ASSESSMENT OF NONLINEAR STATIC (PUSHOVER) PROCEDURES FOR SEISMIC EVALUATION OF REINFORCED CONCRETE STRUCTURES

by

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LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ADRS	Acceleration-Displacement Response Spectrum
AMC	Adaptive Modal Combination
APA	Adaptive Pushover Analysis
ATC	Applied Technology Council
CM	Coefficient Method
CMP	Consecutive Modal Pushover
CSM	Capacity Spectrum Method
DAP	Displacement-based Adaptive Pushover
ELF	Equivalent lateral Force
FEMA	Federal Emergency Management Agency
ICBO	International Conference of Building Officials
ICC	International Code Council
IDA	Incremental Dynamic Analysis
MDOF	Multi Degree of Freedom
MFDC	Mexico Federal District Code
MMP	Multi-Modal Pushover
MPA	Modal Pushover Method
NBC	National Building Code of Canada
NEHRP	National Earthquake Hazards Reduction Program
NRHA	Nonlinear Response History Analysis

NSP	Nonlinear Static Procedure
NTHA	Nonlinear Time History Analysis
PGA	Pick Ground Acceleration
PHL	Plastic Hinge Length
PMPA	Practical Modal Pushover Analysis
PRC	Pushover Result Combination
RSA	Response Spectrum Analysis
Sa	Spectral Acceleration
Sd	Spectral Displacement
S-DAP	Statistical Displacement-base Adaptive Pushover
SDOF	Single Degree of Freedom
SRSS	Square Root of Sum of Squares
SSAP	Story Shear-based Adaptive Pushover
TF	Torsionally-Flexible
TS	Torsionally-Stiff
TSS	Torsionally-Similarly-stiff
UBPA	Upper-Bound Pushover Analysis

LIST OF SYMBOLS

C_0	Differences of displacements
C_1	Modification factor for assessing the maximum inelastic deformation
C_2	Response to possible degradation of stiffness
C_3	Modification factor for the P–Δ effect
C	Neutral axis depth at collapse
C	Damping matrix
d	Effective depth of a beam
d_b	Diameter of longitudinal reinforcement
D_n	Peak deformation of n^{th}-mode
D	Generalized displacement of the equal SDOF
EMM_n	N-th mode of effective modal mass
f_c'	Concrete strength
F_r	Resisting force term
$g(\varepsilon)$	Error in the returning force
h	Section depth
K_i	Elastic lateral stiffness
K	Stiffness matrix
K_e	Effective elastic stiffness
L	Height of a cantilever column
L_V	Shear span
$\ell_{\mathbf{p}}$	Equivalent plastic hinge length
M	Diagonal mass matrix
q	Tension reinforcement index

q'	Compressive reinforcement index
q_b	Balanced tension reinforcement
Q	Global displacements
r_g	Contribution of gravity loads due to n^{th}-mode
R	Applied load
R(q)	Nonlinear function of q
R(u)	Restoring force of the inelastic system
R	Ratio of inelastic strength demand
S_d	Spectra displacement
S_a	Spectral value of acceleration response
T_i	Elastic fundamental period
T_0	Characteristic period of the response spectrum
Teq	Higher period of vibration
T_e	Effective fundamental periods
u	Displacements of the structure
$\dot{u}$	Velocities of the structure
$\ddot{u}$	Accelerations of the structure
U_r	Roof displacement
V	Base shear
V_y	Yield strength
W	Total dead load and expected live load
w_i/g	Mass assigned to level i.
Z	Distance from critical section to point of contraflexure
z/d	Moment gradient

ζeq	Equivalent damping ratios
ζ_0	Elastic viscous damping
μ	Displacement ductility
μ_φ	Curvature ductility
μ_Δ	Displacement ductility
ε	Vector presentation the error
φ^t	Weighting vector
Γ_n	Modal participation factor
ω	Frequency
α_n	Modal mass coefficient for the n natural mode
Δ_{roof}	Roof displacement
$\emptyset_{in}$	Amplitude of n^{th}-mode at level i
$\emptyset_{rn}$	n^{th}-mode shape at the roof
ω_n	Natural frequencies

TAKSIRAN PROSEDUR TIDAK LINEAR STATIK (PUSHOVER) UNTUK PENILAIAN SEISMIK STRUKTUR KONKRIT BETETULANG

ABSTRAK

Secara umum, gempa bumi adalah salah satu bencana semulajadi yang paling serius yang manusia pernah alami sejak hari pertama tamadun. Oleh itu, prestasi seismik struktur tertakluk kepada gempa bumi sentiasa menjadi isu kritikal. Tesis ini membentangkan penilaian prosedur tidak linear statik menggunakan prosedur sejarah masa tidak linear. Prosedur pushover yang dipilih dalam kajian ini merangkumi Kaedah Pekali, Kaedah Kapasiti Spektrum dan Kaedah Pushover Modal. Oleh sebab panjang engsel plastik adalah satu parameter yang berkesan dalam analisis pushover, kajian ini membincangkan panjang engsel plastik yang berbeza iaitu kes takrifan biasa dan takrifan pengguna. Dalam konteks ini, kerangka konkrit bertingkat 2, 5, 8 dan 12 telah dipilih untuk mewakili struktur yang rendah, sederhana dan tinggi. Keputusan Kaedah Analisis Pushover Modal dan Kaedah Pekali (di bawah syarat tertentu) boleh digunakan untuk analisis struktur dan lebih realistik berbanding Kaedah Kapasiti Spektrum. Selain itu, perbandingan keputusan yang diperolehi daripada panjang engsel plastik yang dipilih menunjukkan bahawa, walaupun keputusan panjang engsel plastik untuk kes takrifan pengguna dan takrifan biasa menghasilkan keadaan alah yang hampir sama, tetapi keputusan dalam keadaan muktamad adalah jauh berbeza. Sebagai kesimpulan, cadangan panjang engsel plastik untuk kes takrifan pengguna menunjukkan prestasi yang lebih baik dalam analisis berbanding kes takrifan biasa.

ASSESSMENT OF NONLINEAR STATIC (PUSHOVER) PROCEDURES FOR SEISMIC EVALUATION OF REINFORCED CONCRETE STRUCTURES

ABSTRACT

In general, earthquake is one of the most serious natural disasters that mankind has ever suffered since the first day of civilization. Hence, the seismic performance of structures subjected to earthquake always becomes critical issues. This thesis presents the assessment of current nonlinear static procedures using nonlinear time history procedure. The selected pushover procedures in this research are consisting of Coefficient Method, Capacity Spectrum Method and Modal Pushover Method. Since plastic hinge length is an effective parameter in pushover analysis, this study discusses different plastic hinge lengths. These lengths are calculated for both default and user-defined cases. In this context, 2, 5, 8 and 12 storey frame were selected to represent the real low, medium and high rise regular reinforcement concrete structure. The results of the pushover analysis indicated that behaviour of the structures using modal pushover analysis method and coefficient method (under certain conditions) were more realistically than those analysed using capacity spectrum method. Moreover, the comparison of the results obtained from selected plastic hinge length reveals that, although the results of user-defined and default plastic hinge length in yielding state are almost similar, the results in ultimate state are significantly different. Therefore, it can be concluded that in this study proposed user-defined plastic hinge length shows better performance of hinge in analysis as compared to default plastic hinge length.

CHAPTER 1
INTRODUCTION

1.1 Background

Earthquake is one of the most serious natural disasters that mankind has ever suffered. Although analysis of recent earthquake showed that new analysis methods applied in buildings to preserve human life and reduce the loos of life, economic losses due to property damages and disruption on business and companies might be very huge. For example, a country like Japan with a well-established infrastructure in earthquake engineering, Tohoku earthquake and tsunami in 2011 causes property damage, including heavy damage to roads, railways and dam. Moreover the tsunami caused a number of nuclear accidents in three reactors and the associated evacuation zones affecting hundreds of thousands of residents. Hence, earthquake engineering is very important field for design and construction in civil infrastructure.

An important step in the design of a building to resist earthquakes is performed using the analysis of a structural system to determine the deformations and forces induced by applied ground excitation or loads. A structural analysis approach requires are consisting;

- A model of the building.
- A presentation of the earthquake ground motion or the effects of the earthquake ground motion.
- An approach of analysis for creating and solving the governing equations.

There are many methods depending on the purpose of the analysis in the design process which are classified into two parts including linear and nonlinear. Linear and nonlinear methods are primary used to analyse structure in elastic and inelastic zone, respectively. Each linear and nonlinear method is divided into two categories including static and dynamic analysis to investigate behaviour of structure in earthquake. Although nonlinear methods have better estimation performance of structure in earthquake nonetheless, their use requires precision, skill and consumes additional time, especially in dynamic analysis.

According to Krawinkler and Seneviratna (1998) nonlinear static (pushover) analysis provides useful information to structural engineers regarding the structure's capacity against lateral load via the presentation of formation plastic hinges mechanism, capacity curve and the estimation of inter-storey drift.

There are many way by which a building can dissipate the earthquake input energy. One of these mechanisms can be very effective, called plastic hinges. The lateral strength of a building can be completely attained from designed, if the building is able to form a column or beam mechanism. In plastic hinges mechanism, the inelastic deformations are concentrated at the column ends and the beam ends (column base) of the first storey. These locations are known as the plastic hinges. Also, estimate of the drift and inelastic capacity for a section or a structure depend on the length over which the inelastic deformation will happen. The plastic hinge length (PHL), which happened in the column or beam has been studied widely by many researchers, while lack of studies were found in effect of PHL on structural analysis.

1.2 Problem Statement

Behaviour of structure in earthquake has shown that many buildings cannot tolerate the earthquake forces, even some of the buildings that was designed based on proposed liner static analysis by codes. Since, the dynamic methods were very complicated and time-consuming. Therefore, nonlinear static (pushover) method has made great progress among other methods.

Beginning of nonlinear static analysis is often attributed to the work of many researchers that proposed an approach wherein the response of a Multi-Degree-of-Freedom (MDOF) system was determined from analysis of an equivalent Single-Degree-of-Freedom (SDOF) system. Many researchers have proposed various methods in the form of pushover analysis, which have shown different results. Also the methods that have been proposed by the codes cannot obtain the similar results. Comparison of the proposed methods shows that the main differences between the methods were in the selection of lateral load distribution and in defining the dynamic properties of sections. So, it fill gap of knowledge in assessment of nonlinear static analysis and dynamic properties

Therefore, this research makes an effort to assess the proposed methods by FEMA-356 (2000), ATC-40 (1996) and proposed method by Chopra and Goel (2002) that are widely used procedures in structural analysis.

In the implementation of pushover analysis, modelling is one of the important steps. The model must consider dynamic behaviour of elements. Such a model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities. Lumped plasticity idealization of a cantilever is a commonly used approach in models for deformation capacity estimates. The ultimate deformation capacity of a component depends on the ultimate curvature and plastic hinge length. The use of different criteria for the different plastic hinge length may result in different deformation capacities. Hence, plastic hinge length is investigated as one of effective dynamic properties in pushover analysis.

1.3 Objectives

This research represents an evaluation of the proposed procedures in nonlinear static analysis using nonlinear time history analysis. Case study are SDOF square-shaped 2, 5, 8 and 12-storey reinforcement concrete frame that is covering low, medium and high rise structures. The main objectives of this study are as follows:

i. To evaluate the performance of Coefficient Method (CM), Capacity Spectrum Method (CSM) and Modal Pushover Method (MPA) using Nonlinear Time History Analysis (NTHA).
ii. To determine the effect of user-defined PHL compared with default PHL proposed by FEMA-356 (2000) in pushover analysis.

1.4 Scope of Works

This research considers the following scope of work:

i. Only 4 type of number of storey, 2, 5, 8 and 12 were designed by ACI 318-05 (2005) to represent the low, medium and high rise reinforcement concrete structure with fundamental period, 0.514, 0.956, 1.195 and 1.690 sec, respectively.

ii. Only consider the 3 type of nonlinear static procedure (NSP) presented to analysis of frames including CM of FEMA-356 (2000), CSM recommended by ATC-40 (1996) and MPA method proposed by Chopra and Goel (2002).

iii. Since the NSP is used as method, this research only considers the SDOF system to be investigated.

iv. Only nonlinear time history direct integration considered to evaluate of proposed NSP.

v. The input ground motion to use in NTHA are consist of two horizontal components of ten ground motion with different magnitude (ranging from 6.4 to 7.6) and different Peak Ground Acceleration (PGA) obtained from Pacific Earthquake Engineering Research Centre (PEER, 2006).

vi. All analysis including NSP and NTHA is based on 5 percent damping ratio.

vii. The evaluation of proposed methods are based on default PHL considered by FEMA-356 (2000).

viii. The user-defined PHL for column is based on proposed formula by Park et al. (1982) and for beam is based on proposed formula by Paulay and Priestley (1992).

ix. The default PHL used by SAP2000 is based on proposed formula by FEMA-356 (2000)

x. For convenience in data analysis, P-Δ effect for both NTHA and pushover analysis is neglected.

xi. The SAP2000 (CSI, 2010) computer program was used to perform the NTHA and pushover analysis on proposed frames.

xii. Only 5, 8 and 12 storey frames considered to evaluation of PHL. 2 storeys frame is not selected because, in low-rise structures the behaviour of hinge for different PHL are almost similar.

1.5 Thesis Outline

This thesis consist of five chapters namely as introduction, literature review, methodology, result & discussion and conclusion. Chapter 1 presents the general background of the study, problem statement, objectives and scope of works.

Chapter 2 discusses the previous researches presented by previous researchers related to this study work. This chapter presented a summarized of all method that considered in analysis of building. In continue presented the several NSP proposed by researcher and codes. The effects of PHL on buildings are included in this chapter as well.

The research methodology of this study is discusses by detail in Chapter 3. The pushover analysis methods which are used to analysis including CM, CSM and MPA are presents clearly. In addition case study frames and input ground motions to use in NTHA are presents. In this chapter, the frame hinge properties and how to calculate the PHL is presented in detail as well.

Chapter 4 is presents the results and discussion obtained from this study. This chapter first presents the results obtained from selected pushover methods, and then comparing these results with the results obtained from NTHA. Furthermore, this chapter presents a comparison between the results from pushover analysis based on user-defined and default PHL.

Finally, Chapter 5 presents the conclusions of all findings obtained from this study. This chapter concludes that which of the proposed methods achieved better results. Furthermore, concludes that which of the proposed PHL achieved better results.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter consists of three main sections. The first section provides a summary of the types of analysis used in structures analysis. This summary includes a definition of linear and nonlinear (static or dynamic) analysis and importance and performance in structural analysis. The second section provides a definition of nonlinear static (pushover) analysis and existing methods in this field. Also the proposed method is evaluated in second part of this section. The third section discussed about one of the important properties of pushover analysis that can dissipate the earthquake input energy. This property is known as the plastic hinge length, which is described in this section. Moreover proposed most formulas used in measuring the length of the hinge and selected studies in the evaluation of PHL formulas.

2.1.1 Analysis of structures

The earthquake is amongst the most feared of all natural disasters, exacting a devastating adverse effect on human life. In the last 100 years there have been many major earthquakes including, San Francisco (1906), Messina (1908), Alaska (1964), Manjil-Rudbar (1990), Kobe (1995), Sumatra (2004), Sichuan (2008), Tōhoku (2011) and many more. The devastating tsunami that followed the 9.0 M (magnitude) earthquake off the west coast of northern Sumatra, Indonesia on 26 December, that struck the coastlines of the Indian Ocean, was caused by an underwater earthquake. Most recently the destructive force of the earthquake was felt in Haiti in January 2010 that estimated 50,000 were killed. During in these earthquakes hundreds of thousands people have lost their life and cause billions of dollars in damage to property and infrastructure, and the physical suffering and mental anguish of earthquake survivors are beyond the mind. Consequently, earthquake is one of the most important human concerns.

Earthquake engineering started at the end of the 19^{th} century when some European engineering suggested designing structure with a few percent of the weight of the structure as the horizontal load. This idea of seismic design was taken up and developed in Japan at the beginning of the 20^{th} century (Hu et al., 1996).

For seismic performance assessment of structure analysis is required to determine force and displacement demands in various components of the structure. A significant decision in a structural analysis is to assume whether the relationship between forces and displacements is linear or nonlinear. Linear analysis for static and dynamic loads has been used in structural design for decades. Also, nonlinear analysis procedures were usually used, because emerging performance-based guidelines require representation of nonlinear behaviour. The structural analysis procedures used in earthquake-resistant design are summarized in Table 2.1 (Bozorgnia and Bertero, 2004).

Table 2.1: Structure Analysis Procedure for Earthquake-Resistant Design (Bozorgnia and Bertero, 2004)

Category	Analysis Procedure	Force-Deformation Relationship	Displacements	Earthquake load	Analysis Method
Equilibrium	Plastic Analysis Procedure	Rigid-Plastic	Small	Equivalent Lateral Load	Equivalent Analysis
Linear	Linear Static procedure	Linear	Small	Equivalent Lateral Load	Linear Static Analysis
	Linear Dynamic Procedure I	Linear	Small	Response Spectrum	Response Spectrum Analysis
	Linear Dynamic Procedure II	Linear	Small	Ground Motion History	Linear Response History Analysis
Nonlinear	Nonlinear Static Procedure	Nonlinear	Small or Large	Equivalent Lateral Load	Nonlinear Static Analysis
	Nonlinear Dynamic Procedure	Nonlinear	Small or Large	Ground Motion History	Nonlinear Response History Analysis

Recent guidelines for seismic rehabilitation of structures pioneered the requirements for dynamic and nonlinear analysis procedures, specifically FEMA 356 (2000a) and the predecessor FEMA 273 (1997). The ATC-40 guidelines for R/C structures (ATC, 1996) emphasize the use of a nonlinear static analysis (pushover analysis) procedure to define the displacement capacity for buildings. The classifying of analysis procedures in Table 2.1 is usually applicable to design regulations for new buildings, for example in the 2000, National Earthquake Hazards Reduction Program (NEHRP) recommended requirements FEMA (2000a) and guidelines for steel moment frame structures FEMA-350 (2000b) and for bridges ATC-32 (1996a). These requirements and guidelines are required for the:

- Selection of the analysis procedure depending on the seismic design class and performance level.
- Structural characteristics (e.g., symmetry or complexity).
- Response characteristics (e.g., the fundamental vibration period and participation of higher vibration modes).
- Amount of data available for developing a model and confidence limits (in a statistical sense) for performance evaluation.

2.1.2 Linear Analysis of Structures

On December 28, 1908, a large earthquake (magnitude 7.5) devastated the city of Messina (Italy) with a loss of 83,000 to 120,000 lives. A special commission was formed by the government to investigate the earthquake and to provide recommendations. According to Housner (2002), this earthquake was answerable for the birth of practical earthquake design of structures, and the commission's report appears to be the first engineering suggestion for earthquake-resistant structures by means of the tantamount static method. The method, apparently proposed by Prof. Panetti, recommended designing the first story to withstand a horizontal force equal to 1/12 the building weight above, and the second and third stories to be designed to withstand a horizontal force equal to 1/8 of the building weight above (Bozorgnia and Bertero, 2004). Gradually the equivalent static method was used in earthquake countries around the world and was later adopted by building codes.

In static linear procedure, there is a direct relative between internal forces and internal deformations. A static analysis in code is performed by subjecting the structure to lateral forces got by scaling down the smoothened soil-dependent elastic response spectrum by a structural system reliant on force reduction factor (R). In this method, it is assumed that the real strength of structure is higher than the design strength and the structure is capable to dissipate energy through yielding.

Component examination involves comparing actions with capacities. In building design codes for example the Uniform Building Code (ICBO, 1997) and the International Building Code (ICC, 2000), component actions due to earthquake effects are calculated using elastic spectral forces divided by a response modification factor (R).

The deformations corresponding to the plastic member capacity are not usually excessive, and assessing them is not an arduous task. Since seismic design was developed as an extension to primary load design, it followed the similar procedure, noticing though that inelastic deformations may be utilized to absorb quantifiable levels of energy leading decrease in the forces for which buildings are designed (Borzi and Elnashai, 2000). Provided buildings to sufficiently ductile so that the inelastic energy dissipation can be attained in a slightly controlled way without endangering the integrity or stability of the structural system (Lu et al., 2001). Evaluation the maximum inelastic deformation demands under a specified earthquake ground motion is very important steps in the evaluation and rehabilitation of structures, (Ayoub and Chenouda, 2009; Lee et al., 2006; Levy et al., 2006; Lin et al., 2003).

Although this work can be done by nonlinear response history analyses, it is more often conducted by simple approximate methods derived from linear Single-

Degree-of-Freedom (SDOF) systems. Using approximate methods (SDOF), the displacement coefficient method (Miranda, 2000; Newmark and Hall, 1982) and the equivalent linear method (Gulkan and Sozen, 1974; Iwan, 1980; Kowalsky et al., 1995), are well known. The former has been applied in the FEMA-273 document (1997) as the nonlinear static procedure of structural rehabilitation, and the latter is the fundamental of the capacity spectrum method adopted in the ATC-40 document (1996) for the evaluation of structures (Lin and Miranda, 2009).

Dynamic analysis started during and after the World War II when research on bomb blast effects on structures and structural dynamics analysing the response of structures to such an excitation (loading) in addition, to for their practical design was started. Since 1960, with the increasing interest in earthquake effects and seismic design area, many books and scientific papers on structural dynamics with applications to analysis and design for earthquake, wind, blast and other dynamic loads have been published (Vitelmo and Yousef, 2004). The dynamic analysis may be either response history analysis (linear or nonlinear) or response spectrum analysis (Table 2.1). It is common in many design procedures to perform a dynamic analysis with a response spectrum representation of the ground motion predictable at the site (Chopra, 2001). The simplest dynamic analysis method is based on a linear model of the structure, which allows use of seismic properties (mode shapes and frequencies) and simplification of the resolution with a modal representation of the dynamic response. An estimation of the maximum structural response can be obtained with response spectrum analysis, or the estimate of the maximum response can be computed by response history analysis (linear) with specific earthquake ground motion records.

The response spectrum is a simple and useful method to evaluate the peak response of a Single-Degree-of-Freedom (SDOF) system subjected to an earthquake excitation. Most structure design codes, such as the Mexico Federal District Code (MFDC) (Engineering, 1992), International Building Code (ICC, 2000), and the National Building Code of Canada (NBC, 1995) are based on estimating lateral load demands through response spectrum plots.

The idea of response spectrum was first presented by Biot (1934), but it is general use in structural design is ascribed to Housner (1953). Professor Housner derived the spectra forum four large US earthquake (M ranging from 6.5 to 7.7). The recording sites were located on rock, stiff soil and deep cohesionless soil. A detailed historical overview of the progression of the response spectrum concept as a simple tool for assessing seismic response of structures is presented in Chopra (2007) and Trifunac (2006). Numerically-developed response spectrum plots for both elastic and inelastic systems started to appear with appearance of digital processors and with commercial availability of strong-motion accelerographs in the mid-1960s and (Trifunac and Todorovska, 2001; Trifunac and Todorovska, 2001b; Veletsos and Newmark, 1960).

Newmark and Hall (1969) were first presented design spectra, and were developed by idealized the calculated response spectrum with a series of straight lines. Newmark and Hall (1982) were also the first offering of the constant-ductility inelastic design response spectrum. The work was followed by some other studies, (Ayoub and Chenouda, 2009; Elghadamsi and Mohraz, 1987; Miranda and Bertero, 1994; Ordaz and Pérez-Rocha, 1998; Riddell et al., 2002).

Inelastic response spectra are characteristically used in performance-based or displacement based design provisions for example the capacity spectrum method adopted by ATC-40 (1996), and the method of coefficients adopted by FEMA356 (2000a). In these design methods, a structure is designed based on its ductility capacity rather than strength. Whereas, current inelastic response spectra plots account for ductility and the ensuing strength reduction, they do not account for cyclic degradation effects, which might result in collapse of the structure. It is known that any material degrades in strength and stiffness under repeated cyclic loadings, which might cause a complete loss of strength and possible failure. Consequently, assessment of earthquake response using current response spectra plots lack the exactness needed for a meticulous design using displacement or performance-based procedures (Ayoub and Chenouda, 2009).

2.1.3 Nonlinear Analysis of Structures

The accuracy and reliability of nonlinear response history analysis in simulating the real behaviour of structure under seismic load has been widely accepted since 1960s. Nevertheless, the time essential for good modelling, input arrangements, calculation time, computer budgets and the exertion for the explanation of voluminous output make use of such analyses impractical. This led researchers to propose simplified nonlinear analysis procedures and structural models to estimation inelastic seismic demands. Dynamic response of a practical structure is complex, and consequently, it is better to begin the study of dynamic behaviour using simple systems. A Single-Degree-of-Freedom (SDOF) system is defined as that in which only one type of motion is possible, or in other words the location of the system at any instant of time can be defined in terms of single coordinate (Sen, 2009). Therefore, the proposed simplified nonlinear analysis procedures and structural models are usually based on the reduction of MDOF model of structures to an equivalent SDOF system (Chopra and Goel, 2002).

Rosenblueth and Herrera (1964) proposed a procedure wherein the maximum deformation of inelastic SDOF system is assessed as the maximum deformation of a linear elastic SDOF system with lower lateral stiffness (higher period of vibration, Teq) and normalized equivalent damping ratios (ζeq) than those of inelastic system. Furthermore, they used the secant stiffness at maximum deformation to represent equivalent damping ratio and period shift is calculated by equating the energy dissipated per cycle in nonlinear and equivalent linear SDOF system subjected to harmonic loading (Miranda and Ruiz, 2002; Rupakhety and Sigbjörnsson, 2009).

Gülkan and Sözen (1974) presented hysteretic damping model. According to the Takeda (1970) hysteretic model and experimental shake table results of small-scale reinforced concrete frames (Lin and Miranda, 2004), Gülkan and Sözen (1974) developed the following experiential equation 2.1 to computed the equal damping ratio.

$$\zeta_{eq} = \zeta_0 + 0.2\left(1 - \frac{1}{\sqrt{\mu}}\right) \quad (2.1)$$

where ζ_{eq} is equivalent damping, ζ_0 is elastic viscous damping, μ is displacement ductility.

The experiential procedure proposed by Gülkan and Sözen (1974) was later extended to the design of reinforced concrete frames modeled as Multi-Degree-of-Freedom (MDOF) systems (Medhekar and Kennedy, 2000). Iwan (1980) and Kowalsky (1995) developed the experimental equations to define the period shift and equivalent viscous damping ratio to estimation maximum displacement demand of inelastic SDOF system from its linear representation.

2.2 Pushover Analysis Procedure

Beginning of nonlinear static analysis (Pushover Analysis) is often attributed to the work of Takeda et al. (1970), Freeman et al. (1975), Saiidi and Sozen (1981) and later, Fajfar and Fischinger (1988) proposed an approach wherein the response of a MDOF system was determined from dynamic response analysis of an equivalent SDOF system. Takeda et al. (1970) proposed force-displacement relationship for calculated dynamic response of an equivalent SDOF system. An advance in the development of simplified nonlinear analysis approaches happened in the late 100's with introduce of many prominent nonlinear static analyses (Pushover Analysis) namely, Capacity Spectrum Method (CSM), Coefficient Method (CM), N2 method and Modal Pushover Analysis (MPA) are explained in more details in next section.

2.2.1 History and Development of Pushover Analysis Procedure

Saiidi and Sözen in (1979) proposed Q-model that was a simplified version of the Takeda model in 1970. The force-displacement relationships of the Q-model are shown in Figure 2.2. The Q-model is low-cost analytical model to estimate displacement histories of multi-story reinforced concrete structures subjected to the ground motions. The model is a SDOF system consists of a concentrated mass supported by a massless rigid bar connected to the ground by a hinge and a nonlinear rotational spring. Furthermore, damping forces are exerted on the mass by a viscous damper. The overall performance of Q-model to simulate response of structures without abrupt changes in stiffness and mass along their heights in earthquake was satisfactory.

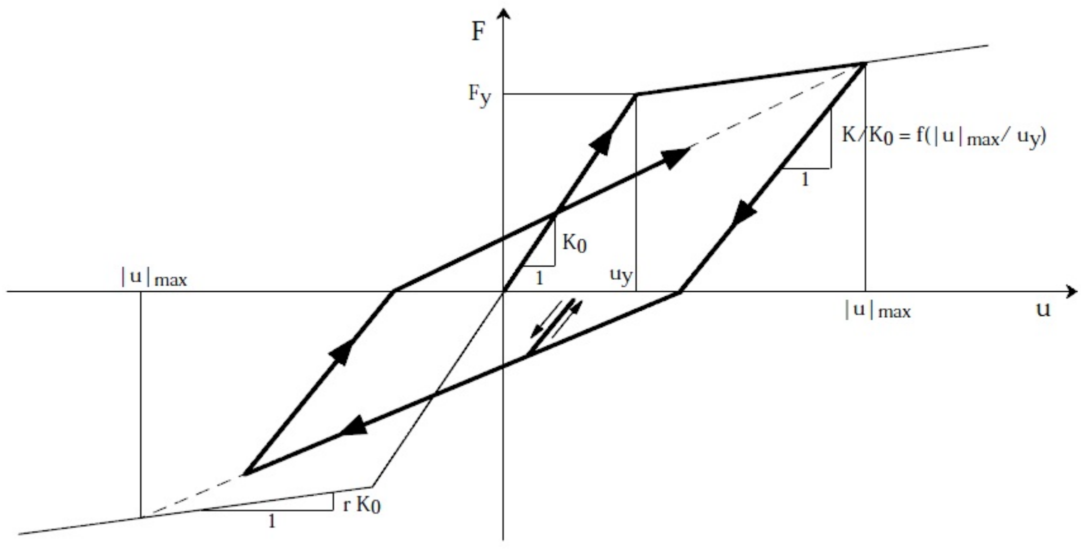

Figure 2.1: Force-Displacement Relationships Defining the Q-model (Lestuzzi and Badoux, 2003).

Fajfar and Fischinger (1987) proposed the N2 method as a simple nonlinear procedure applicable to the logical design of reasonable regular structure oscillating mainly in a single mode. N2 was developed at the University of Ljubljana. Its basic variant has been implemented in Eurocode 8 (CEN, 2005). N2 method is similar to the Capacity Spectrum Method (CSM) but differs in using inelastic response spectra instead of the elastic response spectra. The method was applied to three different 7-story reinforcement concrete buildings. Also the method uses response spectrum approach and nonlinear static analysis. The results show that proposed method is not very sensitive to the details of the equivalent SDOF system (Fajfar and Gašperšič, 1996). An extension of the N2 method proposed important higher mode effects in plan and along the elevation. New version based on the supposition that the structure remainders in the elastic range when vibrating in higher modes, (Kreslin and Fajfar, 2011; Kreslin and Fajfar, 2012).

The capacity spectrum method (CSM), adopted in ATC-40 (1996), was first introduced in the 1970s by Freeman as a fast evaluation procedure for evaluating the seismic vulnerability of buildings (Freeman et al., 1975; Freeman, 1978) . This procedure compares the capacity of the structure (represented by a force-displacement curve) in the form of a pushover curve with demands of earthquake ground motion on the structure in the form of an elastic response spectrum (Figure 2.2). It should be noted that the base shear forces and roof displacements are converted to the spectral accelerations and spectral displacements of an equal SDOF system, respectively (Fajfar, 1999; Lin and Chang, 2003), later, Mahaney et al. (1993) proposed the Acceleration-Displacement Response Spectrum (ADRS) format, that spectral accelerations are plotted against spectral displacements, with the periods (T) represented by radial lines. The crossing of the capacity spectrum and the demand spectrum provides an estimation of the displacement demand and inelastic acceleration (Strength) (Fajfar, 1999). Capacity spectrum method was later updated in FEMA 440 (2006).

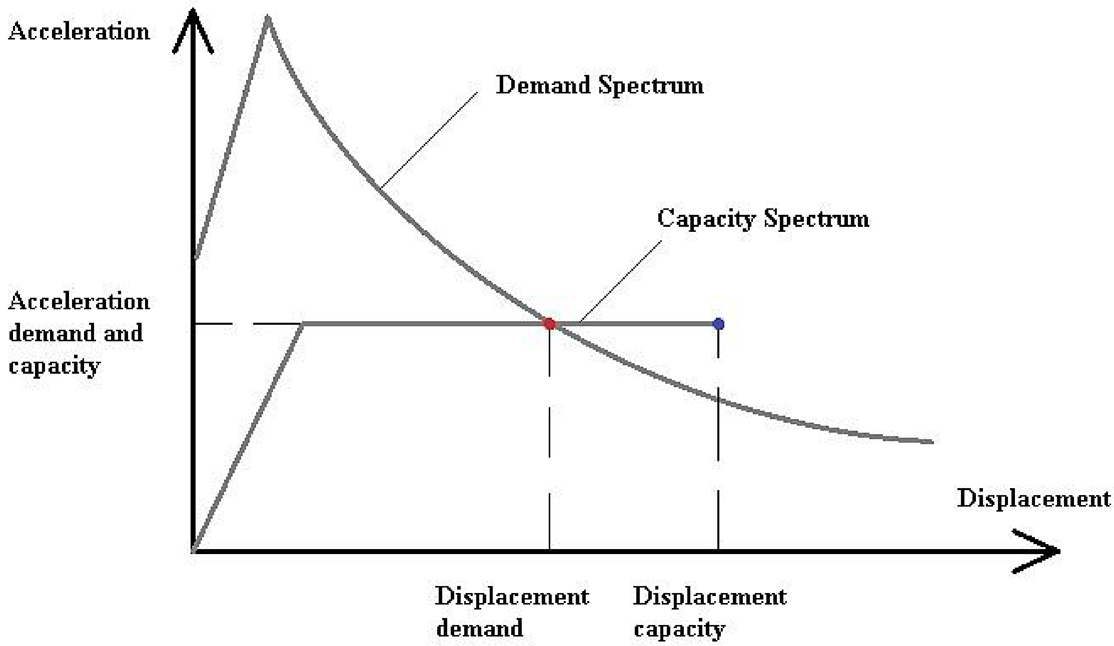

Figure 2.2: Capacity Spectrum Method (CSM)

Newmark and Hall (1982) proposed approaches based on displacement modification factors for estimating inelastic response spectra (MDOF) from elastic response spectra (SDOF). In this method the displacement modification factor varies depending on the spectral area wherein the initial period of vibration of the SDOF system is located in the following method:

$$C = \mu \qquad T < T_a = 1.33 \text{ s} \qquad (2.2)$$

$$C = \frac{\mu}{(2\mu-1)^{\beta}} \qquad T_a \le T < T_b = 0.125 \text{ s} \tag{2.3}$$

$$C = \frac{\mu}{\sqrt{2\mu-1}} \qquad T_b \le T < T_{c'} \tag{2.4}$$

$$C = \frac{T_c}{T} \qquad T_c' \le T \le T_c \tag{2.5}$$

$$C = 1 \qquad T \ge T_c \tag{2.6}$$

where;

$$\beta = \frac{\log(T/T_a)}{2\log(T_b/T_a)} \tag{2.7}$$

$$T_{c'} = \frac{\sqrt{2\mu-1}}{\mu} T_c \tag{2.8}$$

Displacement modification factors computed with Newmark and Hall (1982) Equation are shown in Figure 2.3. Later, Miranda (2000) proposed a statistical study of ratios of maximum inelastic to maximum elastic displacements computed from ground motions recorded on solid soils and proposed the following simplified expression to compute the displacement modification factor:

$$C = \left[1 + \left(\frac{1}{\mu} - 1\right)\exp(-12T\mu^{-0.8})\right]^{-1} \tag{2.9}$$

Displacement modification factors computed with Equation (2.9) are shown in Figure 2.4.

As illustrated in Figure 2.4, overall trend of the displacement modification factors in Miranda’s method is similar to that of Newmark and Hall (1982) method. Moreover, both methods show that inelastic displacements larger than elastic displacements for short periods, and inelastic displacements equal to elastic displacement in the intermediate and long period spectral regions (Miranda and Ruiz, 2002).

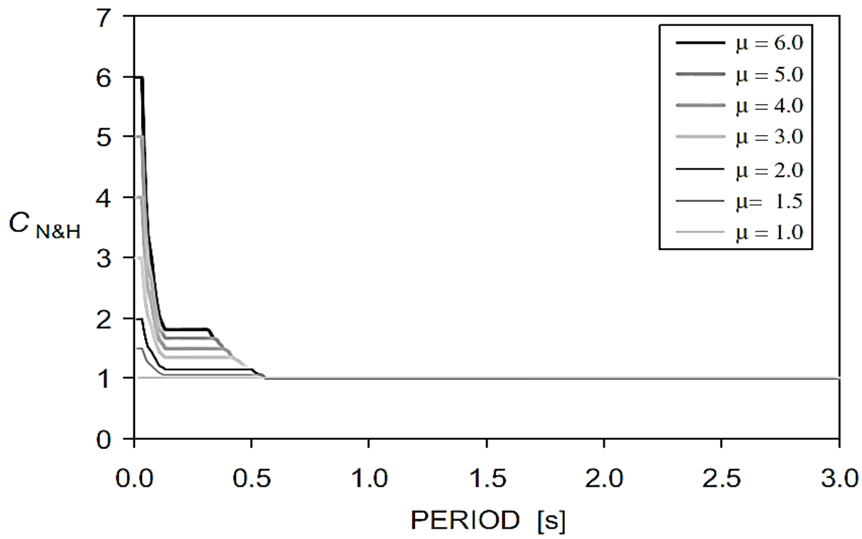

Figure 2.3: Displacement Modification Factors in the Newmark and Hall Method (Miranda and Ruiz, 2002).

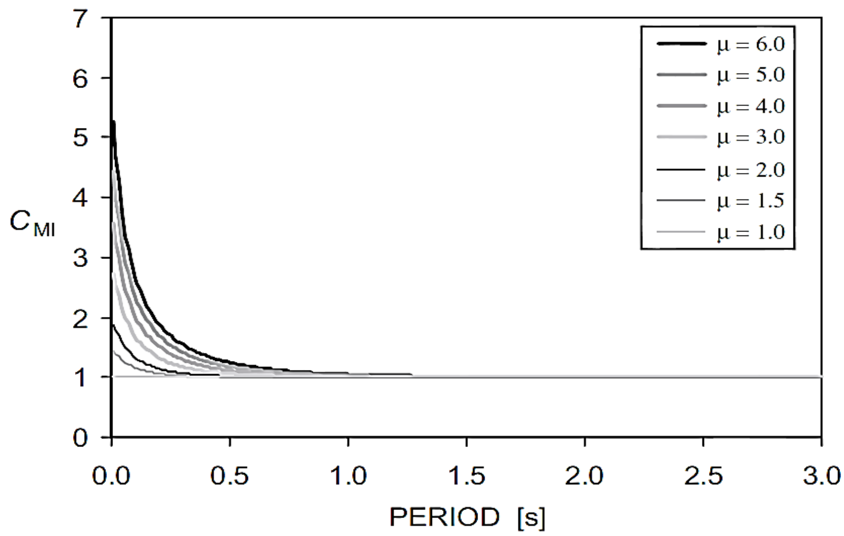

Figure 2.4: Displacement Modification Factors in the Miranda Method (Miranda and Ruiz, 2002).

The Coefficient Method (CM) was first introduced in Federal Emergency Management Agency (FEMA) of the U.S.A (FEMA-273, 1997), and was further developed and published as a pre-standard for seismic rehabilitation of buildings in FEMA-356 (2000a). The method was later updated in FEMA-440 (2006). The displacement demand of the method is determined from the elastic one by using a number of modification factors based on statistical analyses. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients.

According to the coefficient method of FEMA-273 (1997), the target displacement, which is the maximum displacement happening at the top of structures during a selected earthquake, can be determined as:

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{2.10}$$

where C_0 is the differences of displacements between the control node of MDOF structures and equivalent SDOF systems; C_1 is the modification factor for assessing the maximum inelastic deformation of SDOF systems from their maximum elastic deformation; C_2 is the response to possible degradation of stiffness and energy dissipation capacity for structural members during earthquakes; C_3 is the modification factor for the P–Δ effects, that in this study neglected; T_e is the effective periods of evaluated structures; and S_a is the spectral value of acceleration response corresponding to T_e (Lin et al., 2004).

Paret et al. (1996) and Sasaki et al. (1998) proposed Multi-Modal Pushover (MMP) procedure to identify failure mechanisms due to higher modes for structures with significant higher-order modal response. In this method determine the mode shape and period of the building and lateral load patterns for the modes of interest, also each pushover roof displacement and base shear are converted to spectral displacement (Sd) and spectral acceleration (Sa) and plotted in the Acceleration-Displacement Response Spectrum format (ADRS) (Mahaney et al., 1993). Furthermore, the response spectrum is converted to Sd and Sa then plotted on the same ADRS graph as the pushover curves. The intersections of the response spectrum with the pushover curves represent the demands on the structure for that specific ground motion (Sasaki et al., 1998). The procedure is intuitive and in fact provided qualitative information on higher mode effects, which conventional single mode pushover analysis, fails to highlight. Nevertheless, that is not easily to quantify of the effects of these higher modes, since the method does not provide an assessment of the response (Antoniou and Pinho, 2004).

Moghadam and Tso (2002) proposed Pushover Result Combination (PRC) method that was an improvement of the multi-modal pushover procedure. In this method estimation of the maximum seismic response was derived from combining the results of several pushover analyses, which are carried out by a predefined mode-shape as a load pattern. The final response of structure obtained as a weighted summation (by the respective modal participation factors) of the pushover results from each analysis that the first 3 or 4 modes generally are considered, (Antoniou and Pinho, 2004; Poursha et al., 2009; Vassilis and Elnashai, 2005).