA177
NGA189
NGA162
NGA1111
NGA1116
NGA1158
NGA1148
NGA900
NGA848
NGA752
NGA767
NGA778
NGA1634
NGA721-FP
NGA721-FN
NGA725

(a)



3000
2500
2000
1500
1000
500
0
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Base Shear (KN)
Top Displacement (m)
Conventioal Pushover analysis
Adaptive Pushover analysis
NGA1602
NGA1787
NGA169
NGA174
NGA177
NGA189
NGA162
NGA1111
NGA1116
NGA1158
NGA1148
NGA900
NGA848
NGA752
NGA767
NGA778
NGA1634
NGA721-FP
NGA721-FN
NGA725

(b)



(c)

Figure 4.7. Capacity curves of (a) 3-story (b) 8-story (c) 12-story RC frame, determined by performing conventional pushover and DAP, compared against IDA envelopes.

The capacity curves for 3-story building are shown in Figure 4.7(a). Study result of 3-Story RC frame reveals that adaptive and conventional pushover analyses are appropriate procedures when the fundamental modes dominate the response. The figure demonstrate that the accuracy of the conventional pushover and DAP approaches seems to be satisfying up to the displacement equal to 1.4 percent of the total height. After that negative post-yielding stiffness can be observed in DAP method due to decreasing the post yielding stiffness of the structure and the methods cannot trace the exact behavior of the structure up to the collapse point.

In Figure 4.7(b), the curves for 8-story structural system are presented. The result of DAP method are acceptable until the roof displacement reaches to 1.25% of the total height and after that negative post-yielding stiffness can be observed. It can be seen from the results that the conventional pushover method was inadequate to capture the characteristics of the dynamic behavior, but the adaptive curves provided an approximate fit to the dynamic envelopes.

The capacity curves of the 12-story structure are shown in Figure 4.7(c). The curves are derived by conventional pushover analysis and DAP method, then are compared with the IDA envelopes. The curve clearly demonstrates the advantages of using DAP analysis over non-adaptive pushover analysis. Nevertheless, both the DAP and conventional pushover analyses could not reproduce the dynamic envelope results. According to Antoniou and Pinho (2002), there are two main factors that a dynamic behavior may be attributed. Firstly, the sensitivity of the structural response to the peculiarities of each earthquake record, which can cause irregular, erratic-shaped dynamic envelopes. Secondly, and most importantly, the difficulties faced by static procedures, even by the more sophisticated adaptive schemes, to describe complex dynamic phenomena.

Static analyses, in terms of its pushover curve, have the ability to assess the response of structure under seismic load with acceptable accuracy in the case of structures where the structural response is dominated by first mode, as in the case of the 3-Story RC frame. Instead in 8-Story structure, estimation of seismic demand became less accurate at high stages of structural deformations, associated with the response contributions of higher modes. In particular, the conventional pushover curve only in the case of the 3-Story frame provides a pushover curve closer to the IDA envelope but DAP demonstrates higher consistency in leading to less erroneous estimations.

4.7.2 Performance Limit States of Nonlinear Dynamic and Static Analyses

The acceptance limits and actual damage level that obtained by different nonlinear procedures are shown in Tables 4.12. By comparing the Structural seismic performances achieved with damage predicted performing DAP (obtained in section 4.6), IDA (obtained in section 4.5.3) using SeimoStruct software and conventional pushover analysis (obtained in section 4.3.2) using SAP2000 software , in 3, 8 and 12-story frame, all the procedures were found to have approximately the same level of performance.

Table 4.12. Seismic performances of Structures by performing DAP and Pushover Analyses and IDA Methods.

Structure Frame	**δ_t (FEMA440) (*m*)**	**DAP**	**Pushover Analysis**	**Incremental Dynamic Analysis**	
		Limit State Criteria	Limit State Criteria	Limit state	Probability of Exceeding (%)
3-Story	0.071	Immediate Occupancy (IQ)	Immediate Occupancy (IQ)	IQ	64
				LS	0
				CP	0
8-Story	0.318	Immediate Occupancy (IQ)	Immediate Occupancy (IQ)	IQ	100
				LS	40
				CP	0
12-Story	0.47	Life Safety (LS)	Life Safety (LS)	IQ	100
				LS	94
				CP	0

Chapter 5

SUMMARY AND CONCLUSION

5.1 Summary

The general description of structures is as follows; Three RC structures, with different height, are considered to represent high-rise, medium-rise and low-rise RC structures for this work. The structures have a typical RC elements without any shear walls and are supposed to be located in a high-seismicity region of Turkey. Structures are designed according to TS 500-2000 and Turkish Earthruake Code (2007), taking into account seismic and gravity loads. Three reinforced concrete structures, is assessed by employing various nonlinear analysis procedures in this study.

The nonlinear static analysis, in accordance with FEMA440, has been explained and used. The conventional nonlinear static analysis method is a relatively simple method for assessing seismic capacity and demand of RC structures as described in chapter 3. However, further study is still needed to determine limitations of the approach and prove the accuracy and reliability of the method. The method has been performed by using CSI SAP2000 finite element software.

The performance of the Displacement-based Adaptive Pushover (DAP), proposed by Antoniou and Pinho (2004(b)), has been evaluated and compared with conventional pushover analysis and IDA in the cases of 3, 8 and 12-story RC frames by using SeismoStruct software.

Through this study, detailed and fundamental methodology of Incremental Dynamic Analysis has been discussed. Twenty ground motion records have been applied on the considered frames and then nonlinear time history analysis have been executed for different levels of scaling of each ground motion record by using SeismoStruct software. Finally, IDA envelope curves have been derived from the analysis and then damage levels of structures have been demonstrated in accordance with FEMA356 for different level of limit states. Consequently, probabilistic fragility curves are also achieved in terms of top drift and PGA for each considered levels of damaged and probability of each performance level for different drift ratio and earthquake zones given in the code calculated.

The main objectives of this study were to assess and compare performances of traditional pushover and dynamic analysis (IDA) with those obtained with the more recently proposed multi-mode nonlinear static analysis (DAP) and verify reliability of the procedures for RC-frame structures. In detail, in the first part of study the attention has been focused on the performance of static and dynamic methods, in estimating global response of the structures by comparing pushover curves with IDA envelopes. The second study concentrates on the evaluation of the performance of static and dynamic procedures in limit state predictions of RC frames.

5.2 Conclusion

In current engineering design practice, the conventional pushover analysis represents an easier and more practical method with respect to nonlinear dynamic analyses. The procedure avoid the major pitfalls in time-history analyses, which require simulation of time history ground motions records compatible with target response spectrum and they remain computationally demanding particularly in evaluating a 3D structure model that has thousands of elements.

According to the results of the study, displacement-based adaptive pushover analysis represents an improvement regarding to other static procedure, although the method couldn not provide the optimal solution. In fact, the capacity curves clearly demonstrate that DAP provides better estimates particularly for high-rise structures in which the effects of vibration higher modes are significant. In other words, results obtained in present study clearly demonstrates the advantages of using DAP analysis over non-adaptive methods in estimating of the seismic response evaluation of high-rise structure responding in the inelastic range of behavior. Hence, DAP represents simplify and practical procedure that able to predict the response shape of high-rise RC structures with appropriate accuracy, although it cannot estimate completely satisfactory compared with dynamic procedures.

5.3 Recommendations

In this work, the number of analyzed models is not enough to make any definite conclusion, therefore, further and profound study of adaptive and conventional pushover analyses have to be carried out to determine limitations of the methods and establish the generality of the results.

Furthermore, all the tests have been performed on 2D regular plan models which may not clearly highlight the high-mode effects. Thus, more case studies is required particularly for taller irregular plan building structures

Various multi-mode pushover analysis approaches have been proposed and developed over decades by many researchers to take to account the structural responses in several modes and these approaches can be compared to each other to find the best practical multi-mode pushover analysis approaches. The simplest procedure that is as close to reality as possible is usually the best one.

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