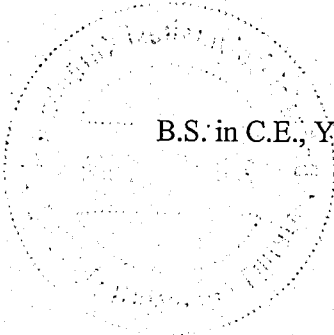


PROBABILISTIC APPROACH TO SEISMIC EVALUATION OF SDOF SYSTEMS:
HIGHWAY BRIDGES

by

Selçuk Altay

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This thesis is dedicated to my grandmother, Hacer.

ABSTRACT

PROBABILISTIC APPROACH TO SEISMIC EVALUATION OF SDOF SYSTEMS: HIGHWAY BRIDGES

In this study, a computational tool was developed using Matlab (The Language of Technical Computing) to perform sectional analysis and to determine the force – displacement relationship, and then the displacement capacity of a fixed-base cantilever column. Also, a nonlinear time history analysis code was written using dynamic analysis procedure, the Newmark's step-by-step Integration Method, to determine the displacement demand of a SDOF system. The inelastic stiffness in dynamic analysis was obtained directly from the force – displacement curve, and a stiffness degrading and pinching hysteretic model was also adapted in the dynamic analysis.

50 artificial earthquake ground motions corresponding to specific response spectrum values were generated and used in the time history analysis to analyze 96 different SDOF structures. Therefore, using the computational tool, first, the force – displacement relationship was found according to a given column cross-section and its displacement capacity was determined. Secondly, using the previously obtained force – displacement relationship, the stiffness backbone curve with hysteretic rules were established and displacement demands were determined. Finally, using 10 different ground motions corresponding to one response spectrum, displacement capacity and displacement demand ratios were obtained, and through Monte Carlo simulation procedure, the probability of failures were determined. The parameters involved in the overall analysis were axial load to ultimate axial load capacity, P/P_0 , ratio, the longitudinal reinforcement ratio, ρ , the cylinder strength, f'_c , and the aspect ratio, L/D .

Finally charts showing the probability of failures were presented for different seismic zones, reinforcement ratios and P/P_0 ratios. The recommendations based on the obtained results are proposed to use for the preliminary seismic screening of SDOF systems.

ÖZET

TEK SERBESTLİK DERECELİ SİSTEMLERİN DEPREMSELLİK DEĞERLENDİRMESİNE OLASILIKSAL BİR YAKLAŞIM: KARAYOLU KÖPRÜLERİ

Bu çalışmada, alt ucu ankastre mesnetli kolonun kesit analizini yapan ve kuvvet – deplasman ilişkisini daha sonra da deplasman kapasitesini saptayan Matlab programlama dilinde (Teknik Hesaplama Dili) hesaplama aracı geliştirilmiştir. Aynı zamanda, tek serbestlik dereceli sistemin deplasman talebini saptamak için dinamik analiz prosedürü, Newmark adım adım İntegrasyon Metodu, kullanılarak lineer olmayan zaman tanım aralığında hesap yönteminin programı yazılmıştır. Dinamik analizde elastik olmayan rijitlik doğrudan doğruya kuvvet – deplasman eğrisinden elde edilmiştir, ve dayanım azalması ve kısıtlanmış histeretik model de dinamik analizde uyarlanmıştır.

96 farklı tek serbestlik dereceli yapıyı analiz etmek için belirli spektrum büyüklüklerine karşılık gelen 50 adet yapay deprem ivme kaydı türetildi ve zaman tanım aralığında hesap yönteminde kullanıldı. Bu nedenle, hesaplama aracını kullanarak, başta, verilen kolon kesit boyutlarına göre kuvvet – deplasman ilişkisi bulundu ve onun deplasman kapasitesi saptandı. İkinci olarak, daha önceden elde edilen kuvvet – deplasman ilişkisi kullanarak, histeretik kurallı rijitlik ana hattı saptandı ve deplasman talebi belirlendi. Son olarakta, bir spektrum büyüklüğüne tekabül eden 10 farklı yer ivme kaydı kullanılarak, deplasman kapasitesi ve deplasman talebi oranları elde edildi, ve Monte Carlo simulasyon prosedürüyle, göçme olasılıkları tespit edildi. Tüm analizde içeren parametreler eksenel kuvvetin eksenel kuvvet taşıma kapasitesine oranı, P/P_0 , boyuna donatı oranı, ρ_1 , eşdeğer silindir dayanımı, f_c' , ve kolon boyunun kesit derinliğine oranı, L/D dir.

Sonuç olarak, farklı deprem bölgelerine, donatı oranlarına ve P/P_0 oranlarına bağlı olarak göçme olasılıklarını gösteren grafikler sunulmuştur. Elde edilmiş sonuçlara dayanılarak, tek serbestlik dereceli sistemlerin depremsellik ön gösterimi için öneriler sunulmuştur.

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LIST OF SYMBOLS / ABBREVIATIONS

$A_{S1,i}$	1 st level i'th reinforcing bar's area
$A_{S2,j}$	2 nd level j'th reinforcing bar's area
b	Column width
C_C	Compressive force of concrete
c	Damping of the system
c^*	The neutral axis depth
c_u	The integration coefficient
D	Column depth
d	Column height
dx	Layer increment
dx^*	Increment length
$d\theta$	Rotation
$d\Delta$	The transverse deflection
f'_c	Cylinder strength of concrete
$f(s)$	Resisting force
Δf	The incremental force
$(\Delta f_s)_i$	Incremental resisting force
i	Reinforcing bar level at the compression
j	Reinforcing bar level at the tension
k	Stiffness of the system
$(k_i)_{sec}$	Secant stiffness at time i
$(k_i)_T$	Tangent stiffness at time i
L	Column length
M	External moment
m	Mass of the system
m^*	Number of reinforcing bar at j'th layer
n	Number of reinforcing bar at i'th layer
P	External force
Δp_i	Increment in $p(t)$ over time step i

P_0	Axial load capacity of the column
$P(t)$	Applied force produced by $m^* \ddot{u}_g(t)$
R	Radius of the curvature
ΔR	Residual force
S_c	Compressive force of steel
S_T	Tensile force of steel
Δt	Time interval
u	The relative displacement with respect to the ground
$\dot{u}$	The relative velocity with respect to the ground
$\ddot{u}$	The relative acceleration with respect to the ground
$\ddot{u}_g$	Ground acceleration vector
$\ddot{u}(\tau)$	The variation of the acceleration over the time interval $t_i \leq \tau \leq t_{i+1}$
$\dot{u}(\tau)$	The variation of the velocity over the time interval $t_i \leq \tau \leq t_{i+1}$
Δu	Incremental displacement
x	Distance from the centroid
x^*	The distance of element
γ, β	Parameters in Newmark's method
ε_s	Steel strain
ε_u	The concrete strain in the extreme compression fiber
$d\theta$	Differential rotation
ρ	Longitudinal reinforcement ratio of the column
σ_c	Stress of concrete
$\sigma_{s1,i}$	1 st level i'th reinforcing bar's stress
$\sigma_{s2,j}$	2 nd level j'th reinforcing bar's stress
φ	Curvature
AASHTO	American Association of State Highway and Transportation Officials
ALADDIN	A Computational Toolkit for Engineering Matrix and Finite Element Analysis
ANSR	Analysis of Nonlinear Structures

ASPIDEA	Nonlinear Analysis of Isolated Bridges under Non-synchronous Seismic Action
ATC	Applied Technology Council
CALANSR	Coupled Nonlinear Seismic Analysis
C / D	The Ratio of Capacity Over Demand
COLA	Column Analysis
FEMA	Federal Emergency Management Agency
FHWA	Federal Highway Administration
ISADAB	Inelastic Static and Dynamic Analysis of Bridges
L / D	Length Over Depth
MicroSARB	Seismic Analysis of Regular Highway Bridges
NEABS	Nonlinear Earthquake Analysis of Bridges Systems
NONLIN	Nonlinear Dynamic Analysis
PGA	Peak Ground Acceleration
PSDA	Probabilistic Seismic Demand Analysis
SDOF	Single Degree of Freedom
SEISAB	Seismic Analysis of Bridges
SMSM	Single Mode Spectral Analysis Method
SSQ	Successive Symmetrical Quadratures
UBC	Uniform Building Code

1. INTRODUCTION

1.1. General

For many years, strong earthquakes occurring around the world have caused tremendous loss of lives and considerable capital damage in human society. Since earthquakes cannot be prevented, society needs to develop effective approaches to mitigate the impact of future strong earthquakes. Researchers and professionals in the field of Earthquake Engineering are making significant and continual contributions to this objective since several decades.

In recent years, it has become apparent that majority of existing bridges in Turkey and in other earthquake-prone countries are inadequate to resist seismic loads (Figures 1-3). Retrofitting all the existing bridges is economically almost impossible and from the perspective of engineering it is not quite practical. But above all these, the detection of the deficient bridges, which is the prior step of retrofitting procedure, requires great effort, capital investment and excessive time.

In order to provide a tool to assess the vulnerability of bridges against strong ground motions, a rational analysis and evaluation method is needed. Thus, it is this research's objective to develop a methodology where the seismic vulnerability of any highway bridge could be rapidly identified by determining its seismic capacity and demand. Software, which was developed for this purpose, assumes a single-degree-of-freedom (SDOF) model for the bridge structure, and evaluates its column's displacement capacity through sectional analysis. Also, seismic displacement demand is determined by using the column's stiffness properties, which is obtained from the capacity analysis, and Newmark's Step-by-Step Integration Numerical Method for the dynamic response of SDOF systems. Finally, comparing the capacity and demand displacements, the vulnerabilities are evaluated. Artificial strong ground motion data and various parameters such as column shear span to cross-section ratio, axial load to ultimate axial load capacity ratio, material strength properties were considered as input data to obtain resulting probabilistic vulnerability assessment. A Monte Carlo simulation technique was used to

evaluate the probability of failure (or survival) of a bridge structure. Here, a high percentage of failure suggests that the bridge structure requires detailed seismic investigation, whereas, and a low percentage of failure indicates that the bridge is adequate to resist the earthquake loads and no further investigation is needed.

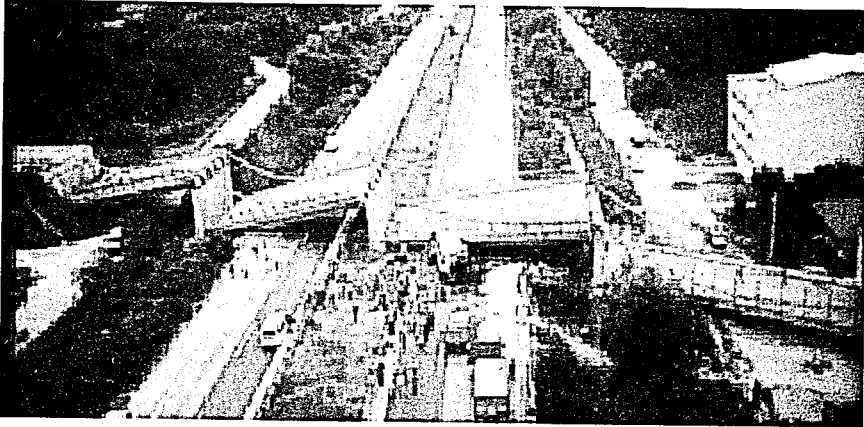


Figure 1.1. Collapse of the Arifiye overpass on TEM Highway in 1999 Kocaeli Earthquake due to inadequate bearing seat width [1]

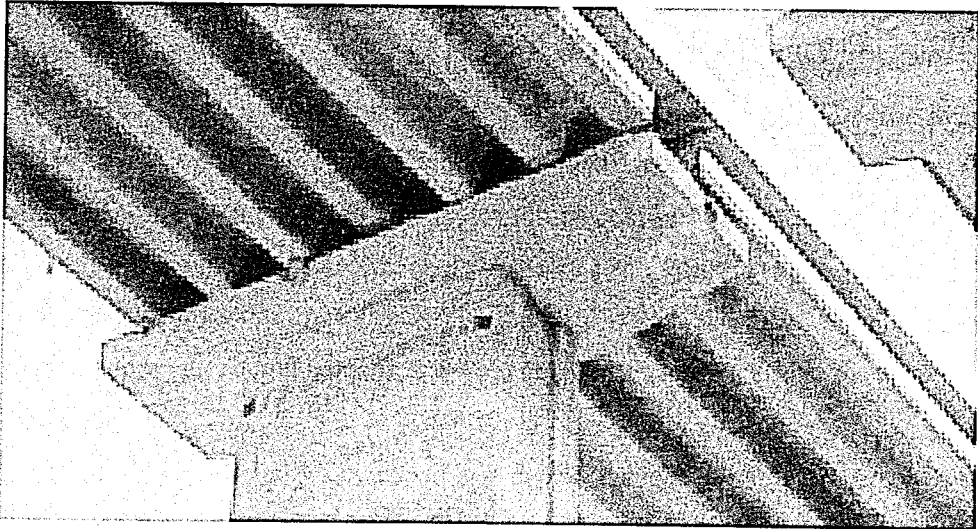


Figure 1.2. Movement between the piers and the girders of Bolu 1 Viaduct [1]

1.2. Literature Review

1.2.1. Seismic Demands of SDOF Systems

In the area of Structural and Earthquake Engineering, there are many methods developed to evaluate the seismic demands of structural systems. Seismic design and evaluation/rehabilitation approaches in current bridge guidelines such as Applied Technology Council (ATC)-6 [2], ATC-18 [3], ATC-18-1 [4], Seismic Design and Retrofit Manual for Highway Bridges [5], and also building provisions such as International Building Code 2000 [6], Federal Emergency Management Agency (FEMA) 302 [7], Uniform Building Code (UBC) [8], FEMA 273 (ATC-33) [9], FEMA 274 (ATC-33) [10], and FEMA 356 [11] explain in detail the use of several static and dynamic analysis procedures; such as equivalent lateral force procedure, modal analysis procedure, capacity spectrum procedure, and nonlinear dynamic analysis procedure.

On the other hand, there are few guidelines for use in the retrofit of existing bridge structures. One of those guidelines is the Seismic Retrofitting Manual for Highway Bridges [12], which was published by the Federal Highway Administration (FHWA) of US Department of Transportation. This manual is the revised form of FHWA's 1983 publication on Seismic Retrofitting Guidelines for Highway Bridges. It describes an evaluation procedure for retrofitting seismically deficient highway bridges and outlines various measures to upgrade these structures. The guidance provided in this manual is applicable in different seismic zones and it also includes a revised preliminary screening procedure. The recommendations are generally compatible with those for the seismic design of new bridges contained in Division I-A of the American Association of State Highway and Transportation Officials' (AASHTO) [13] Standard Specifications for Highway Bridges and should be used in conjunction with these specifications.

ATC-40 [14], originally developed as a rapid evaluation method for a pilot seismic risk project of the Puget Sound Naval Shipyard for the U.S. Navy in the early 1970s, is another guideline, which introduces three nonlinear static analysis procedures. Procedure A is truly iterative, and a formula-based approach where it can easily be programmed to any spreadsheet application. Procedure B simplifies the bilinear modeling of the capacity curve

and that enables a relatively direct solution for the performance point with little iteration. These two methods are mostly analytical methods compared to Procedure C where a pure graphical method to find out the performance point and the most convenient method for the hand analysis.

Chopra and Goel's [15] investigation demonstrated that the peak deformation of inelastic systems computed with analyses procedures described in the ATC-40 report was somehow inaccurate. In particular, the approximate procedure was found to underestimate significantly the deformation for extensive range of periods. In addition, Procedure A in ATC-40 did not necessarily converge, and even it did converge in many occasions, it did not meet the correct peak inelastic displacement.

Fajfar [16] presented a relatively simple nonlinear method for seismic analysis of structures. In his research, demand quantities can be obtained without iteration and one of the approximations made by FEMA 273 and ATC-40 is eliminated.

Miranda [17] also presented two approximate methods to estimate the maximum lateral inelastic deformation demands of SDOF systems. In both methods, maximum inelastic displacements are estimated from maximum elastic displacements as the product of the maximum elastic displacement demand times a modifying factor that is an approximation of the expected value of the ratio of maximum inelastic to maximum elastic displacement. Based on this investigation, it is recommended to use approximate inelastic to elastic displacement ratios that are based directly on statistical results of this ratio and not on statistical studies of the ratio of the lateral forces.

Riddell *et al.*, (2002) [18], in their study on inelastic deformation response of SDOF systems, concluded that inelastic deformations of SDOF systems can be directly obtained from the demand spectrum, which they have developed, without considering the sequence of equivalent linear systems required by the ATC-40 procedures.

The results of the studies of Chopra and Goel [15], Fajfar [16], Miranda [17] and Riddell *et al.*, (2002) [18] have shown that the present criteria for the seismic design of

new structures and for the seismic evaluation of existing structures can be significantly improved.

1.2.2. The Uncertainties of the Structural System

Although, detailed knowledge of the forces that are subjected to the structure is not definitely known, but some information about the static and dynamic excitation is known and could be assumed within certain confidence limits for the evaluation purposes. For example, measurements from previous earthquakes give engineers an opinion on the magnitude and sometimes the frequency content of the ground acceleration expected during an earthquake. Nevertheless, the actual excitations are not periodic, highly irregular and not easily characterized. Time histories recorded from different earthquakes at different times often look very different from each other.

In addition to the uncertainty in the excitation applied to the structures, there is also uncertainty in the mathematical modeling of the system as well. Modeling uncertainty is inherent, as no structural model can completely describe the behavior of a physical system. The models developed are typically based on balance laws, experimental observations, or some combination of the two and often contain a number of parameters, such as elastic modulus, damping ratios, natural frequencies, mode shapes, etc. The values of these parameters, which would give the best agreement between the response of the model and that of the physical system, are not precisely known, and the resulting uncertainty is referred to as 'parametric uncertainty'.

From the uncertainty perspective of the dynamic response of the nonlinear systems, there have been many progress reported. California Institute of Technology's Earthquake Engineering Research Laboratory has made some investigations considering the systems with unpredictability of the input motion and also uncertainty of the system parameters [19-21].

Huang [19] presented an approach for performing second moment analysis on nonlinear dynamic systems with parameter uncertainty. The uncertain parameters were modeled as time-independent random variables. The set of orthogonal polynomials

associated with the probability density function is used as the solution basis. This solution approach was further extended to nonlinear continuous systems involving inhomogeneous random media.

Yang [20] introduced and applied a pragmatic and versatile statistical system identification framework to seismic response records of structures in his research. The framework was based on the interpretation of the probability as a measure of plausibility and on Bayesian statistical inference.

Polidori [21] presented three new methods for analyzing nonlinear dynamic systems under additive stochastic excitation. The first method, probabilistic linearization, finds the Gaussian probability density function that minimizes the Fokker-Planck equation error so that it is expected to work well with systems having small nonlinearities. The second method, a probabilistic nonlinearization, finds a non-Gaussian approximation to the stationary probability density function for the response of the system. The third method provides good estimates to both mean square values and out-crossing rates, as well as having the smallest error in the Fokker-Planck equation.

Carballo and Cornell [22] dealt with issues on structural-system understanding, in light of recent developments in Probabilistic Seismic Demand Analysis (PSDA), which is practical interface between the elastic and inelastic representations of structural behavior and ground motion hazard, in a probabilistic framework. One part of this study deals with requirements of PSDA in terms of structural response representation: efficient and accurate estimation of response where the other part discusses the use of Spectrum Matching Records (also known as Spectrum Compatible Records) for prediction of M-R (Magnitude-Distance) scenario-based displacement demands.

1.2.3. Computer-Aided Structural Analysis Tools

Computer-aided structural analysis tools are now an inevitable part of standard structural engineering practice since they help engineers unravel the basic phenomena of real world structural behavior by their enormous number crunching capabilities. For seismic analysis of highway bridges, many researchers have developed numerous methods

that can help engineers to handle general-purpose problems as well as specific difficulties faced in the Earthquake Engineering field. Most of these methods are utilized in seismic analysis software packages, and some of them could be found on the National Information Service for Earthquake Engineering web server [23]. Table 1.1 contains brief information of these software packages.

These software packages are isolated from the method introduced herein since they do not meet the needs of rapid visual screening procedures. Although the results of these programs are precise, they require structural and earthquake engineering practice and experience for correct data entry, detailed study on the bridge structure, and a careful reading and understanding the program's manual, hence the actual investigation of seismic vulnerability part of the study takes longer time. However, the method introduced in this thesis eliminates evaluation time, and screens and prioritize bridges according to their seismic vulnerabilities. The probabilistic approach is an added feature, which does not exist in any of the above-mentioned software, where it gives an experienced engineer a 'feel' for the current state. Also, it provides a good appreciation of commercial software results.

Table 1.1. Seismic Analysis Software for Highway Bridges

Software	Developer	Description
ANSR	D.P.Mondkar and G.H.Powell	ANSR is a general purpose computer program for static and dynamic analysis of nonlinear structures.
MicroSARB	D.Orie, M.Saidi and B.Douglas	This program implements procedure 1 of the Applied Technology Council (ATC-6) seismic design guidelines for straight regular highway bridges. The ATC-6 procedure 1 employs the Single Mode Spectral Analysis Method (SMSM) for seismic analysis of "regular" highway bridges.
SEISAB	Imbsen & Associates, Inc.	SEISAB is a computer program developed specifically for use in designing new structures to determine retrofit requirements. The program's capabilities include the analysis procedures specified in the current AASHTO Standard Specifications for Seismic Design of Highway Bridges.
NEABS	J. Penzien, R.Imbsen, and W.D.Liu	NEABS performs nonlinear dynamic analysis of long multiple span bridge systems.
ISADAB	M. Saidi, R.Lawver, and J.Hart	ISADAB was developed for the transverse inelastic analysis of reinforced concrete highway bridges.
CALANSR	Pmb Engineering, Inc.	CALANSR is a general purpose structural analysis program specifically developed for fully-coupled, nonlinear seismic analysis. The program can assess ultimate system capacity through "pushover" analyses using specified loading or deformation patterns.
ASPIDEA	R. Giannini, G.Monti, G.Nuti, and T.Pagnoni	APIDEA (A Program for Nonlinear Analysis of Isolated Bridges Under Non-synchronous Seismic Action) is a nonlinear dynamic program specifically developed for the analysis of isolated bridges, using a number of generated accelerograms.

Table 1.1. Seismic Analysis Software for Highway Bridges (Continued)

NONLIN	Finley A. Charney, Ph.D., P.E., Advanced Structural Concepts, Incorporated under a contract with the Federal Emergency Management Agency	NONLIN is a Microsoft Windows based application for the dynamic analysis of single degree of freedom structural systems. The structure may be modeled as elastic, elastic-plastic, or as a yielding system with an arbitrary level of secondary stiffness. The secondary stiffness may be positive, to represent a strain hardening system, or negative, to model P-Delta effects. The dynamic loading may be input as an earthquake accelerogram acting at the base of the structure, or as a linear combination of sine, square, or triangular waves applied at the roof of the structure. The program uses a step-by-step method to solve the incrementally nonlinear equations of motion.
ALADDIN	Mark Austin and Michael Blattau	Aladdin is a computational toolkit for the interactive matrix and finite element analysis of large engineering structures, particularly building and highway bridge structures. Aladdin views finite element computations as a specialized form of matrix computations, matrices as rectangular arrays of physical quantities, and numbers as dimensionless physical quantities. For more information refer to the FHWA Report No: FHWA-RD-99-114

1.2.4. Rapid Visual Screening Procedures

ATC-21 [24] describes an extensive rapid visual screening of buildings for potential seismic hazards. Detailed methodology and supporting background information are contained in the companion ATC-21-1 [25] report. The methodology begins with identifying the primary structural lateral load resisting system and materials of the building. Basic Structural Hazard Scores for various building types are provided on the form, and inspector circles the appropriate one. The inspector then modifies this Basic Structural Hazard score by adding or subtracting Performance Modification Factors in order to arrive at a final Structural Score, S. The final S scores typically range from 0 to about 6, with higher S scores corresponding to better seismic performance. Although, this

procedure is intended to be applicable for all conventional building types, non-building structures such as bridges and large towers are excluded.

On the other hand, literature on rapid visual screening procedures for bridges is limited. Seismic Retrofitting Manual for Highway Bridges, published by FHWA, recommends that structures should be rated according to their need for seismic retrofitting. Seismic Rating System described in this manual may be used as a basis for selecting bridges for more detailed quantitative evaluation. The Seismic Rating System attempts to consider both the technical aspects of the problem and the administrative, economic, and/or political considerations. According to this guideline, Preliminary Screening Procedure gives priority to a bridge for retrofitting rather than a failure percentage or rating/score for a potential seismic hazard.

Literature survey exhibits that there is a need for a tool that considers the engineering aspects (e.g., the capacity of the structure and the displacement demands for an earthquake) and, rapidly and accurately evaluates the potential seismic hazard of highway bridges.

1.3. Research Objectives

The general objective of this research is to develop a tool for assessing the seismic vulnerability of bridge structures using probabilistic approach. Bridge is modeled as a SDOF system and displacement-based analysis methodology is used. Under different ground motions, producing similar acceleration response spectra, drift capacities and drift demands are computed and compared. The following objectives may be summarized as:

- To develop a sectional analysis software in order to determine the moment-curvature and force-displacement relationships, and thus to determine the member's behavior characteristics.
- To develop a new hysteretic model featuring degrading and pinching effects, to represent a realistic reinforced concrete member behavior using the force-displacement backbone curve, which is determined from the sectional analysis procedure.

- To develop a demand software using Newmark's Step-by-Step Integration Method while including the new hysteric reinforced concrete member behavior model.
- To develop a probabilistic method using Monte Carlo simulation to determine the seismic vulnerability.
- To provide charts in order asses and estimate the vulnerability of bridges.

1.4. Thesis Scope and Organization

This procedure is intended to be applicable for all conventional bridge types, which can be assumed as fixed-based cantilever single degree of freedom structures. Building structures and multi degree of freedom systems are excluded. Soil-structure interaction is another issue that is not considered since this topic needs broad investigation and exceeds the needs (e.g., relatively short time, less expenditure and effort) for rapid screening procedure. However, soil types are considered when generating artificial ground motions.

The introduced methodology is meant to be the first phase of detailed evaluation in which selected data are obtained and used to arrive at a preliminary decision as to the potential hazard of the bridge. The subsequent phase consists of more detailed analyses of those bridges identified in the rapid visual screening as potentially hazardous. This procedure can be used for identification of those bridges that require additional, more detailed investigation by qualified engineers and also for prioritization of the bridges to be further investigated, so that technical and other resources could be most effectively utilized.

The research is meant to be fast, accurate and relatively inexpensive, so that a community can develop a list of potentially hazardous bridges without the high cost of a detailed analysis of each bridge.

The rapid screening procedure in this thesis is designed to be a procedure in which the user performs no structural analysis calculations. The task of the user is to obtain the necessary data and input them in the program. A failure percentage is calculated, after the end of the simulation, which is performed by the program.

It should be emphasized that any rapid screening procedure is by definition a very approximate procedure, which could almost certainly fail to identify some potentially seismically hazardous bridges. The goal is to broadly identify most of the potentially hazardous bridges, at a relatively modest expenditure of time and effort, and to eliminate most of the relatively adequate bridges from further review. Lastly, a rapid screening procedure is a methodology intended for rapidly evaluating the hundreds or thousands of bridges in a community. It is definitely not intended for the full determination of the seismic safety of individual bridges.

Chapter 2 describes the fundamentals of sectional analysis and a brief background is given. Also, the approach of the capacity evaluation is explained.

In Chapter 3, some background is given both for the previous researches and the fundamentals of the dynamic analysis of structures. Detailed numerical evaluation procedures of the dynamic response are described and flow charts are presented.

Chapter 4 describes the interpretation of the capacity/demand ratios from the statistical point of view. Approach for the calculation of the possibility for collapse and for failure percentage is explained.

In the last Chapter, conclusions and recommendations are presented.

2. CAPACITY ANALYSIS OF R/C MEMBERS

2.1. Introduction

Reinforced concrete members exhibit a complex nonlinear behavior, both before (elastic) and after yielding (post-elastic or inelastic). The nonlinearity depends on many factors such as member's (or column in this study) cross-sectional geometry, shear span to depth ratio where shear or flexural behavior might be dominant; the confinement, or volumetric reinforcement ratio, near potential plastic hinging zones where spacing of stirrups to determine the column's ductility; the amount of axial load; the amount of longitudinal reinforcement ratio and its splicing details to account for bond slip behavior; and the material strength properties, i.e. concrete and steel stress-strain relationships. Columns could fail by crushing of concrete (lack of confinement or shear-dominant columns resulting brittle failure), by failure initiated by the yielding of longitudinal reinforcing bars (flexural columns, ductile behavior), or by bond slip (not enough development length provided in splicing region). Thus, a close form equation cannot be readily employed to capture the full load-deformation history characteristics of the column. In order to evaluate the behavior and failure mode of columns, a sectional analysis program was developed for flexure-dominant and unconfined columns without considering the splicing details, i.e. no bond failure is assumed.

2.2. Assumptions

In this study, the flexural behavior of a reinforced concrete column under axial load is determined using following assumptions:

- The Bernoulli theorem where plane sections remain plane (deformation due to shear forces are neglected) is valid. In other words, linear distribution of strains over the depth of the section is assumed.
- The reinforcement is subjected to the same variations of concrete strain. It is assumed that there is no slip between concrete and steel.
- The tensile strength of concrete is neglected.

- The concrete stress-strain relationship is adopted from Hognestad's [26] model as shown in Figure 2.1. Confined concrete model is not included in the analysis.
- The stress-strain relationship used for the steel model is assumed to behave linearly in the elastic and yield portions of the curve, forming a bilinear relationship, and the strain hardening portion follows a parabolic function as shown in Figure 2.2. Both behavior in tension and compression is assumed to be the same.
- According to AASHTO [13], members that are not braced against side sway, the effects of slenderness may be neglected where the slenderness ratio, (KL/R) , should be less than 22. In the analysis both slender and short columns are analyzed. However, the critical buckling load is not exceeded. So, no buckling effect is assumed.

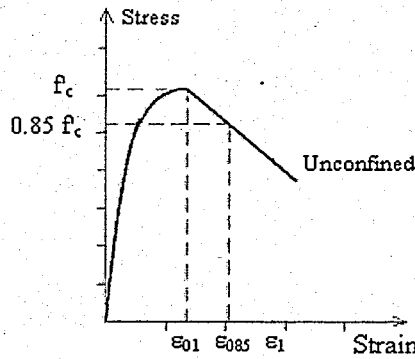


Figure 2.1. Stress-strain relationship for unconfined and confined concrete

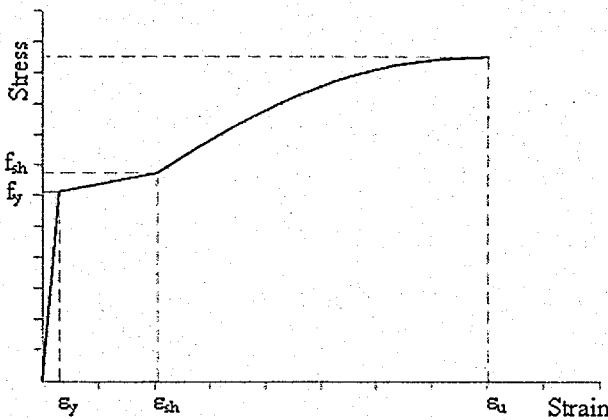


Figure 2.2. Stress-strain relationship for reinforcing steel in tension

2.3. Procedure for Sectional Analysis

When analyzing columns, it is required to know the capacity of the ultimate moment and shear force at the base, and the yield and ultimate displacements at the tip of the column under simulated axial load. Through sectional analysis methodology, the resulting moment-curvature relationship could be determined, and following force-displacement relationship could be found. Thus, under above assumptions, proper column behavior could be determined.

A general cross section of a column, its loading, and applied stress and strain distributions are illustrated in Figure 2.3. The axial load, P , is applied at the centroid of the column with a moment, M . As incremental lateral load is applied, the strain in concrete increases and corresponding resisting forces in concrete and steel are calculated at each increment, and the resulting moment and curvature values are determined.

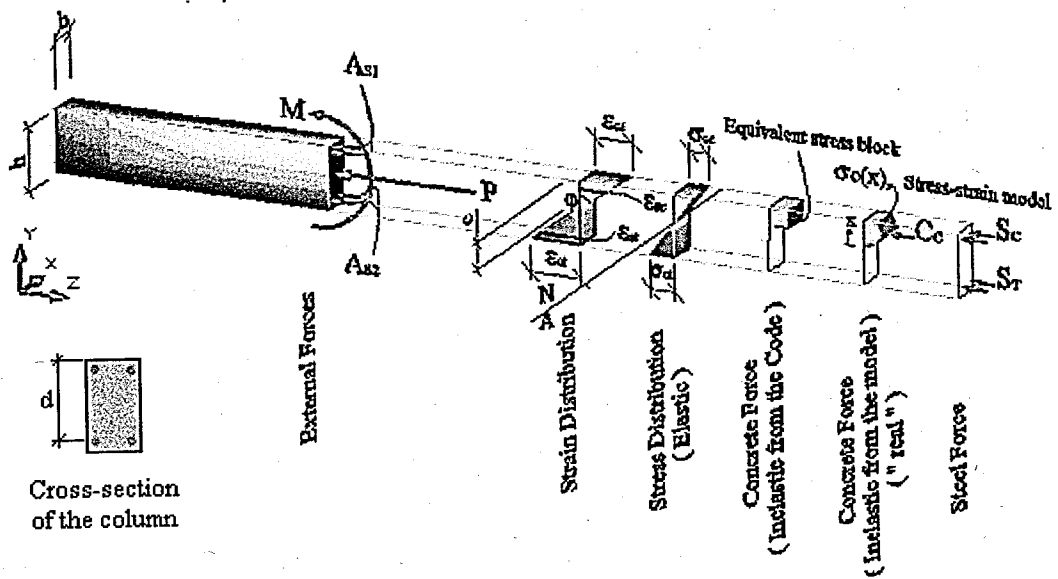


Figure 2.3. Typical stress and strain distribution over a reinforced concrete section

The equations of equilibrium can be obtained by balancing the internal and external forces, and moments about the neutral axis of the column. These internal and external

forces and their respective distances to the neutral axis are shown in Figure 2.3. The related and fundamental equilibrium equations of the section are;

$$P = b \times \int_0^x \sigma_c(x) dx + \sum_{i=1}^n A_{s1,i} \times \sigma_{s1,i} + \sum_{j=1}^m A_{s2,j} \times \sigma_{s2,j} \quad (2.1)$$

$$M = b \times \int_0^x \sigma_c(x) \times x \times dx + \sum_{i=1}^n A_{s1,i} \times \sigma_{s1,i} \times x_1 + \sum_{j=1}^m A_{s2,j} \times \sigma_{s2,j} \times x_2 \quad (2.2) \quad C_c = b \times \int_0^x \sigma_c(x) dx$$

$$S_c = \sum_{i=1}^n A_{s1,i} \times \sigma_{s1,i} \quad (2.4)$$

$$S_T = \sum_{j=1}^m A_{s2,j} \times \sigma_{s2,j} \quad (2.5)$$

where,

$P =$	External Force
$M =$	External Moment
$\sigma_c(x) =$	Stress-Strain Concrete Model
$\sigma_{s1,i} =$	1 st level i'th reinforcing bar's stress, obtained from the steel stress-strain model with respect to steel strain
$\sigma_{s2,j} =$	2 nd level j'th reinforcing bar's stress, obtained from the steel stress-strain model with respect to steel strain
$A_{s1,i} =$	1 st level i'th reinforcing bar's area
$A_{s2,j} =$	2 nd level j'th reinforcing bar's area
$x =$	Distance from the centroid
$dx =$	Layer increment
$n =$	Number of reinforcing bar at i'th layer
$m =$	Number of reinforcing bar at j'th layer
$b =$	Column width
$d =$	Column high
$i =$	Reinforcing bar level at the compression
$j =$	Reinforcing bar level at the tension
$C_c =$	Compressive force of concrete
$S_c =$	Compressive force of steel
$S_T =$	Tensile force of steel

The sign of the stress, σ_s , is compatible with the strain, ε_s , since they are taken from the assumed stress-strain diagram of the steel model. Here, the determination of the neutral axis depth is found on trial and error basis where initially a neutral axis depth is assumed and refined while checking the equilibrium of internal forces with external forces, as shown in Equations 2.1-2.5. Thus, the neutral axis depth is calculated and the required ultimate forces and deformations are determined.

2.4. Moment-Curvature Relationship

The first step in sectional analysis is to determine the moment-curvature relationship where the member's behavior in terms of initial elastic modulus, yield and ultimate moments, and failure mode (ductile or brittle) in inelastic range are clearly shown. Curvature, φ , of an element is defined as the angle of rotation of strain distribution, and it is calculated by dividing the extreme compressive strain of a section to the neutral axis depth. Thus,

$$\varphi = \frac{\varepsilon_c}{c} \quad (2.6)$$

$$\varphi = \frac{\varepsilon_s}{(d - c)} \quad (2.7)$$

or

$$\varphi = \frac{\varepsilon_c + \varepsilon_s}{d} \quad (2.8)$$

where c is the neutral axis depth, ε_c is the concrete strain in the extreme compression fiber, ε_s is the tension steel strain and d is the effective depth of the section.

Figure 2.3 shows a prismatic R/C member with applied external moment and axial load. Theoretical moment-curvature relationship for a given external axial load and moment could be determined by incrementing the concrete strain, ε_c , at the extreme compression fiber. For each value of ε_c the neutral axis depth, c , that satisfies the force

equilibrium is found by adjusting c until the internal forces calculated using Equations 2.3-2.5 satisfies Equation 2.1. This procedure is described in detail by Park and Paulay [27].

2.5. Factors Affecting Moment-Curvature Relationship

2.5.1. Axial Load Effect on Moment-Curvature Relationship

Preceding investigations on the effect of axial load shows that uniaxial loading, when combined with transverse flexure, generally reduces the ultimate moment and curvature of these members. According to Carl Berwanger [28], the reduction in the moment capacity with increased axial load is greater than the reduction in ultimate curvature, when the axial load is greater than the axial load at balanced failure, which causes an apparent increase in ductility. One of the results of column series studied on the effect of axial load on the ultimate moment capacity is shown on the Figure 2.4.

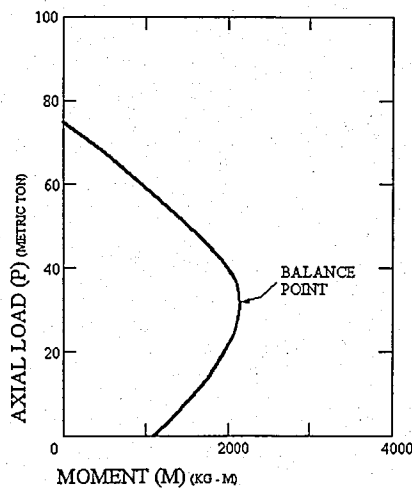


Figure 2.4. Axial Load – Moment Interaction Diagram by Carl Berwanger [28]

2.5.2. The Effect of Shape, Size and Neutral Axis Depth of Cross- Section on Moment – Curvature Relationship

A very important point is arrived at the study of Rüsçh [29]. Most ultimate strength design theories advanced before his study started out from the assumption that the stress distributions in the flexural compression zone, as well as the value of ultimate strain, were constant or at most dependent on the concrete strength. In reality, his research showed that these important design qualities are affected not only by the concrete strength, but even to a greater degree, by the rate of loading, the position of the neutral axis and the shape of the cross-section [27]. The Figure 2.5 apparently shows the effect of the cross-section on the value of ultimate strain.

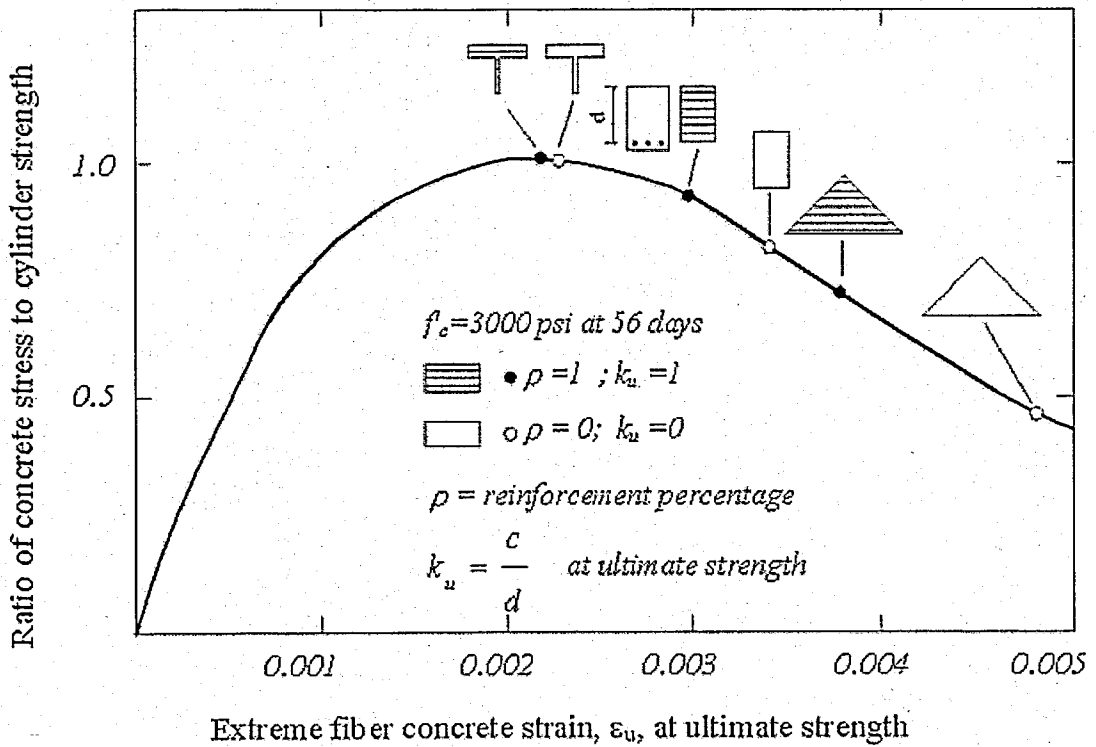


Figure 2.5. Ultimate strength as a function of cross-section and position of neutral axis

2.5.3. The Effect of the Rate of Loading

Concrete stress-strain model, proposed by Hognestad [26], was obtained from short duration loading tests. The effects of small rate of loading and sustained loading are also

studied by Rüsç [29]. He reported that the experimental results of tests on concrete prisms showed an influence of sustained loading on the concrete's compressive stress-strain relationship. Sustained load causes a reduction in the strength of concrete and higher compressive strains develop in flexural strength of the member. Hognestad [26] used Rüsç's sustained load stress block parameters to demonstrate that the strength of columns could be up to 10% less than the short duration loading rectangular stress block parameters. It is clear that the stress-strain relationship obtained from short duration loading tests by Hognestad [26] is satisfactory for the design under all loading conditions.

2.5.4. The Effect of Ratio of Reinforcement and the Relative Rib Area of Reinforcing Bars on Moment – Curvature Relationship

A comparison of different ratios of reinforcement on the effect on moment – curvature relationship was conducted by Pfrang *et al.*, (1964) [30]. The general character of the moment – curvature relationship is not markedly affected by the ratio of reinforcement; regardless of the ratio of reinforcement the cross – section exhibits little ductility for load levels above balance while a certain degree of ductility always exists at low levels below balance. Increasing the amount of reinforcement, however, reduces this ductility. Increasing the ratio of reinforcement decidedly stiffens the section against rotation.

Tholen and Darwin [31] studied the effect of the relative rib area of reinforcing bars on the flexural response. They concluded that for beams with continuous reinforcement, the relative rib area of the reinforcing bars does not affect the distribution of the flexural cracks and the displacement or rotational capacity of beams in which plastic hinges developed.

2.5.5. The Effect of Cover over Reinforcement on Moment – Curvature Relationship

Pfrang *et al.*, (1964) [30] indicated that an increase in the cover over the reinforcement decreases both the stiffness and the strength of the section, but does not

affect its ductility. These effects can be generalized for all load levels except for very small loads. At very small levels of load, increases in cover over reinforcement result in significant decrease in ductility.

2.5.6. The Effect of Yield Stress and Concrete Strength

Aoyama [32] investigated some of the effects of variables and concluded with the results that the reinforcement ratio and concrete strength had minor effects on the shape of moment – curvature diagram. Pfrang *et al.*, (1964) [30] were also in agreement with that investigation except for very small loads, where decrease in ductility is observed with an increase in yield stress.

2.6. Algorithm for the Moment – Curvature Relationship

A useful representation of the theory of sectional analysis is presented by Yalçın and Saatçioğlu [33] by introducing computer software, referred as COLA (Column Analysis). It describes determining of lateral force-displacement relationship derived from moment-curvature analysis using proper concrete and steel models such as confined concrete, buckling of steel, and anchorage slip/extension for circular and rectangular columns. In this research, a similar approach was used, but confinement, reinforcement buckling, and anchorage slip/extension models were excluded.

The cross-section of the column is modeled using a layered approach, or by dividing the cross-section into finite number of horizontal strips, as illustrated in Figure 2.6. Using as many layers as possible would lead more accurate results. Each layer's concrete force is calculated from the corresponding concrete strain and its resulting concrete stress times the strip area. The moment – curvature relationship is obtained by incrementing the concrete strain in the extreme compression fiber, ε_u .

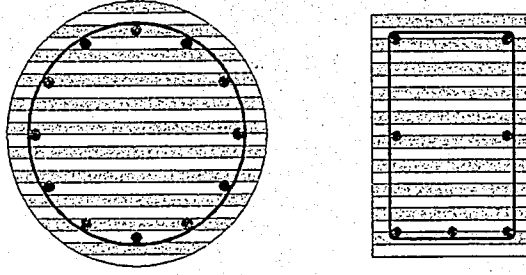


Figure 2.6. Layered cross-section

First, the moment and the corresponding curvature at cracking is calculated. For each value of ε_c , an iterative procedure is used to calculate the neutral axis depth, c . Then the moment corresponding to that value of ε_c is computed. The iterative procedure continues until the equilibrium of the internal and external forces is achieved. The total moment is then calculated by taking moments of the forces corresponding for each strip at its centroid, reinforcements, and forces applied externally about the neutral axis (Equation 2.2.). The curvature is determined from Equation 2.6.

A program was developed using Matlab [34] mathematical software. The input data can be divided mainly into three parts. The first part includes geometric dimensions of the cross-section, and the area and location of the longitudinal reinforcing bars. The second part consists of material strength data for both concrete and steel. The third part includes externally applied forces such as axial load and moment.

The basic algorithm is defined by Park and Paulay [27], and Yalçın and Saatçioğlu [33] as follows:

- Choose a value for c .
- Calculate strains, ε , for each layer of concrete in compression and reinforcing bars in tension and compression and determine the stresses for the respective strains from the stress-strain curves for concrete and steel respectively.
- Check whether the equilibrium is satisfied.
- Repeat steps 1,2 and 3 until a value for c is found that satisfies the equilibrium equations.

- Finally, by taking moment about the neutral axis, the moments and curvature values are obtained.

2.7. Procedure for Obtaining Lateral Force – Displacement Diagram from the Moment Curvature Relationship

The flexural deformation of a member can be calculated by integrating the curvatures along the member.

Figure 2.7 shows fixed-end cantilever column with a deformation due to flexural behavior. Considering that the tangent of a very small angle equals to the angle itself, one can compute the rotation $d\theta$ to be equal to $\frac{dx^*}{R}$. Since the curvature, ϕ , is defined to be the rotation per unit length and also equals to $\frac{1}{R}$, where the rotation can be expressed as ϕdx^* . Here, dx^* is the increment length over the member. Then, the transverse deflection $d\Delta$ at the top from the tangent to the axis of the member at the fixed-end bottom region is $x^* \phi dx^*$, from the similarity of the triangles.

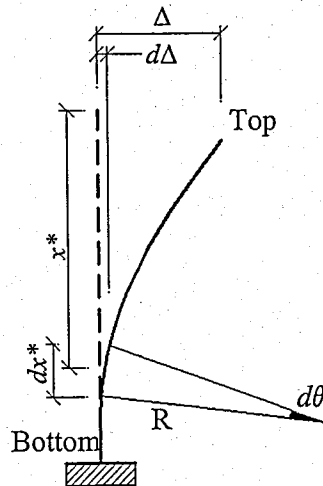


Figure 2.7. Deformation due to flexure

Description given above leads to the Equations 2.10, 2.11 and 2.12.

$$d\theta = dx^* / (1/R) = \varphi dx^* \quad (2.9)$$

$$d\theta = \frac{d\Delta}{x^*} \Rightarrow d\Delta = x^* d\theta \quad (2.10)$$

$$d\Delta = x^* \varphi dx^* \quad (2.11)$$

Thus, the total deflection along the whole length of the member is equal to the integration of the rotation, and is given by the Equation 2.13.

$$\Delta = \int_{bottom}^{top} x^* \varphi dx^* \quad (2.12)$$

where x is the distance of element dx from the top of the member.

Literally, the equation is the generalization of the moment area theorems and it is applicable to both elastic and plastic curvatures. On the other hand, the applied lateral force can be simply obtained by dividing the current moment to the member length.

These procedures are summarized in a flowchart (Figure 2.8).

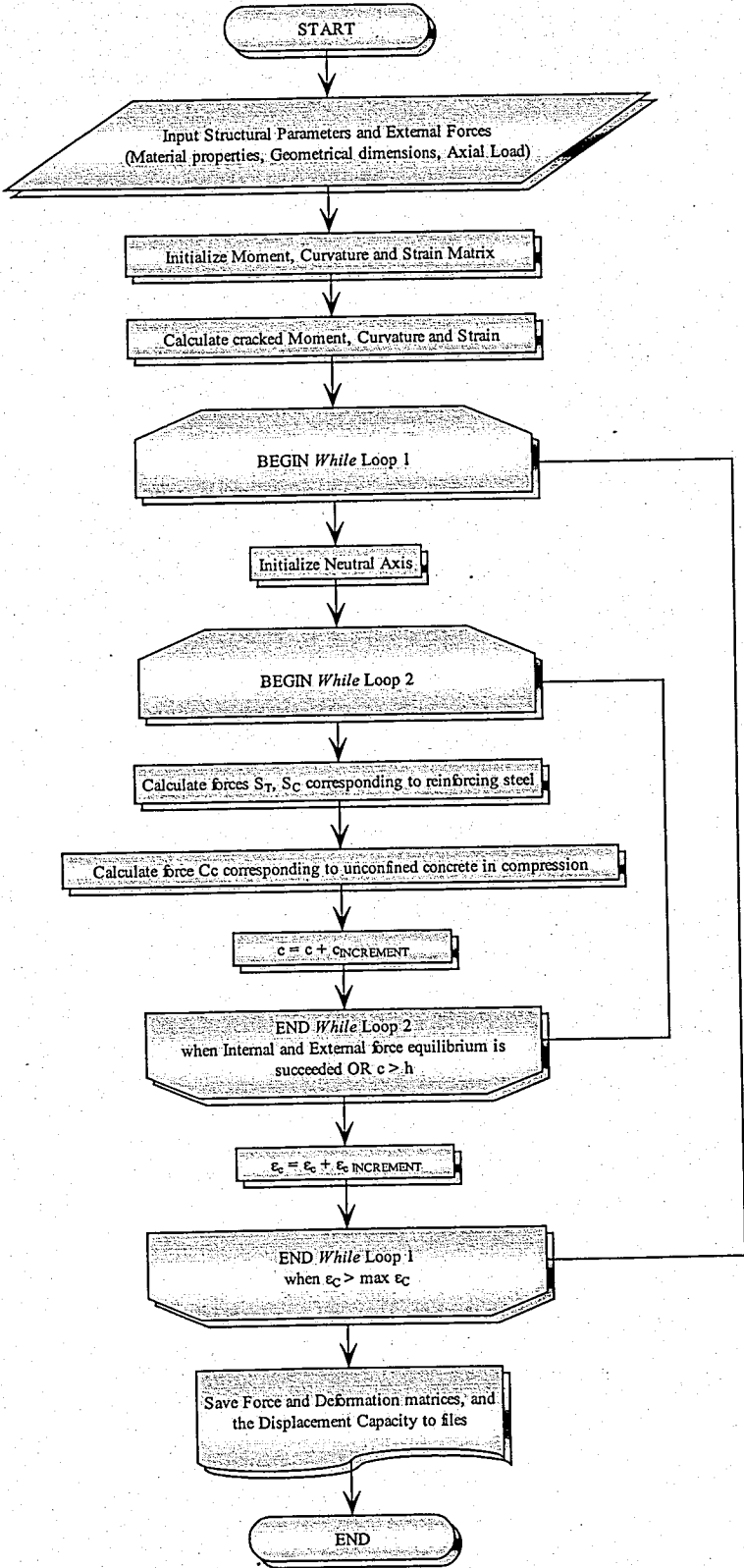


Figure 2.8. Flowchart for the capacity analysis procedure

A basic example is given below in order to provide an understanding of the procedure described above for determining the force – displacement relationship and its corresponding progressive plastic hinging length. For further information see Razvi and Saatçioğlu [35].

Example: Consider the moment – curvature relationship and the column in the Figure 2.8 with height of L . Determine the forces and displacements with respect to these moments and curvatures.

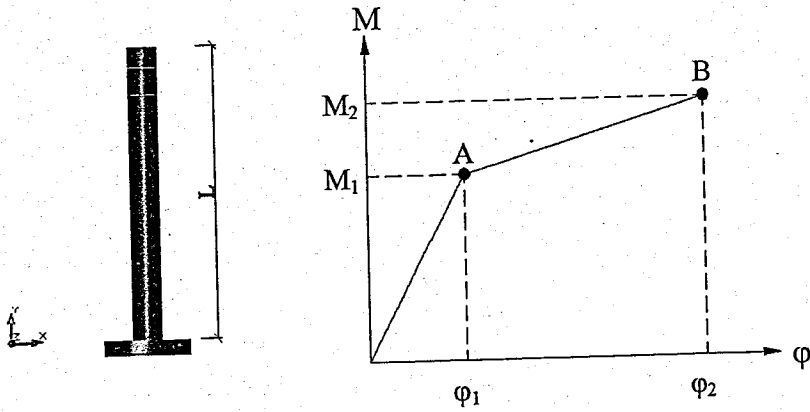


Figure 2.9. Column with height L and the respective moment – curvature

Solution:

For Point A

$$F_1 = \frac{M_1}{L} \quad (2.13)$$

$$\Delta_1 = \left[\phi_1 \times L \times \frac{1}{2} \right] \times (L) \quad (2.14)$$

$$\Delta_1 = \phi_1 \times \frac{L^2}{2} \quad (2.15)$$

For Point B

$$F_2 = \frac{M_2}{L} \quad (2.16)$$

$$\Delta_2 = \left[\varphi_1 \times \frac{L}{2} \times \frac{1}{2} \right] \times \left(\frac{L}{2} \right) + \left[(\varphi_2 - \varphi_1) \times \frac{L}{2} \times \frac{1}{2} \right] \left(\frac{L}{2} + \frac{L}{2} \times \frac{2}{3} \right) + \left(\varphi_1 \times \frac{L}{2} \right) \times \left(\frac{L}{2} + \frac{L}{2} \times \frac{1}{2} \right) \quad (2.17)$$

$$\Delta_2 = \varphi_1 \times \frac{2L^2}{3} + \varphi_2 \times \frac{5L^2}{24} \quad (2.18)$$

2.8. Conclusion

In this research the aim of this chapter is to determine the force – displacement relationship that then will be utilized as the stiffness backbone curve in the dynamic analysis procedure. As it is known, the tangent of the force displacement gives the stiffness of the member and the use and importance of force – displacement relationship will be extensively explained in the dynamic analysis procedures.

3. NUMERICAL EVALUATION OF DYNAMIC RESPONSE

3.1. Preview

An analytical solution of the equation of motion for a single degree of freedom system is almost impossible, especially the forcing function such as ground acceleration varies randomly with time. Another difficulty for the analytical solution arises when the system exhibits an inelastic behavior, which is very difficult to predict and utilize that in the equation of motion. Therefore, many numerical methods have been developed in order to solve the second order differential equation of motion. Since, the solution is numerical evaluation of the differential equation, the accuracy, convergence and stability properties of the numerical procedures become very important.

In this report, Newmark's [36] method of step-by-step integration, which is very useful in dynamic response based on assumed variation of acceleration, is going to be evaluated.

Before the introduction of the methodology, some necessary requirements for numerical procedures are going to be presented.

3.2. Requirements for a Numerical Procedure

3.2.1. Convergence

Since obtaining a general close form equation involving ground acceleration is generally not possible, the problem needs to be solved numerically. Therefore, convergence and accuracy must be considered.

When the time step decreases the solution approaches a unique solution. In other words, as the time step approaches to zero, the limit of the numerical solution should be equal to the exact solution.

$$\lim_{\Delta t \rightarrow 0} (\text{numerical solution}) = (\text{exact solution})$$

3.2.2. Stability

When using any discrete variable method to approximately solve an initial value problem, there are two sources of errors: discretization and round off.

The Global discretization error is the difference between the unique solution and the solution obtained by the discrete variable method.

The local discretization error is the error committed in the single step from time t_i to t_{i+1} . As a result, the numerical solution becomes stable in the presence of these errors. Stability is needed since an earthquake acceleration record and the calculations for each time step are excessive.

3.2.3. Accuracy

Although it depends on the judgment of the analyst, the result obtained from the numerical procedure should be close enough to the exact solution.

3.3. Introduction of the Newmark's Method

Newmark's Method [36] is often used for the solution of structural response problems because the typical choice of integration parameters leads to a procedure that is unconditionally stable. Considering previously discussed requirements, Newmark's Method is the one of the best earthquake response analysis method. Chopra [37] compares some other methods such as Wilson's Method, Central Difference Method, Piece-Wise Interpolation Method to Newmark's Method, and he concluded that Newmark's Method gives the best results and it has a superior accuracy. Williamson and Rungamornrat [38] compared Newmark's Method with Harmonic Acceleration Method and Successive Symmetrical Quadratures (SSQ) method, and the findings of their research was such that SSQ method gives more accurate results. However, this procedure requires approximately

four times as many computations as the Newmark's Method per time step. On the other hand decreasing the time step in the Newmark's method would provide sufficient enough results. Therefore in this project Newmark's Numerical Method is going to be utilized.

Newmark's Method for numerical integration of the equation of motion is based on the assumption that the acceleration varies linearly over a time increment. Newmark [39] developed this relation, which is given in terms of the acceleration at the end of the interval, $\ddot{u}_{i+1}$, the resulting acceleration for next time step.

Two parameters, γ and β are introduced to indicate how much of the acceleration at the end of the interval enters into the relations for velocity and displacement at the end of the step.

The relations, which are adopted, are given below.

$$\dot{u}_{i+1} = \dot{u}_i + (1 - \gamma)\ddot{u}_i\Delta t + \gamma\ddot{u}_{i+1}\Delta t \quad (3.1)$$

$$u_{i+1} = u_i + \dot{u}_i\Delta t + \left[\left(\frac{1}{2} - \beta\right)(\Delta t)^2\right]\ddot{u}_i + \left[\beta(\Delta t)^2\right]\ddot{u}_{i+1} \quad (3.2)$$

Here γ and β also determine the stability and accuracy of the method.

For a system with nonlinear behavior, the equation of motion to be solved is given below.

$$m\ddot{u} + c\dot{u} + f_s(u, \dot{u}) = p(t) \quad (3.3)$$

This is an initial value problem with second order differential equation having conditions of

$$u_0 = u(0), \text{ and } \dot{u}_0 = \dot{u}(0) \quad (3.4)$$

The applied force $p(t)$ is given by a set of discrete values of $p_i = p(t_i)$ and for an earthquake force it can be considered to be equal to $-m\ddot{u}_g(t)$. Where, m is the mass of the system, $\ddot{u}_g(t_i)$ is the ground acceleration at time t_i , c is the damping coefficient, and the responses are displacement, velocity and acceleration and they are denoted by $u_i, \dot{u}_i, \ddot{u}_i$ respectively, $f(s)_i$ is the resisting force of the structure at time i and it is equal ku_i , if the system is linear. Here, k is the stiffness of the structure, and it should be noticed that the stiffness would depend on history of displacement and velocity at the time i if the system exhibits an inelastic behavior.

As a result, this numerical procedure would accommodate the response quantities of $u_{i+1}, \dot{u}_{i+1}$ and $\ddot{u}_{i+1}$ at the time $i+1$, providing the responses of $u_i, \dot{u}_i$ and $\ddot{u}_i$ from the previous time step are known.

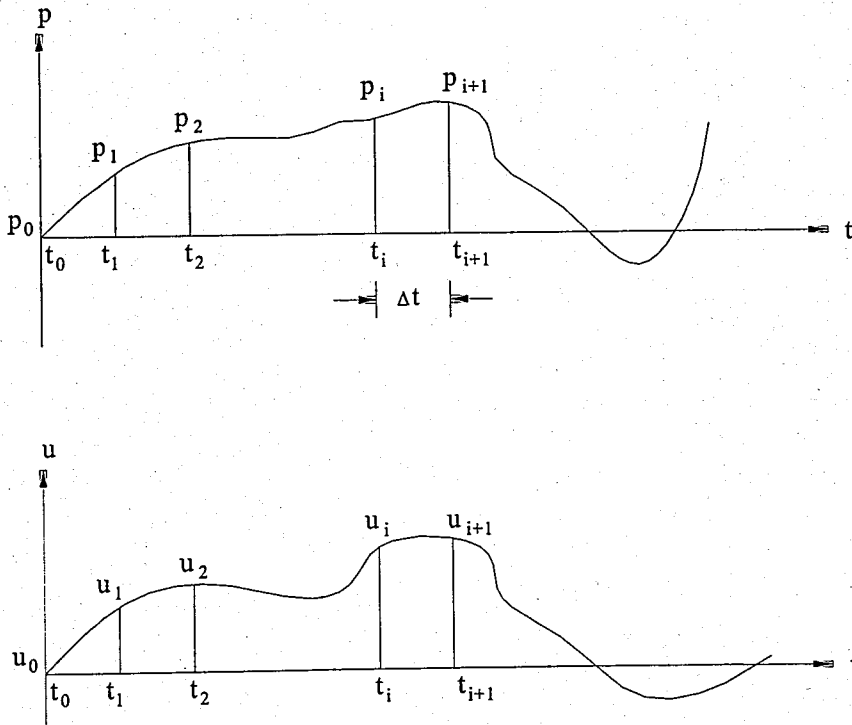


Figure 3.1. Notation for the Newmark's Method

3.4. Average Acceleration Method

In this method it is assumed that the variation of acceleration over a time step is constant and it is equal to the average acceleration of $\ddot{u}_i$ and $\ddot{u}_{i+1}$.

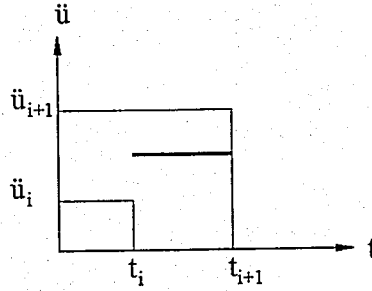


Figure 3.2. Assumption of the acceleration

From the figure above it is seen that

$$\ddot{u}(\tau) = \frac{1}{2}(\ddot{u}_i + \ddot{u}_{i+1}) \quad (3.5)$$

Here τ is a dummy time variable and $\ddot{u}(\tau)$ is the variation of the acceleration over the time step. Integrating the acceleration function (in the time increment), would give the variation of velocity as

$$\dot{u}(\tau) = c_{\dot{u}} + \frac{\tau}{2}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.6)$$

Here, $c_{\dot{u}}$ refers to the integration constant. From the initial conditions of the time step, it is known that when $\tau = 0$, $\dot{u}(0) = \dot{u}_i$ then the above equation can be solved for $\tau = 0$ and the integration constant can be evaluated as shown below:

$$\dot{u}(0) = c_{\dot{u}} + \frac{0}{2}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.7)$$

$$\dot{u}_i = c_u \quad (3.8)$$

So, Equation (3.7) becomes

$$\dot{u}(\tau) = \dot{u}_i + \frac{\tau}{2}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.9)$$

Integration of Equation (3.9) over the time step, would give the variation of the displacement.

$$u(\tau) = c_u + \dot{u}_i \tau + \frac{\tau^2}{4}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.10)$$

Again from the initial conditions of the time step, for $\tau = 0$ $u(\tau)$ is equal to u_i . Then, the integration constant c_u is estimated as $u_i = c_u$, and this is given in Equation (3.11) as

$$u(\tau) = u_i + \dot{u}_i \tau + \frac{\tau^2}{4}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.11)$$

From the end conditions of the time step we know that when $\tau = \Delta t$, $\dot{u}(\tau) = \dot{u}_{i+1}$ and $u(\tau) = u_{i+1}$. Substituting $\tau = \Delta t$ in Equations (3.9) and (3.11), we obtain the desired velocity and displacement quantity at time as

$$\dot{u}_{i+1} = \dot{u}_i + \frac{\Delta t}{2}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.12)$$

and

$$u_{i+1} = u_i + \dot{u}_i \Delta t + \frac{(\Delta t)^2}{4}(\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.13)$$

3.5. Linear Acceleration Method

This method is very similar to the average acceleration method with the exception that the variation of acceleration is assumed to be

$$\ddot{u}(\tau) = \ddot{u}_i + \frac{\tau}{\Delta t}(\ddot{u}_{i+1} - \ddot{u}_i) \quad (3.14)$$

as shown in Figure 3.3.

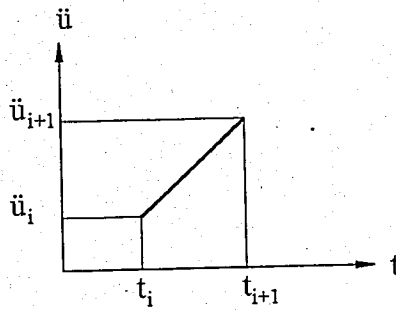


Figure 3.3. Assumption of the acceleration

The derivation of the Linear Acceleration Method is the same as the average acceleration method, and the same procedures are applied. First, the Equation (3.14) is integrated as

$$\dot{u}(\tau) = c_{\dot{u}} + \ddot{u}_i \tau + \frac{\tau^2}{2\Delta t}(\ddot{u}_{i+1} - \ddot{u}_i) \quad (3.15)$$

$c_{\dot{u}}$ is the integration constant and from the initial conditions of the time step it is estimated to be equal to the velocity response $\dot{u}_i$ and it is obtained from the previous time step as

$$\dot{u}(\tau) = \dot{u}_i + \ddot{u}_i \tau + \frac{\tau^2}{2\Delta t}(\ddot{u}_{i+1} - \ddot{u}_i) \quad (3.16)$$

Integration of Equation (3.16) over the time step yields the Equation (3.17).

$$u(\tau) = c_u + \dot{u}_i \tau + \ddot{u}_i \frac{\tau^2}{2} + \frac{\tau^3}{6\Delta t} (\ddot{u}_{i+1} - \ddot{u}_i) \quad (3.17)$$

c_u is the integration coefficient and from the initial conditions of the time step, for $\tau = 0$, $u(0)$ is equal to u_i . Thus, c_u is simply computed and Equation (3.18) is obtained.

$$u(\tau) = u_i + \dot{u}_i \tau + \ddot{u}_i \frac{\tau^2}{2} + \frac{\tau^3}{6\Delta t} (\ddot{u}_{i+1} - \ddot{u}_i) \quad (3.18)$$

The end conditions, for $\tau = \Delta t$, $\dot{u}(\Delta t) = \dot{u}_{i+1}$ and $u(\Delta t) = u_{i+1}$, would provide us the desired responses $\dot{u}_{i+1}$ and u_{i+1} at the considered time increment from t_{i+1} and t_{i+1} .

$$\dot{u}_{i+1} = \dot{u}_i + \frac{\Delta t}{2} (\ddot{u}_{i+1} + \ddot{u}_i) \quad (3.19)$$

and

$$u_{i+1} = u_i + \dot{u}_i \Delta t + \frac{(\Delta t)^2}{6} (\ddot{u}_{i+1} + 2\ddot{u}_i) \quad (3.20)$$

3.6. Modification of Newmark's Method by Noniterative Formulation

It is possible to modify Newmark's Method and to reformulate it by utilizing the incremental quantities such that

$$\Delta u_i \equiv u_{i+1} - u_i, \quad \Delta \dot{u}_i \equiv \dot{u}_{i+1} - \dot{u}_i, \quad \Delta \ddot{u}_i \equiv \ddot{u}_{i+1} - \ddot{u}_i \quad (3.21)$$

and

$$\Delta p_i \equiv p_{i+1} - p_i \quad (3.22)$$

from time i to time $i+1$.

Substituting these incremental quantities into Equations (3.1) and (3.2), the Newmark's original formulation can be rewritten as

$$\dot{u}_{i+1} - \dot{u}_i = \Delta t \ddot{u}_i + \gamma \Delta t (\ddot{u}_{i+1} - \ddot{u}_i) \quad (3.23)$$

$$\Delta \dot{u} = \Delta t \ddot{u}_i + \gamma \Delta t \Delta \ddot{u}_i \quad (3.24)$$

and

$$(u_{i+1} - u_i) = \Delta t \dot{u}_i + \beta (\Delta t)^2 (\ddot{u}_{i+1} - \ddot{u}_i) + \frac{(\Delta t)^2}{2} \ddot{u}_i \quad (3.25)$$

$$\Delta u_i = \Delta t \dot{u}_i + \beta (\Delta t)^2 \Delta \ddot{u}_i + \frac{(\Delta t)^2}{2} \ddot{u}_i \quad (3.26)$$

By rearranging Equation (3.26) and taking $\Delta \ddot{u}_i$ on the left hand side of the equation and all the other components to the right hand side enable us to form the Equation (3.27).

$$\Delta \ddot{u}_i = \frac{1}{\beta (\Delta t)^2} \Delta u_i - \frac{1}{\beta \Delta t} \dot{u}_i - \frac{1}{2\beta} \ddot{u}_i \quad (3.27)$$

Substituting Equation (3.27) into (3.24) gives Equation (3.28).

$$\Delta \dot{u} = \Delta t \ddot{u}_i + \gamma \Delta t \left[\frac{1}{\beta (\Delta t)^2} \Delta u_i - \frac{1}{\beta \Delta t} \dot{u}_i - \frac{1}{2\beta} \ddot{u}_i \right] \quad (3.28)$$

Rearranging this equation by separating displacement, velocity and acceleration components results

$$\Delta \dot{u}_i = \frac{\gamma}{\beta \Delta t} \Delta u_i - \frac{\gamma}{\beta} \Delta \dot{u}_i + \Delta t \left(1 - \frac{\gamma}{2\beta} \right) \ddot{u}_i \quad (3.29)$$

If we write both equations of motion for time i and $i+1$ and subtract from one another we would obtain the incremental equation of motion.

$$m\ddot{u}_{i+1} + c\dot{u}_{i+1} + ku_{i+1} = -m(\ddot{u}_g)_{i+1} \quad (3.30)$$

and

$$m\ddot{u}_i + c\dot{u}_i + ku_i = -m(\ddot{u}_g)_i \quad (3.31)$$

$$m\Delta \ddot{u}_i + c\Delta \dot{u}_i + k\Delta u_i = -m\Delta(\ddot{u}_g)_i \quad (3.32)$$

Equation (3.33) is obtained by substituting Equations (3.27) and (3.29) into the incremental equation of motion of Equation (3.32).

$$m \left[\frac{1}{\beta(\Delta t)^2} \Delta u_i - \frac{1}{\beta \Delta t} \dot{u}_i - \frac{1}{2\beta} \ddot{u}_i \right] + c \left[\frac{\gamma}{\beta \Delta t} \Delta u_i - \frac{\gamma}{\beta} \Delta \dot{u}_i + \Delta t \left(1 - \frac{\gamma}{2\beta} \right) \ddot{u}_i \right] + k\Delta u_i = -m\Delta(\ddot{u}_g)_i \quad (3.33)$$

Rearranging Equation (3.33) by taking the displacement components to the left hand side and all the velocity and acceleration components to the right hand side yields

$$\left[k + \frac{\gamma}{\beta \Delta t} c + \frac{1}{\beta(\Delta t)^2} m \right] \Delta u_i = \left[\frac{\gamma}{\beta} c + \frac{1}{\beta \Delta t} m \right] \dot{u}_i + \left[\Delta t \left(\frac{\gamma}{2\beta} - 1 \right) c + \frac{1}{2\beta} m \right] \ddot{u}_i - m\Delta(\ddot{u}_g)_i \quad (3.34)$$

If we further simplify Equation (3.34) by expressing

$$\hat{k} \Delta u_i = \Delta \hat{p}_i \quad (3.35)$$

where

$$\hat{k} = k + \frac{\gamma}{\beta \Delta t} c + \frac{1}{\beta (\Delta t)^2} m \quad (3.36)$$

and

$$\Delta \hat{p} = \Delta p_i + \left(\frac{1}{\beta \Delta t} m + \frac{\gamma}{\beta} c \right) \dot{u}_i + \left[\frac{1}{2\beta} m + \Delta t \left(\frac{\gamma}{2\beta} - 1 \right) c \right] \ddot{u}_i \quad (3.37)$$

Thus, the desired incremental displacement is computed from

$$\Delta u_i = \frac{\Delta \hat{p}_i}{\hat{k}} \quad (3.38)$$

Next, then incremental velocity and acceleration quantities, $\Delta \dot{u}_i$ and $\Delta \ddot{u}_i$, respectively, can be computed from the Equations (3.27) and (3.29), providing the velocity and acceleration responses $\dot{u}_i$ and $\ddot{u}_i$, respectively, from the previous time step. In that way, the responses, u_{i+1} , $\dot{u}_{i+1}$ and $\ddot{u}_{i+1}$, for the current time step from the incremental formulae of Equation (3.21) can be computed as

$$u_{i+1} \equiv \Delta u_i + u_i, \quad \dot{u}_{i+1} \equiv \Delta \dot{u}_i + \dot{u}_i, \quad \ddot{u}_{i+1} \equiv \Delta \ddot{u}_i + \ddot{u}_i \quad (3.39)$$

But, it should be noticed that after obtaining the response quantities of $\dot{u}_{i+1}$ and u_{i+1} , the acceleration can also be obtained from the equation of motion at time $i+1$ as

$$\ddot{u}_{i+1} = \frac{-m(\ddot{u}_g)_{i+1} - c\dot{u}_{i+1} - ku_{i+1}}{m} \quad (3.40)$$

The algorithm given below summarizes the Newmark's method for linear systems.

- Choose a method

- Average acceleration method $\left(\gamma = \frac{1}{2} \text{ and } \beta = \frac{1}{4} \right)$

- Linear acceleration method $\left(\gamma = \frac{1}{2} \text{ and } \beta = \frac{1}{6}\right)$
- Initial calculations
 - $\ddot{u}_0 = \frac{p_0 - c\dot{u}_0 - (f_s)_0}{m} \quad (3.41)$
 - Select Δt
 - $a = \frac{1}{\beta\Delta t}m + \frac{\gamma}{\beta}c$; and $b = \frac{1}{2\beta}m + \Delta t\left(\frac{\gamma}{2\beta} - 1\right)c \quad (3.42)$
- Calculations for each time step i :
 - $\Delta \hat{p}_i = \Delta p_i + a \dot{u}_i + b \ddot{u}_i \quad (3.43)$
 - $\Delta u_i = \frac{\Delta \hat{p}_i}{\hat{k}} \quad (3.44)$
 - $\Delta \dot{u}_i = \frac{\gamma}{\beta\Delta t} \Delta u_i - \frac{\gamma}{\beta} \Delta \dot{u}_i + \Delta t \left(1 - \frac{\gamma}{2\beta}\right) \ddot{u}_i \quad (3.45)$
 - $\Delta \ddot{u}_i = \frac{1}{\beta(\Delta t)^2} \Delta u_i - \frac{1}{\beta\Delta t} \dot{u}_i - \frac{1}{2\beta} \ddot{u}_i \quad (3.46)$
 - $u_{i+1} = u_i + \Delta u_i, \quad \dot{u}_{i+1} = \dot{u}_i + \Delta \dot{u}_i, \quad \ddot{u}_{i+1} = \ddot{u}_i + \Delta \ddot{u}_i \quad (3.47)$
- Repetition for the next time step. Replace i by $i+1$ and implement the next time step.

3.7. Analysis of Nonlinear Response

Previously described Newmark's Method for linear system can now be extended to nonlinear systems. The only difference is that the stiffness of the system varies for any displacement takes place, so that it should be computed at the beginning of each time step. However, a difficulty arises when predicting the stiffness since an initial displacement value is needed. Figure 3.4 describes this graphically.

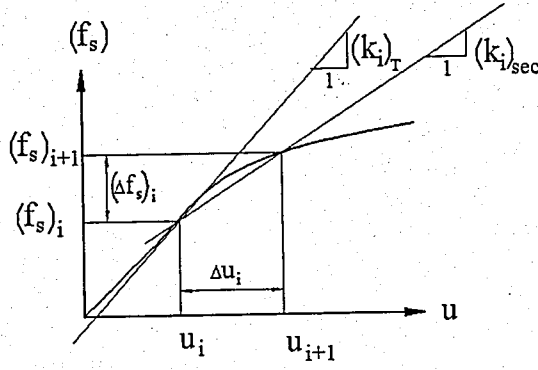


Figure 3.4. The secant and tangent stiffness

In order to compute the response u_{i+1} , $(k_i)_{sec}$ is needed. This problem could be solved by taking the stiffness of the previous time step. This assumption is quite reasonable since using an iterative approach it is possible to compute the responses close enough to the exact solution.

Recalling the incremental equation of motion of Equation (3.32),

$$m\Delta\ddot{u}_i + c\Delta\dot{u}_i + (\Delta f_s)_i = \Delta p_i \quad (3.48)$$

and the incremental resisting force,

$$(\Delta f_s)_i = (k_i)_{sec} \Delta u_i \quad (3.49)$$

then the incremental resisting force would be approximated by

$$(\Delta f_s)_i \cong (k_i)_T \Delta u_i \quad (3.50)$$

Substituting Equation (3.50) into the incremental equation of motion gives

$$m\Delta\ddot{u}_i + c\Delta\dot{u}_i + (k_i)_T \Delta u_i = \Delta p_i \quad (3.51)$$

The next steps are the same procedures as described for the linear systems.

3.7.4. Treatment of the Stiffness

In this study the backbone curve is taken from the sectional analysis computations, and from this curve a new model for the force-deformation relationship is developed. In the Figure 3.5 a typical backbone curve is shown.

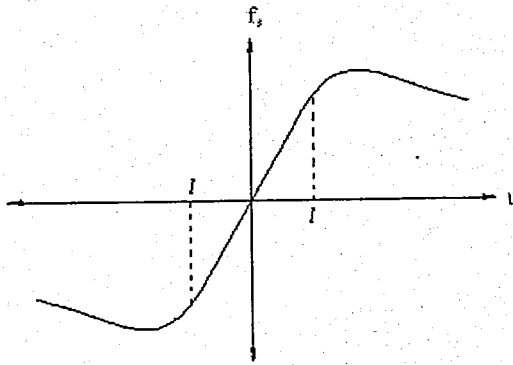


Figure 3.5. Typical backbone curve

Here, the interval, denoted by “ l ”, is the part, which the system behaves linearly and the stiffness is going to be constant. However, rest of the curve exhibits a nonlinear behavior and the stiffness is going to be computed at each time step. The hysteretic model follows the backbone curve and during unloading, up to the point, which the force changes its sign, the curve would be parallel to the curve, which is in the elastic range. The point that the force changes sign would be connected to a point, which has a maximum displacement with an opposite sign. This rule will be followed in each time step. The Figure 3.6 represents a typical hysteretic model which includes the above-mentioned hysteretic rules.

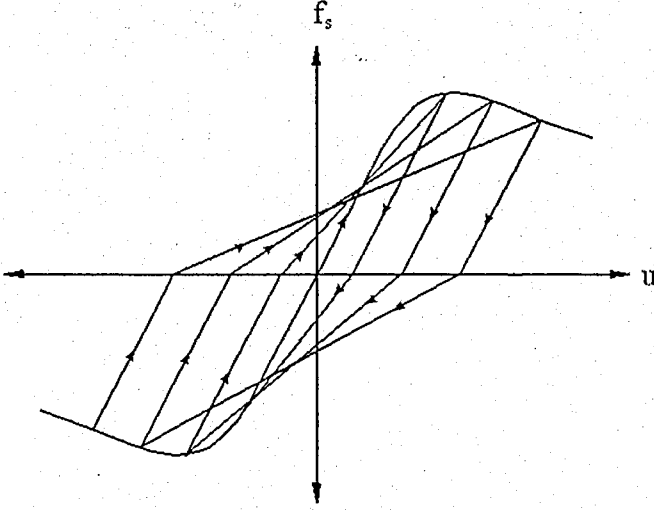


Figure 3.6. Model for a hysteretic behavior

But, as it mentioned before, iteration is needed for predicting the second stiffness, since the tangent stiffness is going to be used instead. The Equation (3.35) which is used in the iteration, is modified for nonlinear systems in such a way that

$$\hat{k}_i \Delta u_i = \Delta \hat{p}_i \quad (3.52)$$

Here, $\Delta \hat{p}_i$ is the same as described for linear systems, and

$$\hat{k} = k_r + \frac{\gamma}{\beta \Delta t} c + \frac{1}{\beta (\Delta t)^2} m \quad (3.53)$$

The only difference from the linear systems is that the tangent stiffness is used instead of the stiffness, which is in the elastic range used for linear systems.

In the first iterative step the aim is to find Δu_i , which is denoted by $(\Delta u_i)^{(1)}$. Here, the superscript (1) refers to the first iterative step. As shown in Figure 3.7, it is seen that $(\Delta f)^{(1)}$ is the corresponding incremental force and it is less than $\Delta \hat{p}$. This means that there exists residual force, which can be computed as shown below.

$$(\Delta R)^{(2)} = \Delta \hat{p} - (\Delta f)^{(1)} \quad (3.54)$$

Here, the superscript (2) refers to the second iterative step since it is going to be used to compute $(\Delta u)^{(2)}$ as

$$\hat{k}_T (\Delta u)^{(2)} = (\Delta R)^{(2)} \quad (3.55)$$

Then, this additional incremental displacement is used to determine a new value of the residual force, and the process is continued until the last incremental displacement $\Delta u^{(j)}$ becomes small enough compared to the current estimate value of Δu , which is the total of the incremental displacements in that time step.

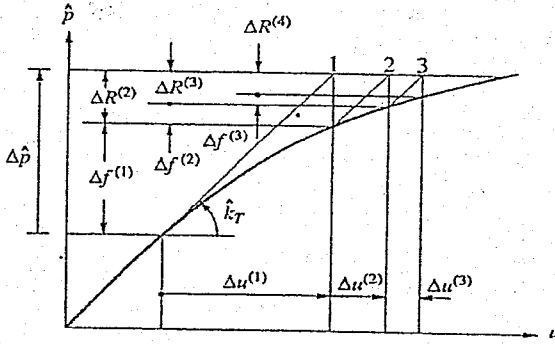


Figure 3.7. Iteration within a time step for nonlinear systems (modified New-Raphson iteration) [37]

This iterative process is known as the Modified Newton-Raphson Method and its algorithm is given below [37].

- Initialize data:
 - $u_{i+1}^{(0)} = u_i \quad f_s^{(0)} = (f_s)_i \quad \Delta R^{(1)} = \Delta \hat{p}_i \quad \hat{k}_T = \hat{k}_i$ (3.56)

- Calculations for each iteration, $j = 1, 2, 3, \dots, n$:
 - 2.1 Solve: $\hat{k}_T \Delta u^{(j)} = \Delta R^{(j)} \Rightarrow \Delta u^{(j)}$ (3.57)

$$\circ \quad 2.2 \quad u_{i+1}^{(j)} = u_{i+1}^{(j-1)} + \Delta u^{(j)} \quad (3.58)$$

$$\circ \quad 2.3 \quad \Delta f^{(j)} = f_s^{(j)} - f_s^{(j-1)} + (\hat{k}_T - k_T) \Delta u^{(j)} \quad (3.59)$$

$$\circ \quad 2.4 \quad \Delta R^{(j+1)} = \Delta R^{(j)} - \Delta f^{(j)} \quad (3.60)$$

- Repetition for next iteration. Replace j by $j+1$ and repeat calculation steps of 2.1 to 2.4

An attention should be given to the iterative step 2.3. In the static analysis, $\Delta f^{(j)} = f_s^{(j)} - f_s^{(j-1)}$. The remaining terms in the equation of step 2.3 come from the dynamics of the system. These terms involve the damping coefficient, c and the mass, m of the system and their multipliers.

Convergence is assumed to be achieved after n iterations when $\Delta u^{(n)}$, obtained from the last iterative step and until it is small enough compared to the total incremental displacements. Thus,

$$\Delta u = \sum_{j=1}^n \Delta u^{(j)} \quad (3.61)$$

Convergence could be evaluated by the formula

$$\frac{\Delta u^{(n)}}{\Delta u} < \varepsilon \quad (3.62)$$

Here, ε is a small value, or a tolerance limit, and it depends on the analyst or the accuracy of the analysis.

Once the convergence criterion is achieved, the displacement increments give the accurate value of Δu_i in the time step from time i to $i+1$. In the Newmark's Method, the Δu_i value, which is obtained without iteration, is replaced by the Δu_i which is obtained after the necessary iterations.

A computer program is developed using Matlab programming tool according to the derivations and formulations explained above. The algorithm of the program is given below and the respective flowchart is shown in Figure 3.8.

- Choose a method:

- Average acceleration method $\left(\gamma = \frac{1}{2} \text{ and } \beta = \frac{1}{4}\right)$
- Linear acceleration method $\left(\gamma = \frac{1}{2} \text{ and } \beta = \frac{1}{6}\right)$

- Initial calculations:

- $$\ddot{u}_0 = \frac{p_0 - c\dot{u}_0 - (f_s)_0}{m} \quad (3.63)$$

- Select Δt

- $$a = \frac{1}{\beta\Delta t}m + \frac{\gamma}{\beta}c; \text{ and } b = \frac{1}{2\beta}m + \Delta t\left(\frac{\gamma}{2\beta} - 1\right)c \quad (3.64)$$

- Calculations for each time step i :

- $$\Delta\hat{p}_i = \Delta p + a\dot{u}_i + b\ddot{u}_i \quad (3.65)$$

- Determine the tangent stiffness k_i

- $$\hat{k} = k_i + \frac{\gamma}{\beta\Delta t}c + \frac{1}{\beta(\Delta t)^2}m \quad (3.66)$$

- Solve for Δu_i from $\hat{k}_i$ and $\Delta\hat{p}_i$ using the iterative procedure

- $$\Delta\dot{u}_i = \frac{\gamma}{\beta\Delta t}\Delta u_i - \frac{\gamma}{\beta}\Delta\dot{u}_i + \Delta t\left(1 - \frac{\gamma}{2\beta}\right)\ddot{u}_i \quad (3.67)$$

- $$\Delta\ddot{u}_i = \frac{1}{\beta(\Delta t)^2}\Delta u_i - \frac{1}{\beta\Delta t}\dot{u}_i - \frac{1}{2\beta}\ddot{u}_i \quad (3.68)$$

- $$u_{i+1} = u_i + \Delta u_i, \quad \dot{u}_{i+1} = \dot{u}_i + \Delta\dot{u}_i, \quad \ddot{u}_{i+1} = \ddot{u}_i + \Delta\ddot{u}_i \quad (3.69)$$

- Repetition for the next time step: replace i by $i+1$ and implement steps 2.1 to 2.7 in the next time step.

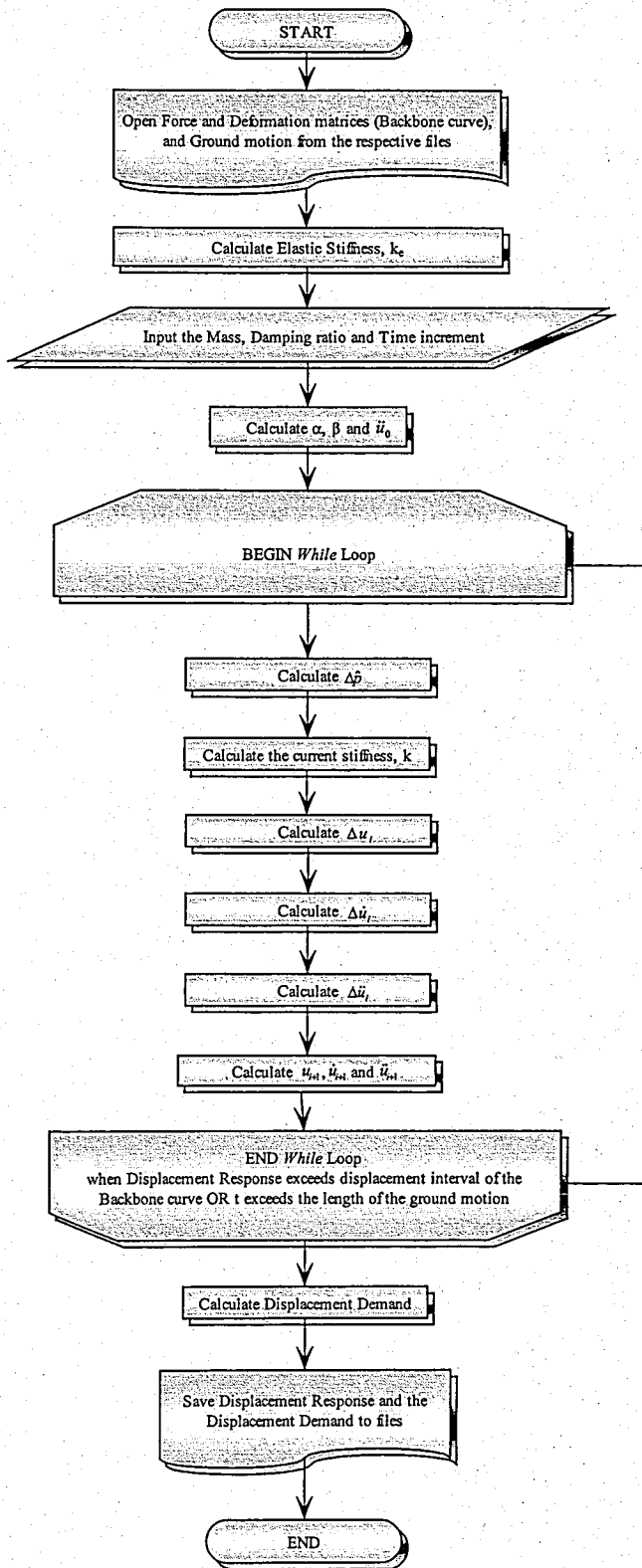


Figure 3.8. Flowchart for the dynamic analysis

4. PROBABILITY OF FAILURE

In this Chapter, first the displacement capacity of the column is calculated as described in the capacity procedure. Then, the displacement demand with respect to Force – Displacement relationship is computed. From the results of these computations capacity/demand ratios are evaluated. Finally the probability of collapse (or failure) percentage is estimated and interpreted from statistical point of view. In this interpretation, Monte Carlo Method [39] is used. As a result, a tool, which gives the probability of failure of SDOF system, is developed. The objective of this Chapter is to obtain the probability of failure for SDOF structures with different parameters. This undertaking can be achieved with the use of herein developed tool. The parameters that separate each SDOF system from one another are given below in Table 4.1.

Table 4.1. Parameters that separate each SDOF system from one to another

Parameter Number	Parameter	Description
1	P/P_0	The ratio of applied axial load over the axial load capacity of the column
2	ρ	Longitudinal reinforcement ratio of the column
3	L/D	Aspect ratio, defined as the ratio of the column length over the column depth
4	f'_c	Cylinder strength of concrete
P is the applied axial load P_0 is the axial load capacity of the column L is the column length D is the column depth		

Considering different seismic zones and parameters given in Table 4.1, all SDOF structural models are analyzed using the herein developed procedures. For each seismic zone earthquake, acceleration records are generated [40]. In order to obtain a reliable

probability of failure percentage, a large amount of earthquake motions are generated and utilized instead of using only one ground motion record.

All analyses are performed by a program developed in MATLAB [34]. These analysis procedures, performed for a SDOF system, are summarized in a flowchart (Figure 4.1).

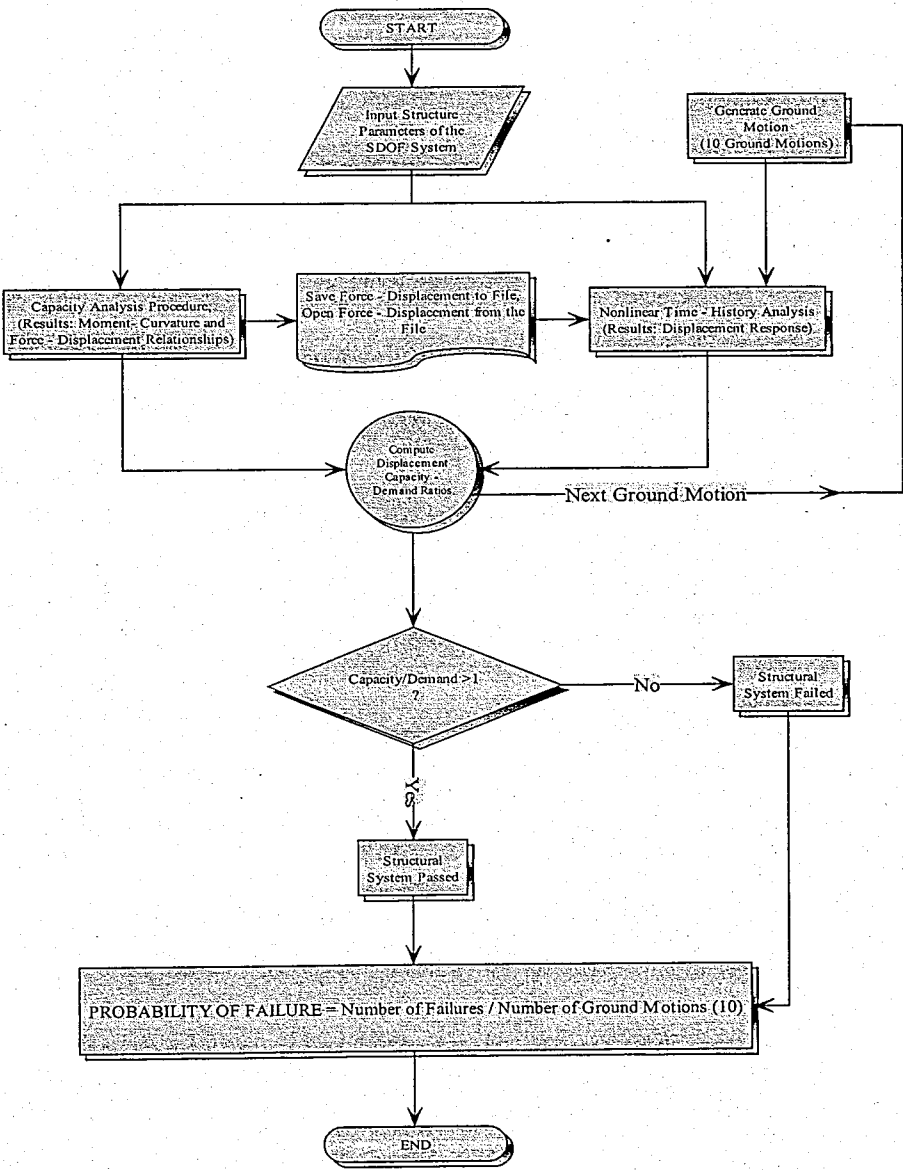


Figure 4.1. Flowchart for obtaining the required Probability of Failure of a SDOF system

4.1. Aspect Ratio Criterion

Constant reinforcement ratios, cylinder strengths, P/P_0 and aspect ratios with different column lengths and depths are selected. Each SDOF structure is analyzed. Both displacement capacities and demands are evaluated. The results showed that the constant aspect ratios with different column heights and depths give approximately the same capacity – demand ratios within close proximity. Verifications are made for aspect ratios between 3 and 9 and the results obtained herein are valid only for that range of aspect ratios. Inputs used for verifications and the results obtained from the analyses are shown in Tables 4.3.-4.11. Also, charts exhibiting the Capacity/Demand ratios (C/D) versus aspect ratios are plotted (Figures 4.7. through 4.11).

4.2. Description of Parameters

In this research four different P/P_0 , three reinforcement ratios, four aspect ratios and two cylinder strengths are considered. From the combination of these parameters, 96 different SDOF systems are obtained and analyzed. For each probable Peak Ground Acceleration (PGA), 10 earthquake motions are generated, so that 960 analyses are performed.

Table 4.2. Assumed parametric values for the analyses

P/P_0	ρ	L/D	f'_c (MPa)	PGA
0.05	0.01	3	20	0.40g
0.10	0.02	5	25	0.35g
0.15	0.03	7		0.30g
0.20		9		0.25g
				0.20g

It should also be noticed that the generated earthquake motions are assumed to be for soil type Z3. According to the Turkish Earthquake Code [41] spectrum characteristic periods for soil type Z3 are $T_A = 0.15$ s and $T_B = 0.60$ s. Peak Ground Accelerations used for the generation of the ground motions are 0.40g, 0.35g, 0.30g, 0.25g and 0.20g.

According to the Turkish Earthquake Code Peak Ground Accelerations for Seismic Zone 1, Seismic Zone 2 and Seismic Zone 3 are $0.40g$, $0.30g$, and $0.20g$ respectively. In order to obtain a refine solution, intermediate Peak Ground Accelerations for the specified seismic zone are also included.

A sample analysis is performed by utilizing the developed computational tool. The structural parameters and the ground motion record are given below in Figure 4.2-4.3 respectively. And the outputs from the capacity and dynamic analyses are given in Figure 4.4-4.6.

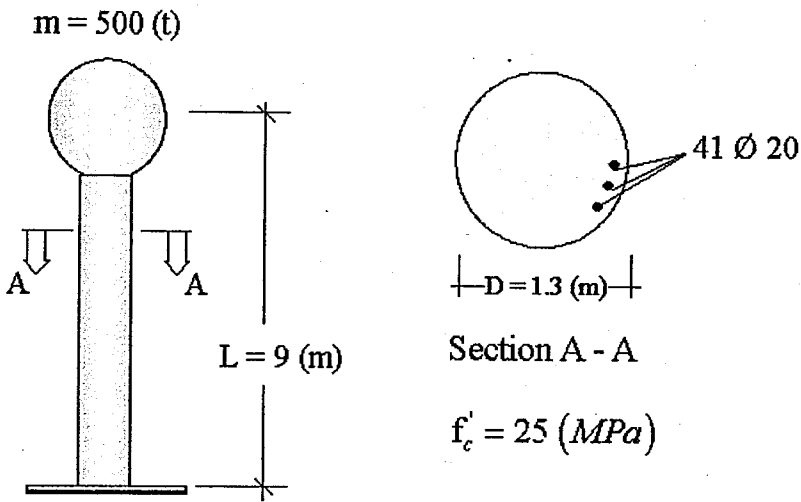


Figure 4.2. Sample SDOF system and its respective input parameters

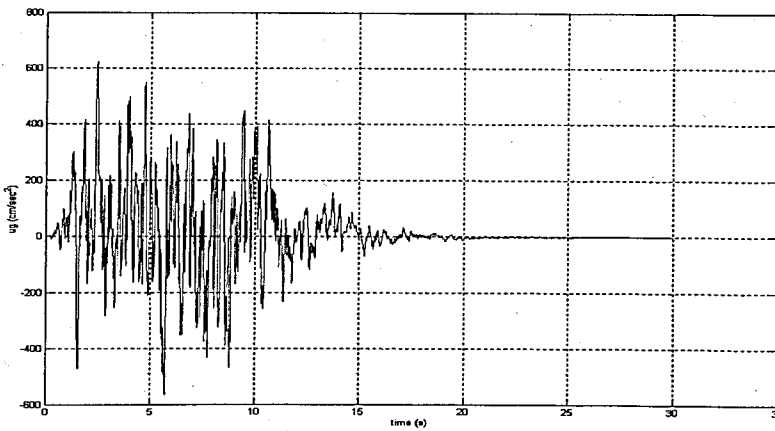


Figure 4.3. Input ground motions used for the sample structure

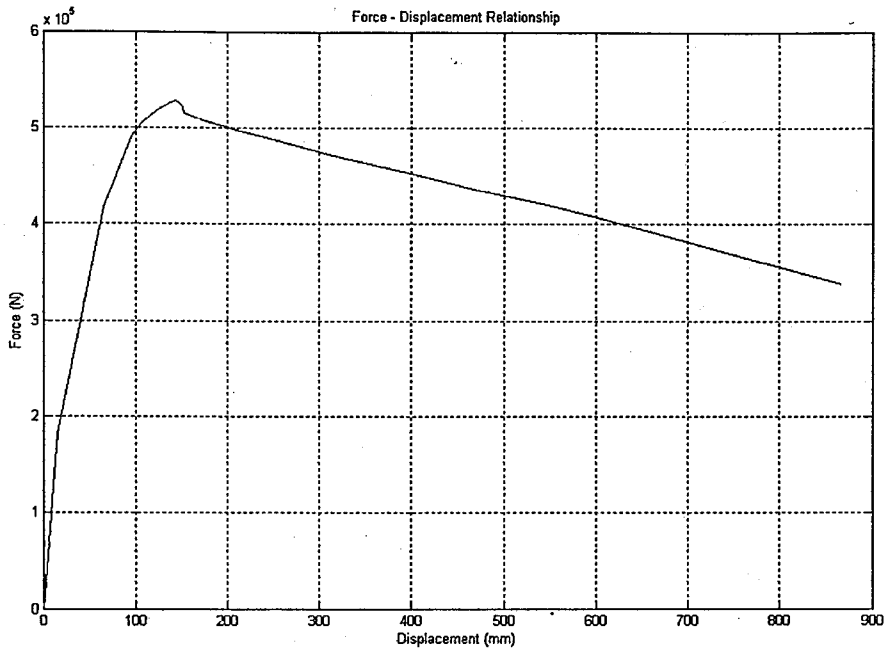


Figure 4.4. Force – Displacement relationship of the column obtained from the capacity analysis

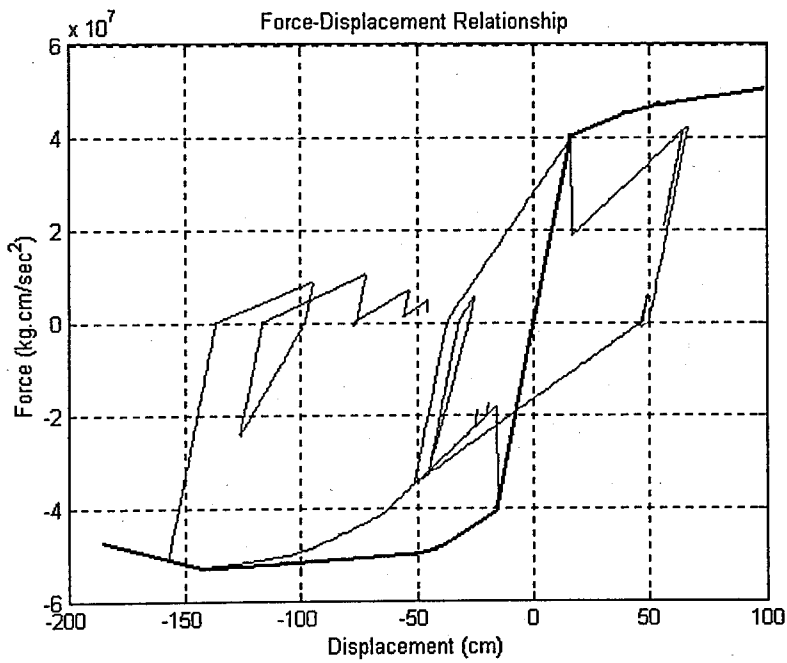


Figure 4.5. Force – Displacement relationship for the column obtained from the dynamic analysis

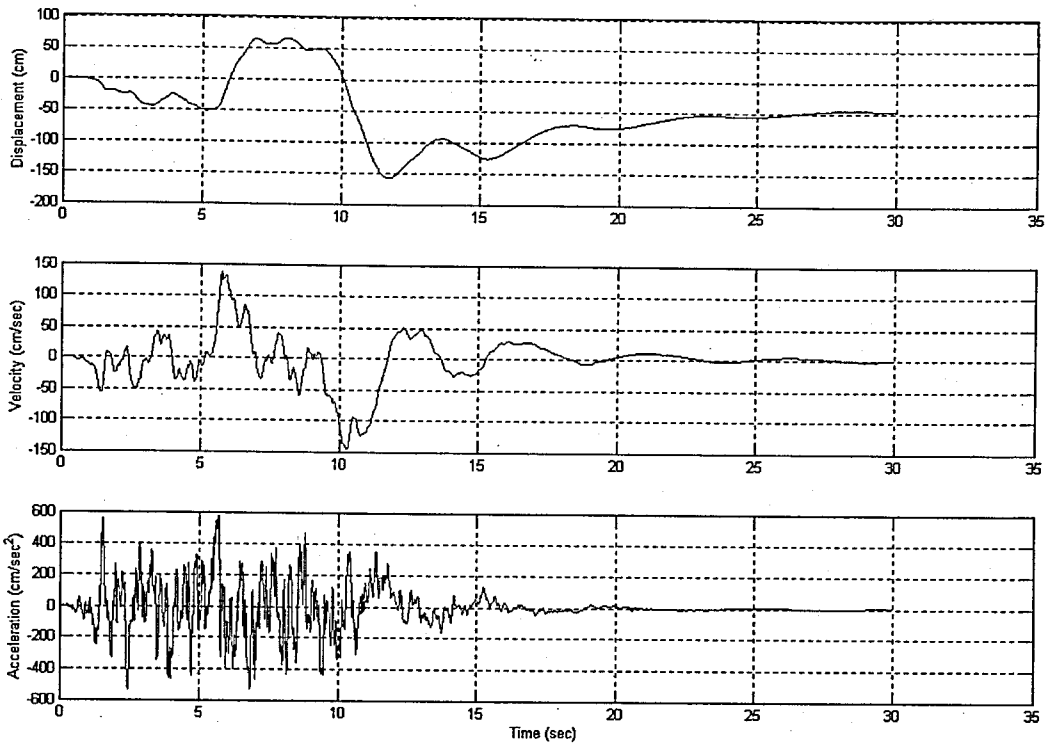


Figure 4.6. Displacement, velocity and acceleration responses obtained from the dynamic analysis

Based on these varying parameters, the analyses are performed for 96 different SDOF structures, and the respective Capacity/Demand ratios and the possibilities of failure are determined, and tabulated in the Tables 4.18–4.28. Also, for a clearer understanding of the effect of aspect ratios over the percentage of failure, aspect ratio versus probability of failure charts are demonstrated (Figures 4.12-4.55).

Table 4.3. Input parameters for $\rho = 0.01$ and $P/P_0 = 0.10$

f_c'	20000000 (N/m ²)							
ρ	0,01							
P/P_0	0,1							
L/D	L (m)	D (m)	P ₀ (N)	P (N)	m (kg)	Concrete Area (m ²)	Steel Area (m ²)	Reinforcement
3	9,00	3,00	141371669	14137167	1441098	7,068583	0,070686	133φ26
5	9,00	1,80	50893801	5089380	518795	2,544690	0,025447	56φ24
7	9,00	1,29	25966225	2596622	264691	1,298311	0,012983	41φ20
9	9,00	1,00	15707963	1570796	160122	0,785398	0,007854	21φ22
3	4,20	1,40	30787608	3078761	313839	1,539380	0,015394	49φ20
5	7,00	1,40	30787608	3078761	313839	1,539380	0,015394	49φ20
7	9,80	1,40	30787608	3078761	313839	1,539380	0,015394	49φ20
9	12,60	1,40	30787608	3078761	313839	1,539380	0,015394	49φ20

Table 4.4. Input parameters for $\rho = 0.02$ and $P/P_0 = 0.10$

f_c'	20000000 (N/m ²)							
ρ	0,02							
P/P ₀	0,1							
L/D	L (m)	D (m)	P ₀ (N)	P (N)	m (kg)	Concrete Area (m ²)	Steel Area (m ²)	Reinforcement
3	9,00	3,00	141371669	14137167	1441098	7,068583	0,141372	176φ32
5	9,00	1,80	50893801	5089380	518795	2,544690	0,050894	63φ32
7	9,00	1,29	25966225	2596622	264691	1,298311	0,025966	49φ26
9	9,00	1,00	15707963	1570796	160122	0,785398	0,015708	30φ26
3	4,20	1,40	30787608	3078761	313839	1,539380	0,030788	58φ26
5	7,00	1,40	30787608	3078761	313839	1,539380	0,030788	58φ26
7	9,80	1,40	30787608	3078761	313839	1,539380	0,030788	58φ26
9	12,60	1,40	30787608	3078761	313839	1,539380	0,030788	58φ26

Table 4.5. Input parameters for $\rho = 0.01$ and $P/P_0 = 0.15$

fc'	20000000 (N/m2)							
ρ	0,01							
P/P0	0,15							
L/D	L (m)	D (m)	P0 (N)	P (N)	m (kg)	Concrete Area (m2)	Steel Area (m2)	Reinforcement
3	9,00	3,00	141371669	21205750	2161646	7,068583	0,070686	13 ϕ 26
5	9,00	1,80	50893801	7634070	778193	2,544690	0,025447	5 ϕ 24
7	9,00	1,29	25966225	3894934	397037	1,298311	0,012983	41 ϕ 20
9	9,00	1,00	15707963	2356194	240183	0,785398	0,007854	21 ϕ 22
3	4,20	1,40	30787608	4618141	470759	1,539380	0,015394	49 ϕ 20
5	7,00	1,40	30787608	4618141	470759	1,539380	0,015394	49 ϕ 20
7	9,80	1,40	30787608	4618141	470759	1,539380	0,015394	49 ϕ 20
9	12,60	1,40	30787608	4618141	470759	1,539380	0,015394	49 ϕ 20

Table 4.6. Input parameters for $\rho = 0.02$ and $P/P_0 = 0.15$

f_c'	20000000 (N/m ²)							
ρ	0,02							
P/P ₀	0,15							
L/D	L (m)	D (m)	P ₀ (N)	P (N)	m (kg)	Concrete Area (m ²)	Steel Area (m ²)	Reinforcement
3	9,00	3,00	141371669	21205750	2161646	7,068583	0,141372	17 ϕ 32
5	9,00	1,80	50893801	7634070	778193	2,544690	0,050894	63 ϕ 32
7	9,00	1,29	25966225	3894934	397037	1,298311	0,025966	49 ϕ 26
9	9,00	1,00	15707963	2356194	240183	0,785398	0,015708	30 ϕ 26
3	4,20	1,40	30787608	4618141	470759	1,539380	0,030788	58 ϕ 26
5	7,00	1,40	30787608	4618141	470759	1,539380	0,030788	58 ϕ 26
7	9,80	1,40	30787608	4618141	470759	1,539380	0,030788	58 ϕ 26
9	12,60	1,40	30787608	4618141	470759	1,539380	0,030788	58 ϕ 26

Table 4.8. Output parameters for $\rho = 0.01$ and $f_c' = 20$ MPa

P/P ₀	ρ	L (m)	D (m)	L/D	Ground Motion Number	Capacity (mm)	Demand (mm)	C/D	Pass=0 Fail=1	Probability Of Failure
0.10	0.01	9.00	3.00	3.00	1	120.10	59.75	2.01	0	0.00
					2	120.10	52.44	2.29	0	
					3	120.10	41.99	2.86	0	
					4	120.10	67.97	1.77	0	
					5	120.10	41.38	2.90	0	
					6	120.10	62.14	1.93	0	
					7	120.10	84.59	1.42	0	
					8	120.10	64.42	1.86	0	
					9	120.10	69.16	1.74	0	
					10	120.10	87.00	1.38	0	
			1.80	5.00	1	196.44	115.83	1.70	0	0.10
					2	196.44	196.49	1.00	1	
					3	196.44	175.43	1.12	0	
					4	196.44	93.44	2.10	0	
					5	196.44	183.05	1.07	0	
					6	196.44	121.27	1.62	0	
					7	196.44	103.77	1.89	0	
					8	196.44	124.90	1.57	0	
					9	196.44	161.86	1.21	0	
					10	196.44	93.32	2.11	0	
			1.29	7.00	1	276.34	151.38	1.83	0	0.00
					2	276.34	201.97	1.37	0	
					3	276.34	151.92	1.82	0	
					4	276.34	176.88	1.56	0	
					5	276.34	256.44	1.08	0	
					6	276.34	209.32	1.32	0	
					7	276.34	251.62	1.10	0	
					8	276.34	224.80	1.23	0	
					9	276.34	221.56	1.25	0	
					10	276.34	197.14	1.40	0	
	1.00	9.00	1	293.19	225.24	1.30	0	0.80		
			2	293.19	296.47	0.99	1			
			3	293.19	294.72	0.99	1			
			4	293.19	296.34	0.99	1			
			5	293.19	296.34	0.99	1			
			6	293.19	296.34	0.99	1			
			7	293.19	296.34	0.99	1			
			8	293.19	296.34	0.99	1			
			9	293.19	296.34	0.99	1			
			10	293.19	274.67	1.07	0			
	0.01	4.20	3.00	1	56.34	49.94	1.13	0	0.60	
				2	56.34	56.61	1.00	1		
				3	56.34	56.61	1.00	1		
				4	56.34	35.54	1.59	0		
				5	56.34	56.60	1.00	1		
				6	56.34	56.90	0.99	1		
				7	56.34	24.95	2.26	0		
				8	56.34	27.28	2.07	0		
				9	56.34	57.32	0.98	1		
				10	56.34	56.84	0.99	1		
		7.00	5.00	1	156.49	134.08	1.17	0	0.00	
				2	156.49	83.99	1.86	0		
				3	156.49	111.65	1.40	0		
				4	156.49	90.90	1.72	0		
				5	156.49	104.62	1.50	0		
				6	156.49	120.45	1.30	0		
				7	156.49	106.68	1.47	0		
				8	156.49	106.34	1.47	0		
				9	156.49	95.02	1.65	0		
				10	156.49	92.91	1.68	0		
9.80		7.00	1	306.73	204.78	1.50	0	0.00		
			2	306.73	198.26	1.55	0			
			3	306.73	174.45	1.76	0			
			4	306.73	221.36	1.39	0			
			5	306.73	200.98	1.53	0			
			6	306.73	182.64	1.68	0			
			7	306.73	180.49	1.70	0			
			8	306.73	242.40	1.27	0			
			9	306.73	217.45	1.41	0			
			10	306.73	180.89	1.70	0			
12.60	9.00	1	507.04	249.64	2.03	0	0.10			
		2	507.04	396.01	1.28	0				
		3	507.04	282.46	1.80	0				
		4	507.04	511.58	0.99	1				
		5	507.04	362.49	1.40	0				
		6	507.04	321.84	1.58	0				
		7	507.04	384.51	1.32	0				
		8	507.04	230.89	2.20	0				
		9	507.04	454.59	1.12	0				
		10	507.04	329.49	1.54	0				

Table 4.9. Output parameters for $\rho = 0.02$ and $f_c' = 20$ MPa

P/P ₀	ρ	L (m)	D (m)	L/D	Ground Motion Number	Capacity (mm)	Demand (mm)	C/D	Pass=0 Fail=1	Probability Of Failure
0.10	0.02	9.00	3.00	3.00	1	102.30	47.50	2.15	0	0.00
					2	102.30	52.78	1.94	0	
					3	102.30	51.57	1.98	0	
					4	102.30	86.74	1.18	0	
					5	102.30	53.89	1.90	0	
					6	102.30	45.20	2.26	0	
					7	102.30	52.15	1.96	0	
					8	102.30	48.29	2.12	0	
					9	102.30	51.25	2.00	0	
					10	102.30	58.30	1.75	0	
			1.80	5.00	1	164.97	122.91	1.34	0	0.00
					2	164.97	104.43	1.58	0	
					3	164.97	127.17	1.30	0	
					4	164.97	85.52	1.93	0	
					5	164.97	135.47	1.22	0	
					6	164.97	100.55	1.64	0	
					7	164.97	113.96	1.45	0	
					8	164.97	103.60	1.59	0	
					9	164.97	107.80	1.53	0	
					10	164.97	122.74	1.34	0	
			1.29	7.00	1	225.07	134.61	1.67	0	0.00
					2	225.07	198.15	1.14	0	
					3	225.07	140.81	1.60	0	
					4	225.07	159.23	1.41	0	
					5	225.07	191.63	1.17	0	
					6	225.07	186.27	1.21	0	
					7	225.07	156.02	1.44	0	
					8	225.07	159.72	1.41	0	
					9	225.07	152.03	1.48	0	
					10	225.07	129.91	1.73	0	
	1.00	9.00	1	281.61	284.29	0.99	1	0.60		
			2	281.61	286.94	0.98	1			
			3	281.61	251.02	1.12	0			
			4	281.61	263.23	1.07	0			
			5	281.61	285.48	0.99	1			
			6	281.61	275.16	1.02	0			
			7	281.61	286.99	0.98	1			
			8	281.61	284.39	0.99	1			
			9	281.61	284.39	0.99	1			
			10	281.61	263.87	1.07	0			
0.02	4.20	1.40	3.00	1	47.08	25.62	1.84	0	0.10	
				2	47.08	32.54	1.45	0		
				3	47.08	37.45	1.26	0		
				4	47.08	47.21	1.00	1		
				5	47.08	27.17	1.73	0		
				6	47.08	36.10	1.30	0		
				7	47.08	39.57	1.19	0		
				8	47.08	27.65	1.70	0		
				9	47.08	33.85	1.39	0		
				10	47.08	24.58	1.92	0		
	7.00		5.00	1	130.78	92.62	1.41	0	0.00	
				2	130.78	95.91	1.36	0		
				3	130.78	128.90	1.01	0		
				4	130.78	96.76	1.35	0		
				5	130.78	116.62	1.12	0		
				6	130.78	100.32	1.30	0		
				7	130.78	111.59	1.17	0		
				8	130.78	112.57	1.16	0		
				9	130.78	101.99	1.28	0		
				10	130.78	96.20	1.36	0		
	9.80		7.00	1	256.34	164.81	1.56	0	0.00	
				2	256.34	209.69	1.22	0		
				3	256.34	181.32	1.41	0		
				4	256.34	227.28	1.13	0		
				5	256.34	206.00	1.24	0		
				6	256.34	187.52	1.37	0		
				7	256.34	173.48	1.48	0		
				8	256.34	167.82	1.53	0		
				9	256.34	183.87	1.39	0		
				10	256.34	144.86	1.77	0		
12.60	9.00		1	423.74	287.63	1.47	0	0.00		
			2	423.74	368.14	1.15	0			
			3	423.74	274.35	1.54	0			
			4	423.74	247.70	1.71	0			
			5	423.74	330.66	1.28	0			
			6	423.74	310.55	1.36	0			
			7	423.74	394.17	1.08	0			
			8	423.74	307.99	1.38	0			
			9	423.74	334.67	1.27	0			
			10	423.74	294.63	1.44	0			

Table 4.10. C/D ratio results for $\rho = 0.01$ and $f_c' = 20$ MPa

L/D	L (m)	D (m)	Capacity (mm)	Demand (mm)	C/D
3	9.00	3.00	102	55	1.87
5	9.00	1.80	165	112	1.47
7	9.00	1.29	225	161	1.40
9	9.00	1.00	282	277	1.02
3	4.20	1.40	47	33	1.42
5	7.00	1.40	131	105	1.24
7	9.80	1.40	256	185	1.39
9	12.60	1.40	424	315	1.34

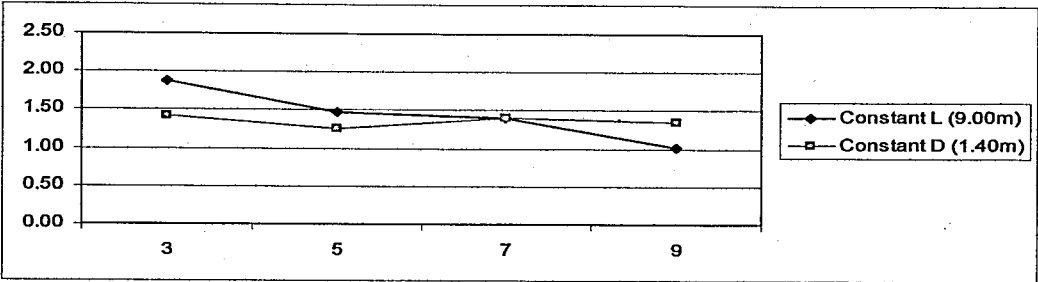


Figure 4.7. L/D versus C/D for $\rho = 0.01$ and $f_c' = 20$ MPa

Table 4.11. C/D ratio results for $\rho = 0.02$ and $f_c' = 20$ MPa

L/D	L (m)	D (m)	Capacity (mm)	Demand (mm)	C/D
3	9.00	3.00	120	63	1.90
5	9.00	1.80	196	137	1.43
7	9.00	1.29	276	204	1.35
9	9.00	1.00	293	287	1.02
3	4.20	1.40	56	48	1.18
5	7.00	1.40	156	105	1.50
7	9.80	1.40	307	200	1.53
9	12.60	1.40	507	352	1.44

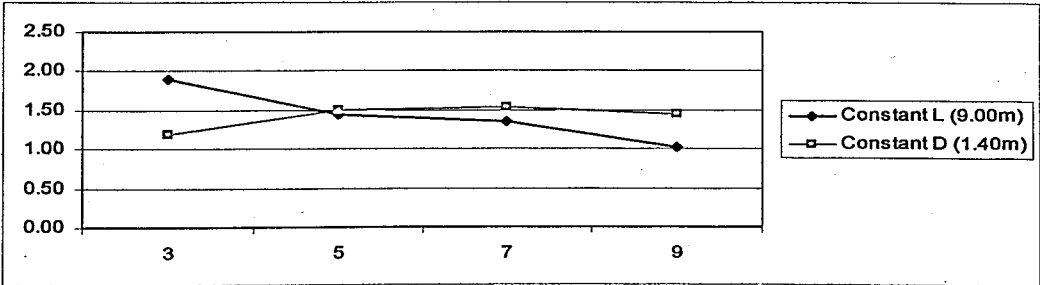


Figure 4.8. L/D versus C/D for $\rho = 0.02$ and $f_c' = 20$ MPa

Table 4.12. Output for $\rho = 0.01$ and $f_c' = 25$ MPa

P/P_0	ρ	L (m)	D (m)	L/D	Ground Motion Number	Capacity (mm)	Demand (mm)	C/D	Pass=0 Fail=1	Probability Of Failure
0.10	0.01	9.00	3.00	3.00	1	76.34	76.42	1.00	1	0.80
					2	76.34	76.48	1.00	1	
					3	76.34	50.29	1.52	0	
					4	76.34	78.03	0.98	1	
					5	76.34	78.03	0.98	1	
					6	76.34	48.06	1.59	0	
					7	76.34	76.97	0.99	1	
					8	76.34	77.16	0.99	1	
					9	76.34	77.60	0.98	1	
					10	76.34	77.60	0.98	1	
			1.80	5.00	1	131.77	107.35	1.23	0	0.50
					2	131.77	133.42	0.99	1	
					3	131.77	132.50	0.99	1	
					4	131.77	130.74	1.01	0	
					5	131.77	104.64	1.26	0	
					6	131.77	134.54	0.98	1	
					7	131.77	120.04	1.10	0	
					8	131.77	132.15	1.00	1	
					9	131.77	132.31	1.00	1	
					10	131.77	121.55	1.08	0	
			1.29	7.00	1	166.88	167.87	0.99	1	1.00
					2	166.88	167.87	0.99	1	
					3	166.88	169.65	0.98	1	
					4	166.88	169.65	0.98	1	
					5	166.88	168.13	0.99	1	
					6	166.88	168.23	0.99	1	
					7	166.88	171.75	0.97	1	
					8	166.88	171.75	0.97	1	
					9	166.88	168.50	0.99	1	
					10	166.88	168.50	0.99	1	
			1.00	9.00	1	216.52	218.46	0.99	1	1.00
					2	216.52	218.01	0.99	1	
					3	216.52	218.01	0.99	1	
					4	216.52	216.96	1.00	1	
					5	216.52	217.01	1.00	1	
					6	216.52	217.69	0.99	1	
					7	216.52	217.42	1.00	1	
					8	216.52	218.66	0.99	1	
					9	216.52	218.66	0.99	1	
					10	216.52	217.73	0.99	1	
	0.01	9.00	4.20	3.00	1	34.89	35.06	1.00	1	1.00
					2	34.89	35.06	1.00	1	
					3	34.89	35.45	0.98	1	
					4	34.89	34.90	1.00	1	
					5	34.89	35.66	0.98	1	
					6	34.89	35.13	0.99	1	
					7	34.89	35.05	1.00	1	
					8	34.89	35.73	0.98	1	
					9	34.89	35.34	0.99	1	
					10	34.89	35.23	0.99	1	
			7.00	5.00	1	96.93	98.23	0.99	1	0.90
					2	96.93	98.68	0.98	1	
					3	96.93	98.23	0.99	1	
					4	96.93	85.42	1.13	0	
					5	96.93	97.56	0.99	1	
					6	96.93	97.74	0.99	1	
					7	96.93	99.46	0.97	1	
					8	96.93	99.46	0.97	1	
					9	96.93	97.56	0.99	1	
					10	96.93	97.59	0.99	1	
			9.80	7.00	1	189.98	192.07	0.99	1	1.00
					2	189.98	190.13	1.00	1	
					3	189.98	190.14	1.00	1	
					4	189.98	192.54	0.99	1	
					5	189.98	191.84	0.99	1	
					6	189.98	191.05	0.99	1	
					7	189.98	191.95	0.99	1	
					8	189.98	191.95	0.99	1	
					9	189.98	191.87	0.99	1	
					10	189.98	191.91	0.99	1	
			12.60	9.00	1	314.05	311.45	1.01	0	0.80
					2	314.05	318.07	0.99	1	
					3	314.05	318.07	0.99	1	
					4	314.05	319.10	0.98	1	
					5	314.05	319.10	0.98	1	
					6	314.05	319.10	0.98	1	
					7	314.05	319.10	0.98	1	
					8	314.05	295.34	1.06	0	
					9	314.05	315.99	0.99	1	
					10	314.05	317.54	0.99	1	

Table 4.13. Outputs for $\rho = 0.02$ and $f_c' = 25$ MPa

P/P_0	ρ	L (m)	D (m)	L/D	Ground Motion Number	Capacity (mm)	Demand (mm)	C/D	Pass=0 Fail=1	Probability Of Failure
0.10	0.02	9.00	3.00	3.00	1	81.32	62.96	1.29	0	0.10
					2	81.32	63.90	1.27	0	
					3	81.32	78.13	1.04	0	
					4	81.32	75.28	1.08	0	
					5	81.32	66.07	1.23	0	
					6	81.32	59.75	1.36	0	
					7	81.32	71.02	1.15	0	
					8	81.32	62.78	1.30	0	
					9	81.32	81.34	1.00	1	
					10	81.32	58.71	1.39	0	
			1.80	5.00	1	123.79	110.22	1.12	0	0.80
					2	123.79	124.01	1.00	1	
					3	123.79	126.62	0.98	1	
					4	123.79	123.30	1.00	0	
					5	123.79	124.63	0.99	1	
					6	123.79	125.18	0.99	1	
					7	123.79	124.55	0.99	1	
					8	123.79	124.55	0.99	1	
					9	123.79	124.97	0.99	1	
					10	123.79	124.59	0.99	1	
			1.29	7.00	1	171.68	171.81	1.00	1	1.00
					2	171.68	174.14	0.99	1	
					3	171.68	175.24	0.98	1	
					4	171.68	175.24	0.98	1	
					5	171.68	171.88	1.00	1	
					6	171.68	173.39	0.99	1	
					7	171.68	173.39	0.99	1	
					8	171.68	174.41	0.98	1	
					9	171.68	173.39	0.99	1	
					10	171.68	173.71	0.99	1	
			1.00	9.00	1	223.17	225.42	0.99	1	1.00
					2	223.17	225.42	0.99	1	
					3	223.17	225.42	0.99	1	
					4	223.17	223.45	1.00	1	
					5	223.17	223.82	1.00	1	
					6	223.17	223.27	1.00	1	
					7	223.17	224.22	1.00	1	
					8	223.17	224.22	1.00	1	
					9	223.17	224.56	0.99	1	
					10	223.17	224.56	0.99	1	
	0.02	4.20	3.00	3.00	1	36.15	36.68	0.99	1	1.00
					2	36.15	36.85	0.98	1	
					3	36.15	36.68	0.99	1	
					4	36.15	38.68	0.93	1	
					5	36.15	36.46	0.99	1	
					6	36.15	36.46	0.99	1	
					7	36.15	36.67	0.99	1	
					8	36.15	36.79	0.98	1	
					9	36.15	36.79	0.98	1	
					10	36.15	36.79	0.98	1	
		7.00	5.00	5.00	1	100.41	101.35	0.99	1	0.90
					2	100.41	101.35	0.99	1	
					3	100.41	100.43	1.00	1	
					4	100.41	92.40	1.09	0	
					5	100.41	103.71	0.97	1	
					6	100.41	103.71	0.97	1	
					7	100.41	103.71	0.97	1	
					8	100.41	103.71	0.97	1	
					9	100.41	103.71	0.97	1	
					10	100.41	103.71	0.97	1	
		9.80	7.00	7.00	1	196.81	197.32	1.00	1	1.00
					2	196.81	198.56	0.99	1	
					3	196.81	198.87	0.99	1	
					4	196.81	198.87	0.99	1	
					5	196.81	199.39	0.99	1	
					6	196.81	199.09	0.99	1	
					7	196.81	199.09	0.99	1	
					8	196.81	199.09	0.99	1	
					9	196.81	199.09	0.99	1	
					10	196.81	199.28	0.99	1	
		12.60	9.00	9.00	1	325.34	329.93	0.99	1	0.90
					2	325.34	329.93	0.99	1	
					3	325.34	329.93	0.99	1	
					4	325.34	330.32	0.98	1	
					5	325.34	330.32	0.98	1	
					6	325.34	328.17	0.99	1	
					7	325.34	330.06	0.99	1	
					8	325.34	330.06	0.99	1	
					9	325.34	328.17	0.99	1	
					10	325.34	302.78	1.07	0	

Table 4.14. Outputs for $\rho = 0.03$ and $f_c' = 25$ MPa

P/P_0	ρ	L (m)	D (m)	L/D	Ground Motion Number	Capacity (mm)	Demand (mm)	C/D	Pass=0 Fail=1	Probability Of Failure
0.10	0.03	9.00	3.00	3.00	1	89.41	46.12	1.94	0	0.10
					2	89.41	55.48	1.61	0	
					3	89.41	58.30	1.53	0	
					4	89.41	90.10	0.99	1	
					5	89.41	51.78	1.73	0	
					6	89.41	57.09	1.57	0	
					7	89.41	53.12	1.68	0	
					8	89.41	46.43	1.93	0	
					9	89.41	71.99	1.24	0	
					10	89.41	49.61	1.80	0	
			1.80	5.00	1	143.30	124.61	1.15	0	0.00
					2	143.30	113.35	1.26	0	
					3	143.30	125.05	1.15	0	
					4	143.30	77.26	1.85	0	
					5	143.30	133.85	1.07	0	
					6	143.30	124.21	1.15	0	
					7	143.30	117.01	1.22	0	
					8	143.30	99.61	1.44	0	
					9	143.30	109.71	1.31	0	
					10	143.30	123.53	1.16	0	
			1.29	7.00	1	195.97	169.08	1.16	0	0.30
					2	195.97	196.14	1.00	1	
					3	195.97	171.99	1.14	0	
					4	195.97	196.80	1.00	1	
					5	195.97	188.43	1.04	0	
					6	195.97	183.42	1.07	0	
					7	195.97	156.37	1.25	0	
					8	195.97	167.36	1.17	0	
					9	195.97	197.60	0.99	1	
					10	195.97	134.88	1.45	0	
			1.00	9.00	1	242.48	243.01	1.00	1	1.00
					2	242.48	244.29	0.99	1	
					3	242.48	243.98	0.99	1	
					4	242.48	243.29	1.00	1	
					5	242.48	245.64	0.99	1	
					6	242.48	243.59	1.00	1	
					7	242.48	243.59	1.00	1	
					8	242.48	243.80	0.99	1	
					9	242.48	243.80	0.99	1	
					10	242.48	243.80	0.99	1	
	0.03	9.00	4.20	3.00	1	38.64	39.44	0.98	1	0.40
					2	38.64	37.49	1.03	0	
					3	38.64	29.36	1.32	0	
					4	38.64	39.72	0.97	1	
					5	38.64	33.21	1.16	0	
					6	38.64	38.65	1.00	1	
					7	38.64	36.40	1.06	0	
					8	38.64	28.64	1.35	0	
					9	38.64	38.92	0.99	1	
					10	38.64	24.70	1.56	0	
			7.00	5.00	1	107.34	95.58	1.12	0	0.30
					2	107.34	93.34	1.15	0	
					3	107.34	107.25	1.00	0	
					4	107.34	104.27	1.03	0	
					5	107.34	107.43	1.00	1	
					6	107.34	100.79	1.06	0	
					7	107.34	108.66	0.99	1	
					8	107.34	106.14	1.01	0	
					9	107.34	108.68	0.99	1	
					10	107.34	97.51	1.10	0	
			9.80	7.00	1	210.39	178.12	1.18	0	0.20
					2	210.39	210.90	1.00	1	
					3	210.39	175.55	1.20	0	
					4	210.39	204.91	1.03	0	
					5	210.39	203.13	1.04	0	
					6	210.39	178.99	1.18	0	
					7	210.39	160.82	1.31	0	
					8	210.39	167.28	1.26	0	
					9	210.39	210.79	1.00	1	
					10	210.39	137.96	1.53	0	
			12.60	9.00	1	347.78	305.55	1.14	0	0.20
					2	347.78	349.35	1.00	1	
					3	347.78	294.68	1.18	0	
					4	347.78	250.82	1.39	0	
					5	347.78	340.37	1.02	0	
					6	347.78	309.23	1.12	0	
					7	347.78	351.34	0.99	1	
					8	347.78	280.88	1.24	0	
					9	347.78	345.77	1.01	0	
					10	347.78	318.50	1.09	0	

Table 4.15. C/D ratio results for $\rho = 0.01$ and $f_c' = 25$ MPa

L/D	L (m)	D (m)	Capacity (mm)	Demand (mm)	C/D
3	9.00	3.00	76	72	1.07
5	9.00	1.80	132	125	1.05
7	9.00	1.29	167	169	0.99
9	9.00	1.00	217	218	0.99
3	4.20	1.40	35	35	0.99
5	7.00	1.40	97	97	1.00
7	9.80	1.40	190	192	0.99
9	12.60	1.40	314	315	1.00

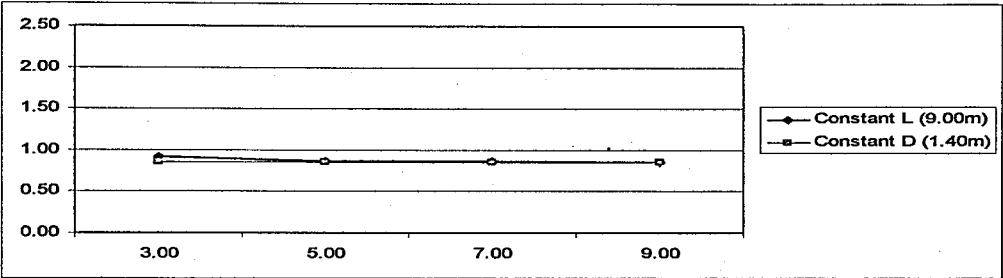


Figure 4.9. L/D versus C/D for $\rho = 0.01$ and $f_c' = 25$ Mpa

Table 4.16. C/D ratio results for $\rho = 0.02$ and $f_c' = 25$ MPa

L/D	L (m)	D (m)	Capacity (mm)	Demand (mm)	C/D
3	9.00	3.00	81	68	1.20
5	9.00	1.80	124	123	1.00
7	9.00	1.29	172	174	0.99
9	9.00	1.00	223	224	0.99
3	4.20	1.40	36	37	0.98
5	7.00	1.40	100	102	0.99
7	9.80	1.40	197	199	0.99
9	12.60	1.40	325	327	1.00

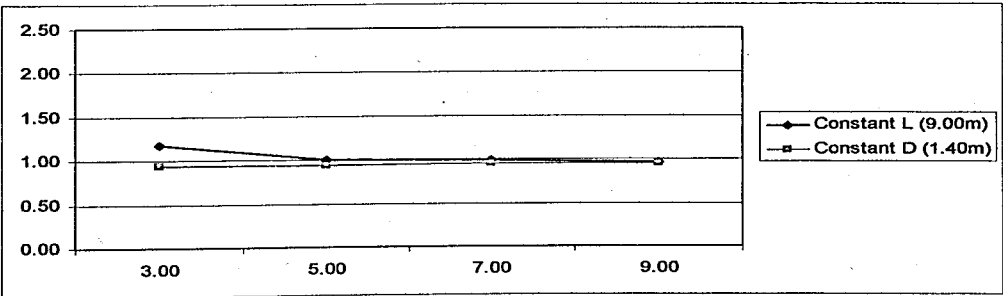


Figure 4.10. L/D versus C/D for $\rho = 0.02$ and $f_c' = 25$ Mpa

Table 4.17. C/D ratio results for $\rho = 0.03$ and $f_c' = 25 \text{ MPa}$

L/D	L (m)	D (m)	Capacity (mm)	Demand (mm)	C/D
3	9.00	3.00	89	58	1.54
5	9.00	1.80	143	115	1.25
7	9.00	1.29	196	176	1.11
9	9.00	1.00	242	244	0.99
3	4.20	1.40	39	35	1.12
5	7.00	1.40	107	103	1.04
7	9.80	1.40	210	183	1.15
9	12.60	1.40	348	315	1.11

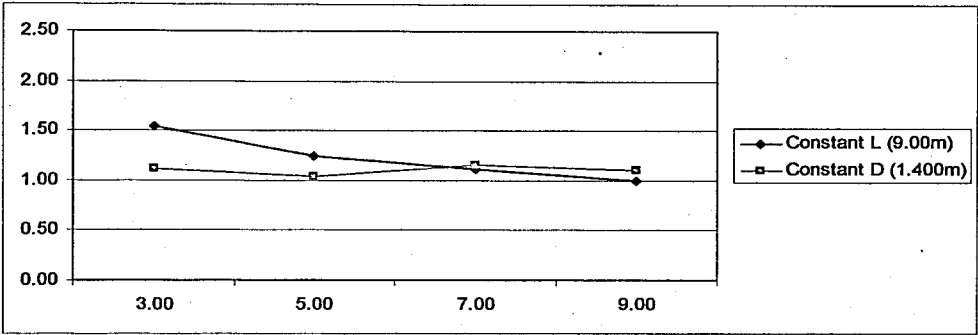


Figure 4.11. L/D versus C/D for $\rho = 0.02$ and $f_c' = 25 \text{ Mpa}$

Table 4.18. Probability of Failure for $PGA = 0.20g$ and $f'_c = 20 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.40
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	0.50
		5	0.60
		7	0.50
		9	0.50
	0.02	3	0.00
		5	0.10
		7	0.00
		9	0.20
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00

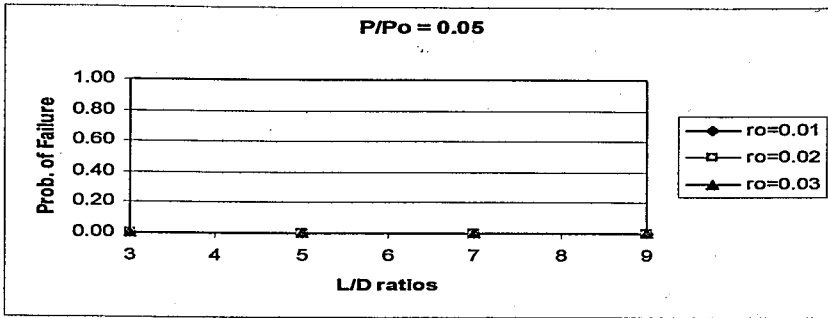


Figure 4.12. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.05$

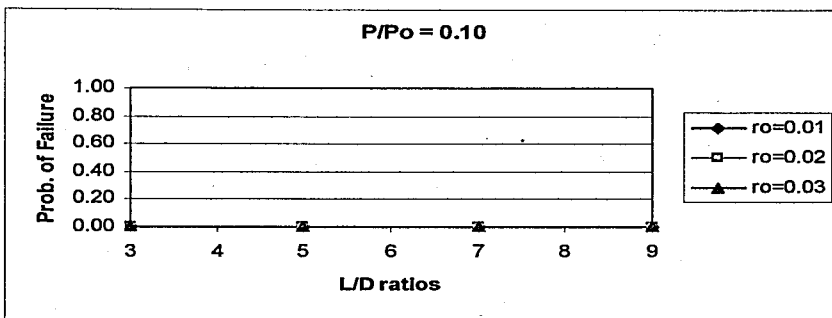


Figure 4.13. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.10$

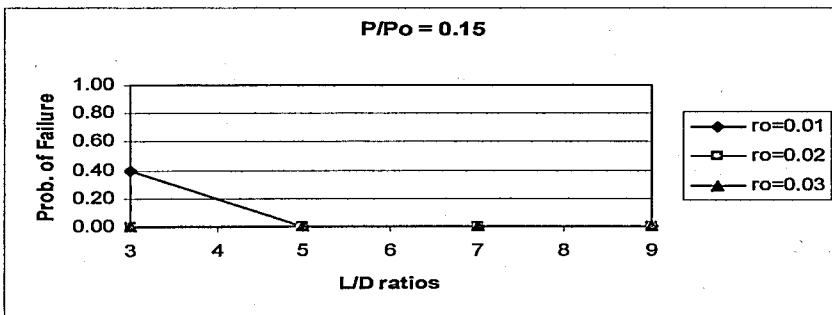


Figure 4.14. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.15$

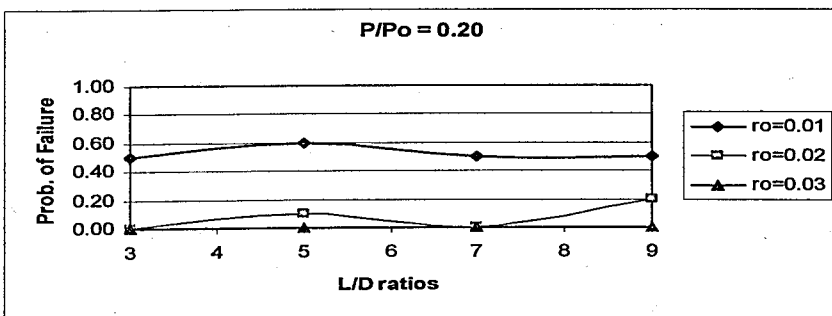


Figure 4.15. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.20$

Table 4.19. Probability of Failure for $PGA = 0.25g$ and $f'_c = 20 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.30
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.80
		5	0.10
		7	0.10
		9	0.20
	0.02	3	0.20
		5	0.20
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	1.00
		5	0.90
		7	0.90
		9	0.90
	0.02	3	0.90
		5	0.90
		7	0.80
		9	0.80
	0.03	3	0.20
		5	0.20
		7	0.10
		9	0.00

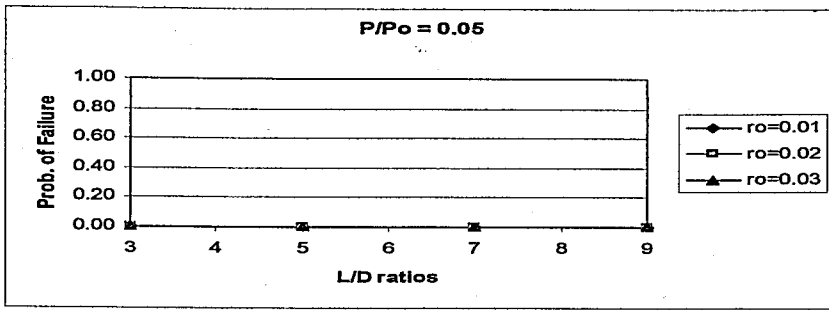


Figure 4.16. Probability of Failure for PGA=0.25g and $P/P_0=0.05$

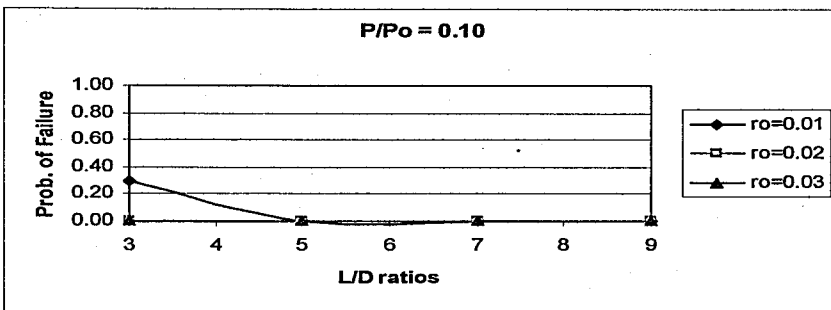


Figure 4.17. Probability of Failure for PGA=0.25g and $P/P_0=0.10$

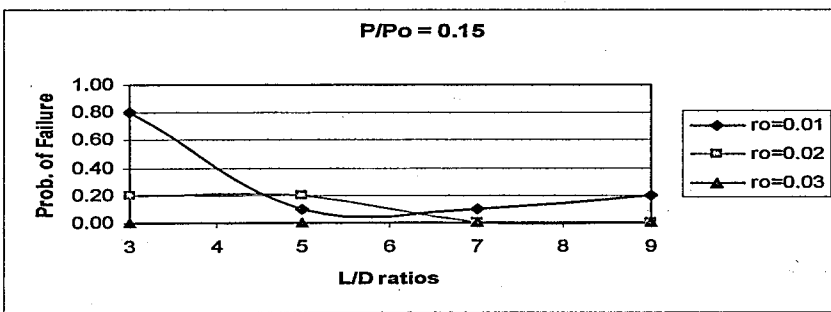


Figure 4.18. Probability of Failure for PGA=0.25g and $P/P_0=0.15$

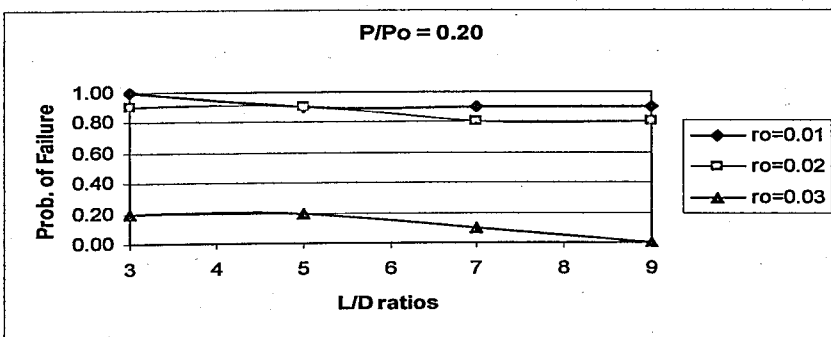


Figure 4.19. Probability of Failure for PGA=0.25g and $P/P_0=0.20$

Table 4.20. Probability of Failure for $PGA = 0.30g$ and $f'_c = 20 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0,05	0,01	3	0,00
		5	0,00
		7	0,00
		9	0,00
	0,02	3	0,00
		5	0,00
		7	0,00
		9	0,00
	0,03	3	0,00
		5	0,00
		7	0,00
		9	0,00
0,1	0,01	3	0,00
		5	0,10
		7	0,00
		9	0,00
	0,02	3	0,00
		5	0,00
		7	0,00
		9	0,00
	0,03	3	0,00
		5	0,00
		7	0,00
		9	0,00
0,15	0,01	3	0,70
		5	0,60
		7	0,60
		9	0,20
	0,02	3	0,10
		5	0,10
		7	0,10
		9	0,00
	0,03	3	0,00
		5	0,00
		7	0,00
		9	0,00
0,2	0,01	3	1,00
		5	1,00
		7	1,00
		9	1,00
	0,02	3	0,90
		5	0,80
		7	1,00
		9	0,80
	0,03	3	0,10
		5	0,20
		7	0,20
		9	0,10

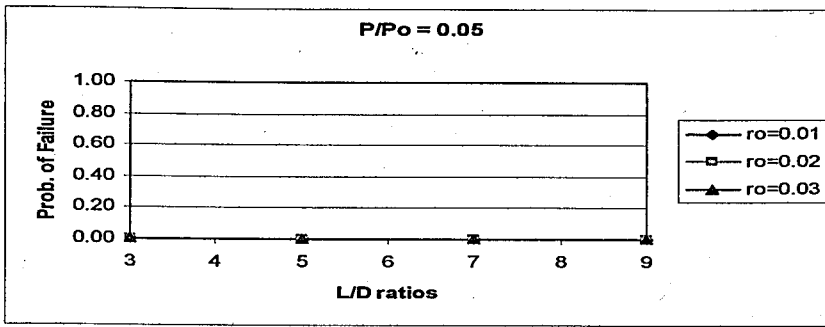


Figure 4.20. Probability of Failure for $PGA=0.30g$ and $P/P_0=0.05$

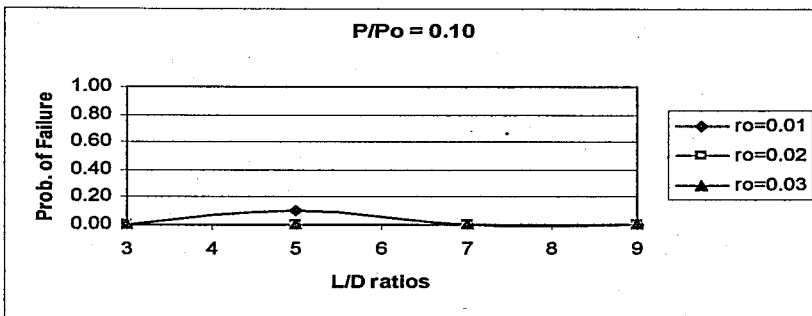


Figure 4.21. Probability of Failure for $PGA=0.30g$ and $P/P_0=0.10$

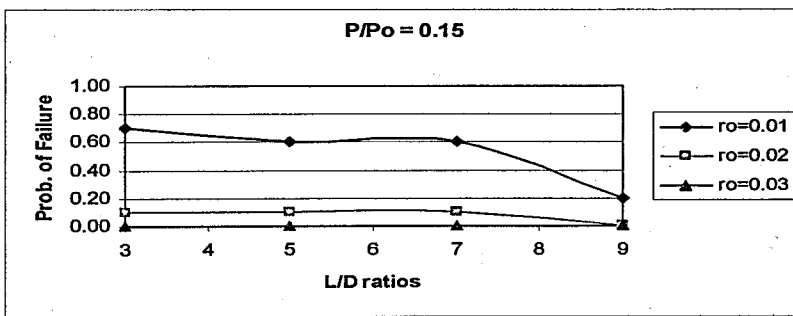


Figure 4.22. Probability of Failure for $PGA=0.30g$ and $P/P_0=0.15$

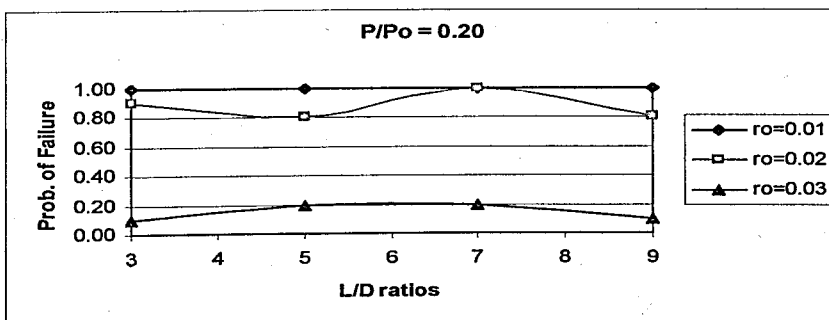


Figure 4.23. Probability of Failure for $PGA=0.30g$ and $P/P_0=0.20$

Table 4.21. Probability of Failure for $PGA = 0.35g$ and $f'_c = 20\text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.70
		5	0.70
		7	0.60
		9	0.20
	0.02	3	0.10
		5	0.30
		7	0.00
		9	0.20
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	1.00
		5	1.00
		7	0.90
		9	0.80
	0.02	3	1.00
		5	0.90
		7	1.00
		9	0.80
	0.03	3	0.20
		5	0.40
		7	0.10
		9	0.20

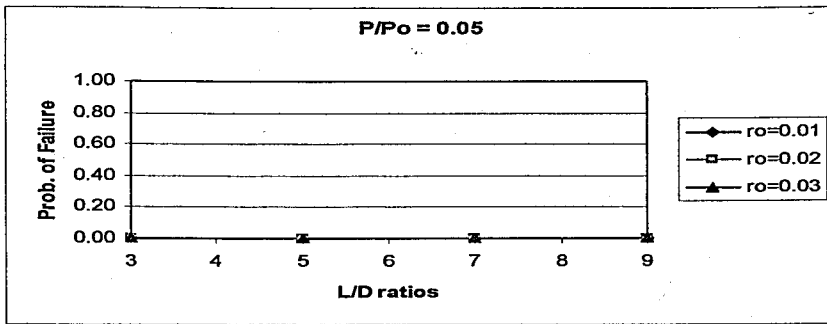


Figure 4.24. Probability of Failure for PGA=0.35g and $P/P_0=0.05$

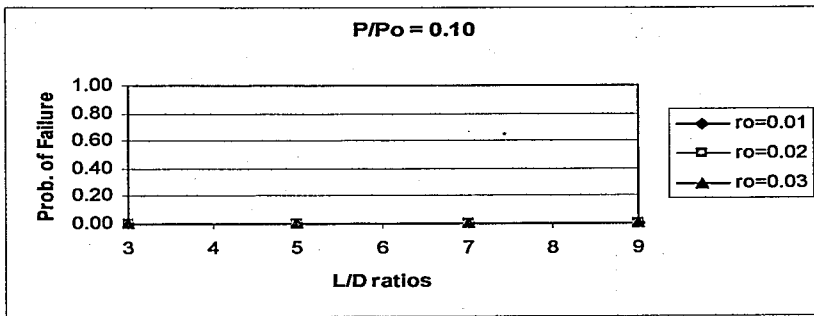


Figure 4.25. Probability of Failure for PGA=0.35g and $P/P_0=0.10$

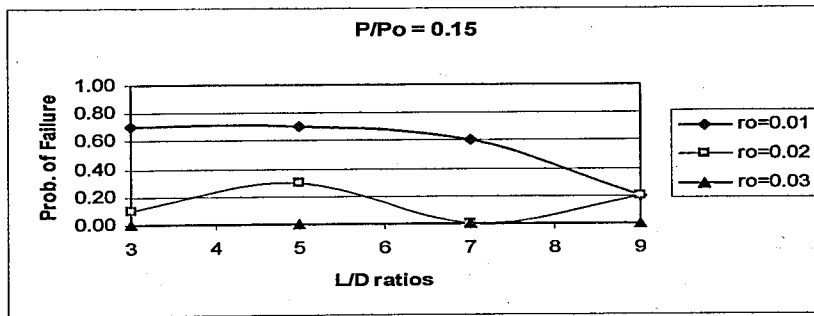


Figure 4.26. Probability of Failure for PGA=0.35g and $P/P_0=0.15$

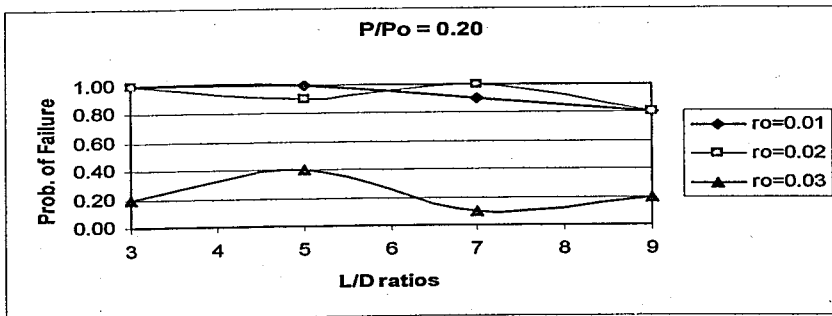


Figure 4.27. Probability of Failure for PGA=0.35g and $P/P_0=0.20$

Table 4.22. Probability of Failure for $PGA = 0.40g$ and $f_c' = 20 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.90
		5	0.40
		7	0.20
		9	0.40
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	1.00
		5	1.00
		7	1.00
		9	0.90
	0.02	3	0.80
		5	0.90
		7	1.00
		9	0.90
	0.03	3	0.30
		5	0.30
		7	0.20
		9	0.10
0.2	0.01	3	1.00
		5	1.00
		7	1.00
		9	1.00
	0.02	3	1.00
		5	1.00
		7	1.00
		9	1.00
	0.03	3	1.00
		5	0.90
		7	1.00
		9	1.00

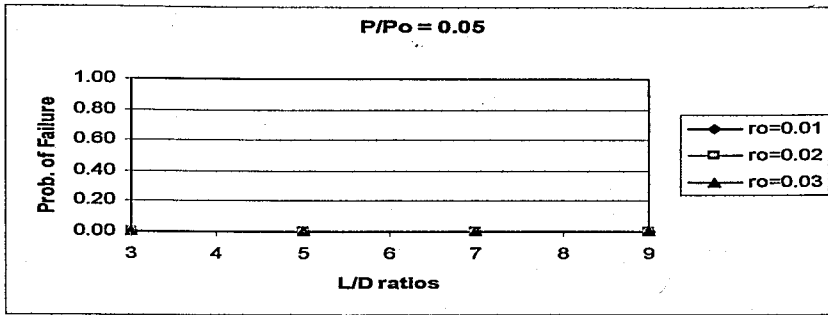


Figure 4.28. Probability of Failure for $PGA=0.40g$ and $P/P_0=0.05$

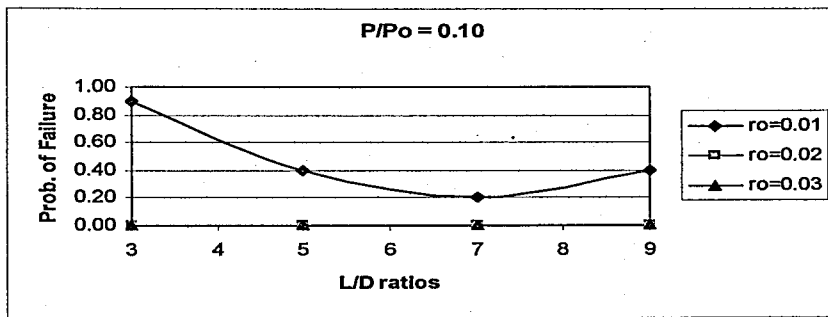


Figure 4.29. Probability of Failure for $PGA=0.40g$ and $P/P_0=0.10$

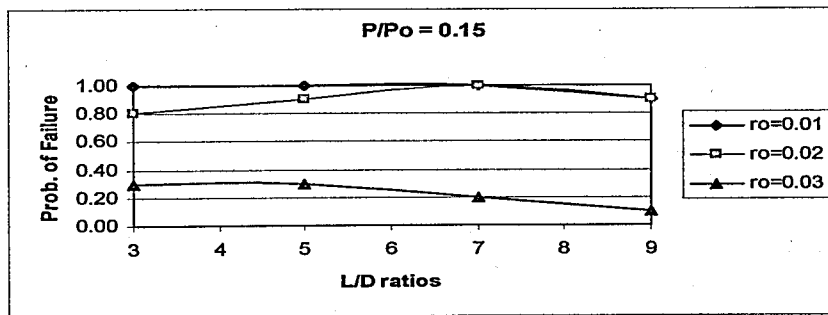


Figure 4.30. Probability of Failure for $PGA=0.40g$ and $P/P_0=0.15$

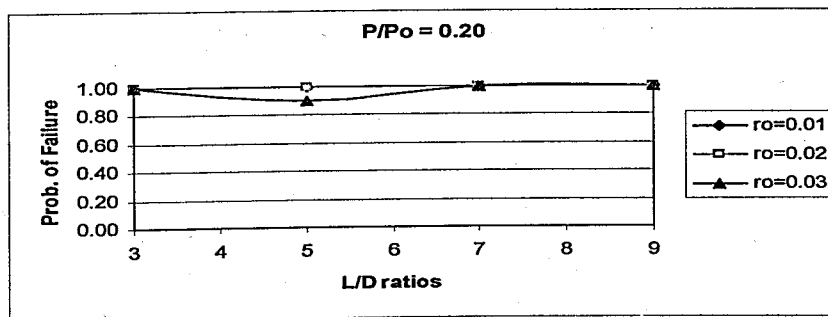


Figure 4.31. Probability of Failure for $PGA=0.40g$ and $P/P_0=0.20$

Table 4.23. Probability of Failure for $PGA = 0.20g$ and $f'_c = 25 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	0.60
		5	0.40
		7	0.00
		9	0.20
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.10
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00

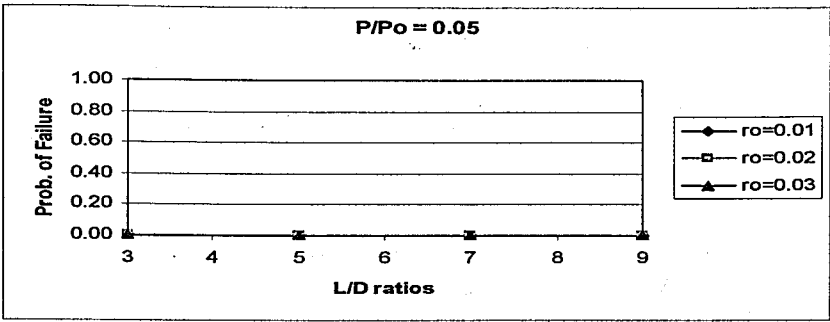


Figure 4.32. Probability of Failure for PGA=0.20g and P/P₀=0.05

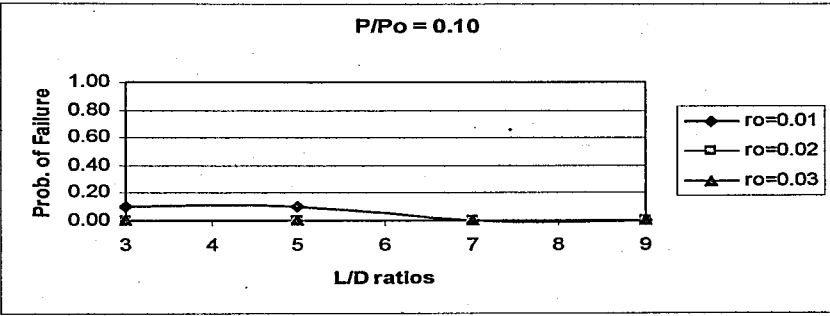


Figure 4.33. Probability of Failure for PGA=0.20g and P/P₀=0.10

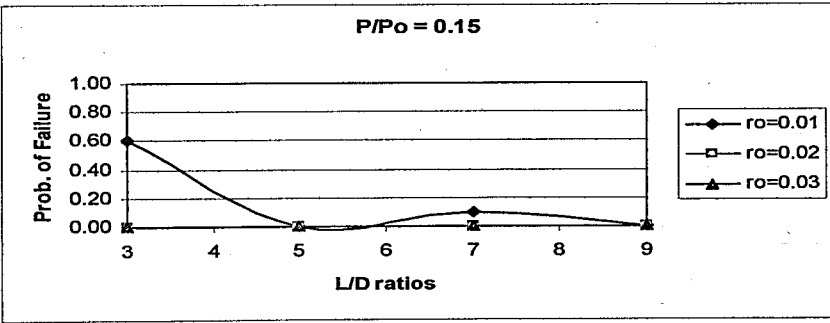


Figure 4.34. Probability of Failure for PGA=0.20g and P/P₀=0.15

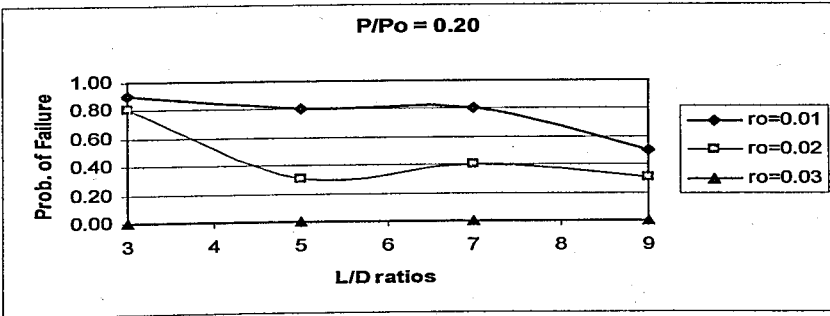


Figure 4.35. Probability of Failure for PGA=0.20g and P/P₀=0.20

Table 4.24. Probability of Failure for $PGA = 0.25g$ and $f'_c = 25 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.10
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.80
		5	0.10
		7	0.20
		9	0.00
	0.02	3	0.00
		5	0.10
		7	0.10
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	1.00
		5	1.00
		7	0.60
		9	0.90
	0.02	3	0.60
		5	0.60
		7	0.60
		9	0.30
	0.03	3	0.10
		5	0.10
		7	0.10
		9	0.00

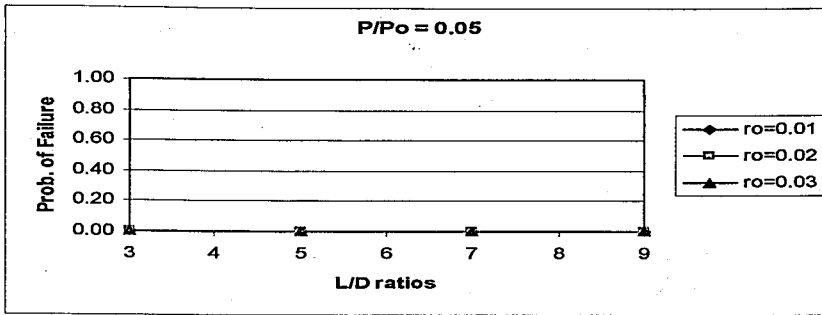


Figure 4.36. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.05$

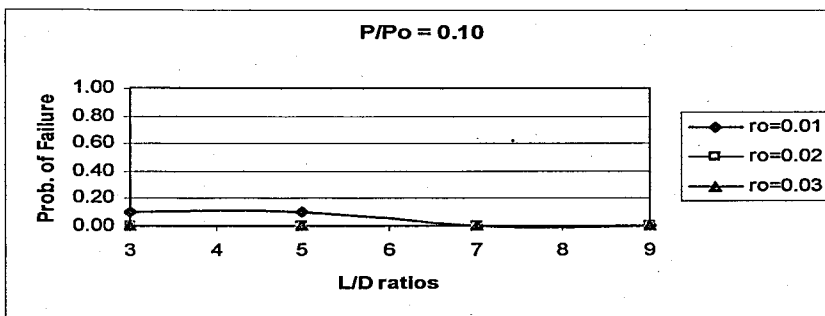


Figure 4.37. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.10$

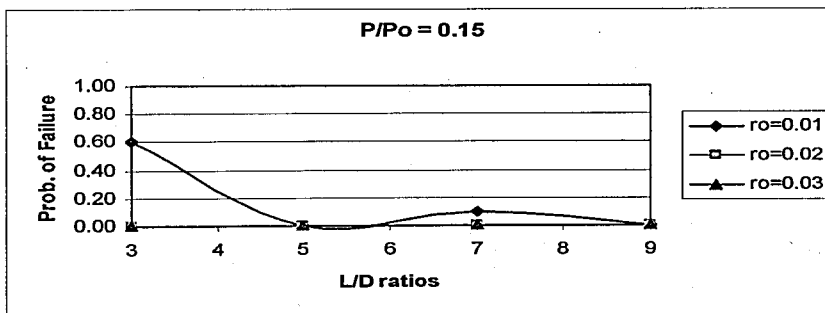


Figure 4.38. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.15$

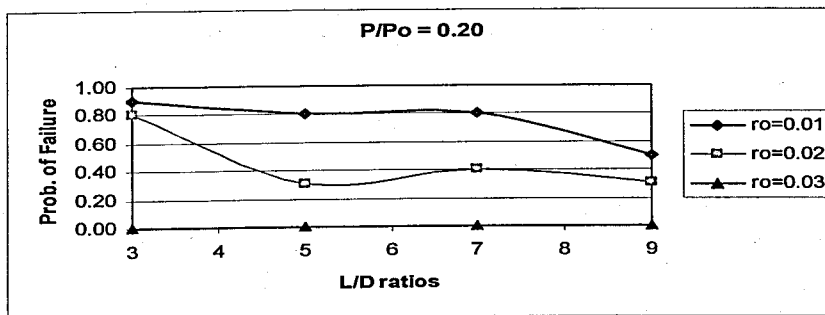


Figure 4.39. Probability of Failure for $PGA=0.20g$ and $P/P_0=0.20$

Table 4.25. Probability of Failure for $PGA = 0.25g$ and $f'_c = 25 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.10
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.80
		5	0.10
		7	0.20
		9	0.00
	0.02	3	0.00
		5	0.10
		7	0.10
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	1.00
		5	1.00
		7	0.60
		9	0.90
	0.02	3	0.60
		5	0.60
		7	0.60
		9	0.30
	0.03	3	0.10
		5	0.10
		7	0.10
		9	0.00

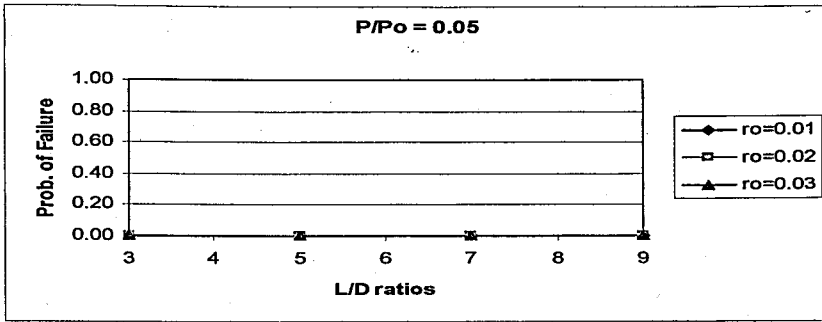


Figure 4.40. Probability of Failure for $PGA = 0.25g$, $f'_c = 25$ Mpa and $P/P_0=0.05$

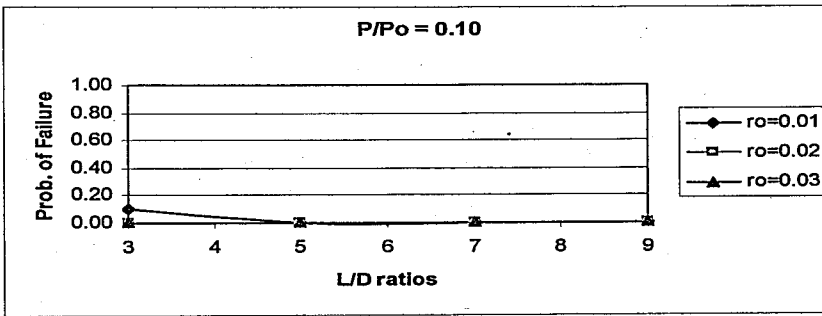


Figure 4.41. Probability of Failure for $PGA = 0.25g$, $f'_c = 25$ Mpa and $P/P_0=0.10$

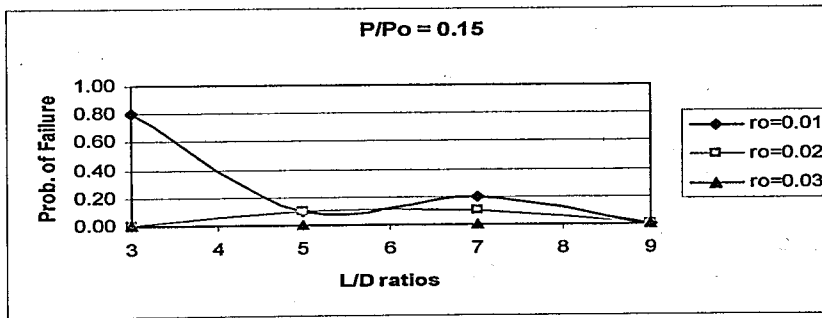


Figure 4.42. Probability of Failure for $PGA = 0.25g$, $f'_c = 25$ Mpa and $P/P_0=0.15$

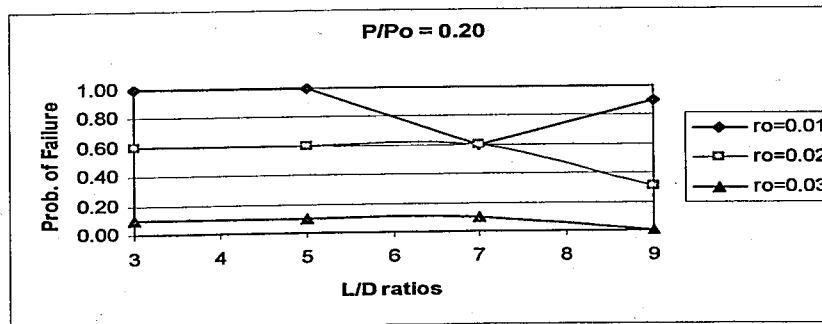


Figure 4.43. Probability of Failure for $PGA = 0.25g$, $f'_c = 25$ Mpa and $P/P_0=0.20$

Table 4.26. Probability of Failure for $PGA = 0.30g$ and $f'_c = 25 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.10
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.80
		5	0.10
		7	0.20
		9	0.00
	0.02	3	0.00
		5	0.10
		7	0.10
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	1.00
		5	1.00
		7	0.60
		9	0.90
	0.02	3	0.60
		5	0.60
		7	0.60
		9	0.30
	0.03	3	0.10
		5	0.10
		7	0.10
		9	0.00

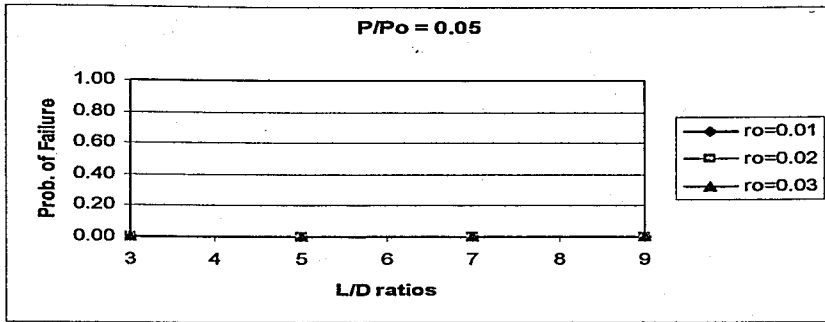


Figure 4.44. Probability of Failure for PGA = 0.30g, $f_c' = 25$ Mpa and $P/P_0=0.05$

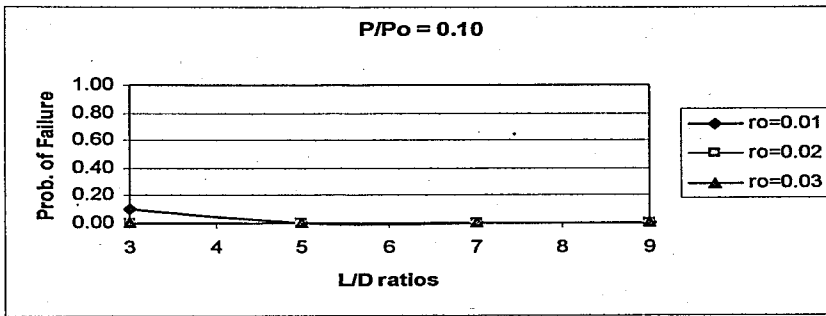


Figure 4.45. Probability of Failure for PGA = 0.30g, $f_c' = 25$ Mpa and $P/P_0=0.10$

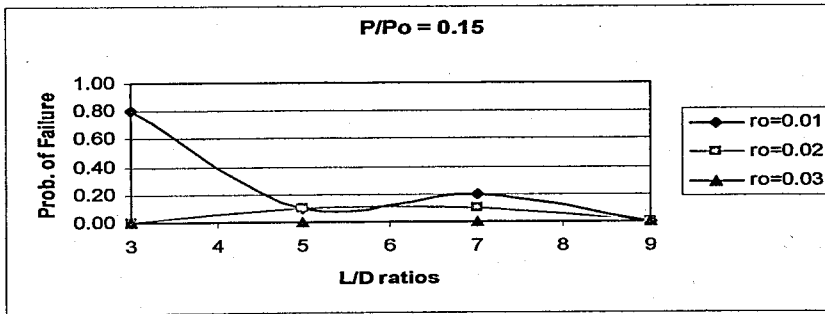


Figure 4.46. Probability of Failure for PGA = 0.30g, $f_c' = 25$ Mpa and $P/P_0=0.15$

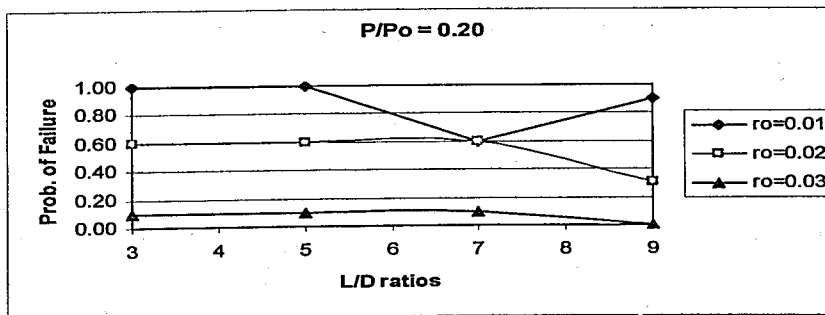


Figure 4.47. Probability of Failure for PGA = 0.30g, $f_c' = 25$ Mpa and $P/P_0=0.20$

Table 4.27. Probability of Failure for $PGA = 0.35g$ and $f'_c = 25 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.10
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	0.70
		5	0.10
		7	0.10
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.2	0.01	3	0.90
		5	0.90
		7	0.70
		9	0.60
	0.02	3	0.80
		5	0.90
		7	0.50
		9	0.50
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00

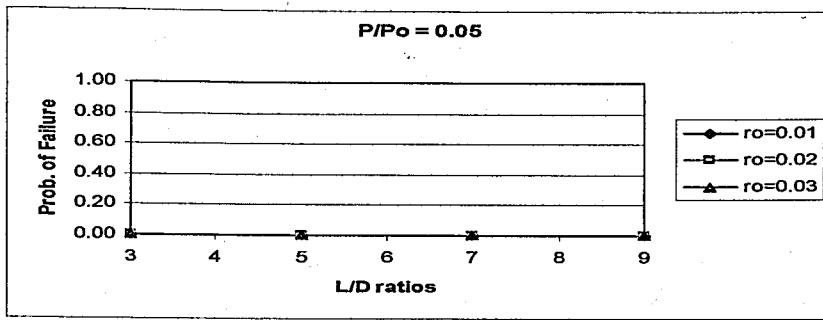


Figure 4.48. Probability of Failure for $\text{PGA} = 0.35\text{g}$, $f'_c = 25 \text{ Mpa}$ and $P/P_0 = 0.05$

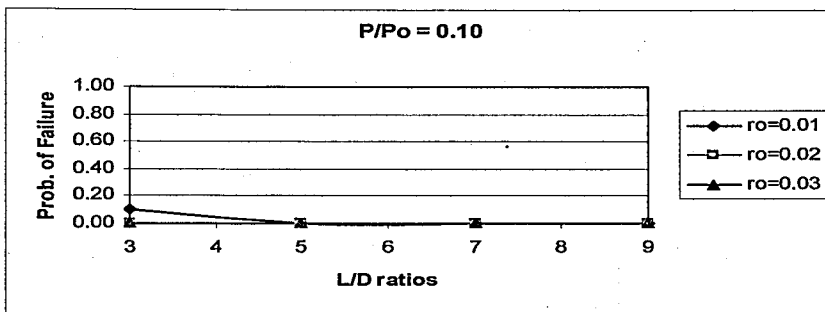


Figure 4.49. Probability of Failure for $\text{PGA} = 0.35\text{g}$, $f'_c = 25 \text{ Mpa}$ and $P/P_0 = 0.10$

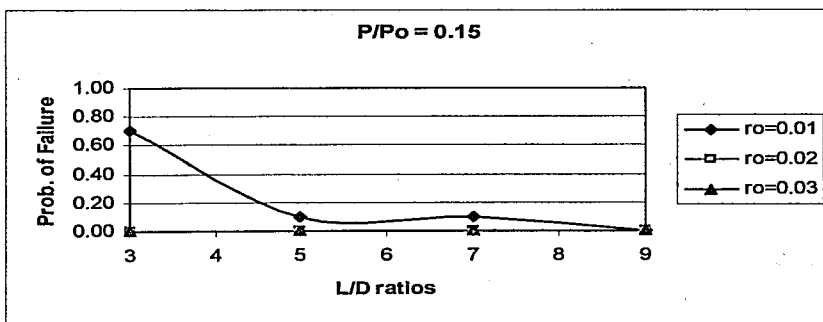


Figure 4.50. Probability of Failure for $\text{PGA} = 0.35\text{g}$, $f'_c = 25 \text{ Mpa}$ and $P/P_0 = 0.15$

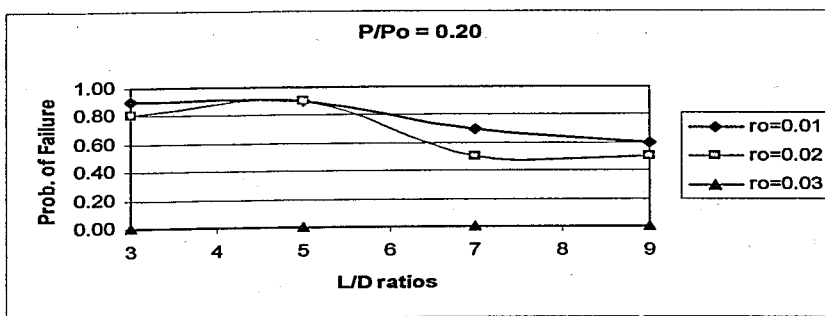


Figure 4.51. Probability of Failure for $\text{PGA} = 0.35\text{g}$, $f'_c = 25 \text{ Mpa}$ and $P/P_0 = 0.20$

Table 4.28. Probability of Failure for $PGA = 0.40g$ and $f'_c = 25 \text{ MPa}$

P/P_0	ρ	L/D	Probability of Failure
0.05	0.01	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.1	0.01	3	0.60
		5	0.20
		7	0.00
		9	0.10
	0.02	3	0.00
		5	0.00
		7	0.00
		9	0.00
	0.03	3	0.00
		5	0.00
		7	0.00
		9	0.00
0.15	0.01	3	1.00
		5	1.00
		7	0.90
		9	0.70
	0.02	3	0.70
		5	0.70
		7	1.00
		9	0.70
	0.03	3	0.30
		5	0.10
		7	0.10
		9	0.10
0.2	0.01	3	1.00
		5	1.00
		7	1.00
		9	1.00
	0.02	3	1.00
		5	1.00
		7	1.00
		9	1.00
	0.03	3	0.90
		5	0.90
		7	1.00
		9	0.80

5. CONCLUSIONS AND RECOMMENDATIONS

In this research, probabilistic approach to seismic evaluation of SDOF systems is proposed. A computer program that enables the user to compute both inelastic capacity and nonlinear demand of a SDOF structure is developed and utilized to predict the potential seismic hazard of the structure, namely bridges. In the nonlinear time history analysis procedure, force-displacement relationship is used in order to determine the inelastic stiffness of the system.

The results obtained herein are proposed to use for preliminary screening of an existing, fixed-base SDOF structures.

Basic parameters that define a SDOF structure are selected. These parameters are P/P_0 , ρ_l , PGA , f_c' and the aspect ratio, L/D . By keeping any of the parameters constant and changing the others, combinations of these parameters are considered to be the inputs of the analyses. Also, unconfined concrete model is selected assuming the existing reinforced concrete bridge columns do not have any ductility.

In this study, the parameters mentioned above are examined and their effects on the structural behavior and the probability of failure are calculated. In light of these extensive analyses, following conclusions are reached:

- When P/P_0 ratio is increased, the C/D ratio generally decreases. This can be interpreted as, with an increase in the axial load, additional moment is produced and that causes reduction in the capacity of the member. At the same time, with an increase in the mass (increase in axial load) of the system, the demand also increases as expected. Naturally, the probability of failure increases. However, P/P_0 with values less than 0.10 are almost ineffective on the percentage of failure, even with maximum code-specified PGA of 0.40g. Exception is seen only on the structures with longitudinal reinforcement smaller than 0.01, aspect ratios smaller than 4, and excited the ground motion with PGA 0.40g. In this case, probability of failure is between

0.40 – 0.60. Structures in this probability range are recommended to be evaluated in detail and refine analysis is needed.

- It should be noticed that all analyzed structures with P/P_0 ratio of 0.20 in seismic zone 1 collapsed. In one respect, this result resembles the recommendation of AASHTO [13] where the design axial force for flexural resistance should be less than $0.20P_0$. Certainly it does not mean that the structure would unavoidably fail.
- With an increase in the longitudinal reinforcement ratio an increase is observed in the capacity/demand ratio. This affirmative distinction is noted especially in the structures with greater P/P_0 (0.15 – 0.20) and greater PGA (0.30g – 0.40g). All structures with P/P_0 ratio of 0.05 and with different reinforcement ratios did not fail. This effect is also expected since the moment that should be carried by the reinforcement becomes smaller than before. This effect causes an increase in the capacity and decrease in the demand.
- When all parameters are kept constant and the aspect ratio is increased, a slight decrease in the percentage of failure is observed. This can be explained by increasing the aspect ratio; in many cases the earthquake demand increase is greater than the increase in the displacement capacity. Therefore capacity/demand ratio slightly decreases.

In the analysis of some subsequent parameters (P/P_0 , ρ and L/D), certain jumps in the percentage of the probability of failure are observed. The limited number of ground motions (10), used in the determination of the probability of failure, may be the reason for this discontinuity. The capacity/demand ratios below or above but very close to 1 are assumed to fail or pass respectively. Comparing the capacity/demand ratio exactly with 1 instead of a certain range (e.g. 0.95 – 1.05) may be another reason for these jumps. As a recommendation, the capacity/demand ratios of structures, which fall in that range, should be analyzed in detail. And also, in order to obtain refined probability of failure percentages the number of the ground motions should be increased.

APPENDIX A: MATLAB PROGRAMS

A.1. Capacity Analysis Main Code

```

clc;
clear all;
close all;

r=input('INPUT THE RADIUS OF THE SECTION (mm)=');
dT=input('INPUT THE ANGLE INCREMENT (degree) dT=');
[SA,TCL]=CCS(r,dT);
NR=input('INPUT THE NUMBER OF REINFORCEMENT, N=');
DR=input('INPUT THE DIAMETER OF THE REINFORCEMENT (mm), d=');
CC=input('INPUT CONCRETE COVER WIDTH (mm), cc=');
[TCLR,RF,RS]=CRLA(r,NR,DR,CC);

TCLaxialload=input('INPUT THE AXIAL LOAD LOCATION FROM
TOP(mm)=');
cm=1;
curvature(1)=0;
moment(1)=0;

ec=input('INPUT STRAIN VALUE=');
maxec=input('INPUT MAXIMUM STRAIN VALUE=');
e100=input('INPUT STRAIN VALUE FOR THE MAXIMUM CONCRETE
STRENGTH, e100=');
e85=input('INPUT STRAIN VALUE FOR THE 85 PERCENT OF THE
MAXIMUM CONCRETE STRENGTH WHICH IS GREATER THAN e100, e85=');
incec=input('INPUT INCREMENT STRAIN VALUE=');
fcp=input('INPUT CYLINDER STRENGTH OF CONCRETE (MPa)=');
axialload=input('INPUT AXIAL LOAD (MPa)=');
ey=input('INPUT STEEL STRAIN AT THE STEEL YIELDING POINT =');
eh=input('INPUT STEEL STRAIN AT THE STEEL HARDENING POINT =');

```

```

eu=input('INPUT STEEL STRAIN AT THE STEEL ULTIMATE STRENGTH
POINT =');
ess=0;
maxes=0.02;
fy=input('INPUT STEEL STENGTH AT THE STEEL YIELDING POINT
(MPa)=');
fh=input('INPUT STEEL STENGTH AT THE STEEL HARDENING POINT
(MPa)=');
fu=input('INPUT ULTIMATE STEEL STENGTH (MPa)=');
deltacur=0;
mom(1)=0;
momm=0;
slope=0;
stripno=length(TCL);
cm=2;
concstress=0;
while ec<=maxec&ess<=maxes&momm>=0&deltacur>=0&concstress>=0
    i=1;
    diff=-1;
    while (i<=stripno)&(diff<0);
        j=i;
        c=TCL(j);
        slope=ec/c;
        totalconcforce=0;
        totalsteelforce=0;
        totalconcmoment=0;
        totalsteelmoment=0;
        while j>=1
            concstrain=slope*(c-TCL(j));
            [concstress]=hognestad(fcp,concstrain,e100,e85);
            concforce=concstress*SA(j);
            totalconcforce=totalconcforce+concforce;
            totalconcmoment=totalconcmoment+concforce*(c-TCL(j));

```

```

    j=j-1;
end
for k=1:RS
    es=slope*(c-TCLR(k));
    steelstrain=abs(es);
    [steelstress]=steel(steelstrain,ey,eh,eu,fy,fh,fu);
    if es>0
        steelstress=steelstress;
    else
        steelstress=-1*steelstress;
    end
    steelforce=steelstress*RF(k);
    totalsteelforce=totalsteelforce+steelforce;
    totalsteelmoment=totalsteelmoment+steelforce*(c-TCLR(k));
end
diff=totalconccforce+totalsteelforce-axialload;
totalconccforce;
totalsteelforce;
i=i+1;
end
curvature=slope;
moment=totalconccmoment+totalsteelmoment+axialload*(TCLaxialload-c);
%      if mom(1)<=moment
mom(cm)=moment;
cur(cm)=curvature;

momm=mom(cm);
deltacur=cur(cm)-cur(cm-1);
cm=cm+1;
ec=ec+incec;
end

L=input('INPUT THE COLUMN LENGTH (mm) L='); %column length

```

```

[force,displacement]=FDson(mom,cur,L);

datano=1

filename=['ForceData',num2str(datano),'.dat'];
save(filename,'force','-ascii');

filename=['DisplacementData',num2str(datano),'.dat'];
save(filename,'displacement','-ascii');

```

A.2. Functions Used in the Capacity Code

```

function [SA,TCL]=CCS(r,dT)
% Creation of the Circular Column Strips (CCS)
CA=pi*(r)^2; % claculates the area of the column
dT=dT*pi/180; % converts the angle from degree to radian
% initialize
CW(1)=0;
TCL=0;
EA=0;
lengthh=pi/dT;
T=0:dT:pi;
CW=2*r*sin(T);
SN=1:lengthh;
for i=1:lengthh
    SW(i)=(CW(i+1)+CW(i))/2;
    SH(i)=r*(cos(T(i))-cos(T(i+1)));
    SA(i)=SW(i)*SH(i);
end
TCL(1)=0;
for i=1:lengthh
    TCL(i+1)=TCL(i)+SH(i);

```

[illegible]

```

function [force,displacement]=FDdeneme(mom,cur,L)
%Generates the force-displacement graph with respect to the
%moment-curvature--

displacement=0;
znak=0;% values which exceeds maxmax and reaches the second plastic length
% Creation of the force-displacement graph from the moment-curvature graph
displacement(1)=0;
force(1)=0;
sign=1;% sign for the determination of the position of i on the moment-curvature graph
for i=2:(length(mom)-1)
    if mom(i)>mom(i+1)&sign==1
        maxmom=mom(i);
        sign=2;
    elseif mom(i)<mom(i+1)&sign==2
        minmom=mom(i);
        sign=3;
    end
end
sign=1;
for i=2:(length(mom)-1)
    if mom(i)>mom(i+1)&sign==1 %point D
        maxmomi=i;

        deltaM=0;
        for k=1:(i-1)
            deltaM(k)=mom(k+1)-mom(k);
        end
        unit=(L)/sum(deltaM);
        ll=0;
        for p=1:(i-1)
            ll(p)=deltaM(p)*unit;%dilim basina gelen uzunluk
        end
    end
end

```

```

    ll=cumsum(ll);
    dispp=cur(2)*ll(1)*(1/2)*ll(1)*(2/3);
    for j=3:(i)
        dispp=((cur(j)-cur(j-1))/2*ll(j-1))*(ll(j-2)+ll(j-1)*2/3)+(cur(j-1)*ll(j-1))*(ll(j-2)+ll(j-1)/2)+dispp;
    end
    displacement(i)=dispp;
    force(i)=mom(i)/L;

    sign=2;

elseif mom(i)<mom(i+1)&sign==2; %point F
    Lpl=(1-(mom(i)/maxmom))*L; %plastic length of the column with respect to the
    applied moment
    Mpl=mom(i)*(L-Lpl)/L; %moment value for the begining of the plastification
    t=1;
    while Mpl>mom(t);
        t=t+1;
    end
    dilim_sayisi=t;
    curpl=((((cur(t)-cur(t-1))/(mom(t)-mom(t-1)))*(Mpl-mom(t-1)))+cur(t-1);

    deltaM=0;
    for k=1:(dilim_sayisi-2)
        deltaM(k)=mom(k+1)-mom(k);
    end
    k=k+1;
    deltaM(k)=Mpl-mom(k);
    unit=(L-Lpl)/sum(deltaM);
    ll=0;
    for p=1:(dilim_sayisi-1)
        ll(p)=deltaM(p)*unit; %dilim basina gelen uzunluk
    end

```

```

p=p+1;
ll(p)=Lpl;
ll=cumsum(ll);

dispp=cur(2)*ll(1)*(1/2)*ll(1)*(2/3);
for j=3:(dilim_sayisi-1)
    dispp=((cur(j)-cur(j-1))/2*ll(j-1))*(ll(j-2)+ll(j-1)*2/3)+(cur(j-1)*ll(j-1))*(ll(j-2)+ll(j-1)/2)+dispp;
end
dispcurpl=(curpl-cur(dilim_sayisi-1))*ll(dilim_sayisi-1)/2*(ll(dilim_sayisi-2)+ll(dilim_sayisi-1)*2/3)+cur(dilim_sayisi-1)*ll(dilim_sayisi-1)*(ll(dilim_sayisi-2)+ll(dilim_sayisi-1)/2);
displacement(i)=dispp+dispcurpl+(Lpl*cur(i))*(ll(dilim_sayisi-1)+ll(dilim_sayisi)*(1/2));
if displacement(i)<displacement(i-1)
    deltadispp=abs(displacement(i-1)-displacement(i));
    displacement(i)=abs(displacement(i-1))+deltadispp;
end
force(i)=mom(i)/L;
sign=3;
F=2;
disp('F');

elseif mom(i)<=mom(i+1)&sign==1; %points A,B,C
    deltaM=0;
    for k=1:(i-1)
        deltaM(k)=mom(k+1)-mom(k);
    end
    unit=(L)/sum(deltaM);
    ll=0;
    for p=1:(i-1)
        ll(p)=deltaM(p)*unit;%dilim basina gelen uzunluk
    end

```

```

    ll=cumsum(ll);
    dispp=cur(2)*ll(1)*(1/2)*ll(1)*(2/3);
    if i>2
        for j=3:(i);
            dispp=((cur(j)-cur(j-1))/2*ll(j-1))*(ll(j-2)+ll(j-1)*2/3)+(cur(j-1)*ll(j-1))*(ll(j-
2)+ll(j-1)/2)+dispp;
        end
    end
    displacement(i)=dispp;
    force(i)=mom(i)/L;
    disp('ABC');
elseif mom(i)>mom(i+1)&sign==2; %point E
    Lpl=(1-(mom(i)/maxmom))*L; %plastic length of the column with respect to the
    applied moment
    Mpl=mom(i)*(L-Lpl)/L; %moment value for the begining of the plastification
    t=1;
    while Mpl>mom(t)
        t=t+1;
    end
    dilim_sayisi=t;
    curpl=((((cur(t)-cur(t-1))/(mom(t)-mom(t-1)))*(Mpl-mom(t-1)))+cur(t-1);

    deltaM=0;
    for k=1:(dilim_sayisi-2)
        deltaM(k)=mom(k+1)-mom(k);
    end
    k=k+1;
    deltaM(k)=Mpl-mom(k);
    unit=(L-Lpl)/sum(deltaM);
    ll=0;
    for p=1:(dilim_sayisi-1)
        ll(p)=deltaM(p)*unit; %dilim basina gelen uzunluk
    end

```

```

p=p+1;
ll(p)=Lpl;
lll=cumsum(ll);

dispp=cur(2)*ll(1)*(1/2)*ll(1)*(2/3);
for j=3:(dylim_sayisi-1)
    dispp=((cur(j)-cur(j-1))/2*ll(j-1))*(lll(j-2)+ll(j-1)*2/3)+(cur(j-1)*ll(j-1))*(lll(j-
2)+ll(j-1)/2)+dispp;
end
dispcurpl=(curpl-cur(dylim_sayisi-1))*ll(dylim_sayisi-1)/2*(lll(dylim_sayisi-
2)+ll(dylim_sayisi-1)*2/3)+cur(dylim_sayisi-1)*ll(dylim_sayisi-1)*(lll(dylim_sayisi-
2)+ll(dylim_sayisi-1)/2);
displacement(i)=dispp+(Lpl*cur(i))*(lll(dylim_sayisi-
1)+ll(dylim_sayisi)*(1/2))+dispcurpl;
if displacement(i)<displacement(i-1)
    deltadispp=abs(displacement(i-1)-displacement(i));
    displacement(i)=abs(displacement(i-1))+deltadispp;
end
force(i)=mom(i)/L;
E=4;
disp('E');

else %point G,H,I,J,K
    Lpl=(1-(mom(i)/maxmom))*L; %plastic length of the column with respect to the
applied moment
    Mpl=mom(i)*(L-Lpl)/L; %moment value for the beginning of the plastification

    t=1;
    while Mpl>mom(t)&t<=i
        t=t+1;
    end

```

```

dilim_sayisi=t;
curpl=((((cur(t)-cur(t-1))/(mom(t)-mom(t-1)))*(Mpl-mom(t-1)))+cur(t-1);%cur(i);

deltaM=0;
for k=1:(dilim_sayisi-2)
    deltaM(k)=mom(k+1)-mom(k);
end
k=k+1;
deltaM(k)=Mpl-mom(k);
unit=(L-Lpl)/sum(deltaM);
ll=0;
for p=1:(dilim_sayisi-1)
    ll(p)=deltaM(p)*unit; %dilim basina gelen uzunluk
end
p=p+1;
ll(p)=Lpl;
lll=cumsum(ll);

dispp=cur(2)*ll(1)*(1/2)*ll(1)*(2/3);
for j=3:(dilim_sayisi-1)
    dispp=((cur(j)-cur(j-1))/2*ll(j-1))*(lll(j-2)+ll(j-1)*2/3)+(cur(j-1)*ll(j-1))*(lll(j-
2)+ll(j-1)/2)+dispp;
end
dispcurpl=(curpl-cur(dilim_sayisi-1))*ll(dilim_sayisi-1)/2*(lll(dilim_sayisi-
2)+ll(dilim_sayisi-1)*2/3)+cur(dilim_sayisi-1)*ll(dilim_sayisi-1)*(lll(dilim_sayisi-
2)+ll(dilim_sayisi-1)/2);
displacement(i)=dispp+(Lpl*cur(i))*(lll(dilim_sayisi-
1)+ll(dilim_sayisi)*(1/2))+dispcurpl;
if displacement(i)<displacement(i-1)
    deltadispp=abs(displacement(i-1)-displacement(i));
    displacement(i)=abs(displacement(i-1))+deltadispp;
end

```

```

    force(i)=mom(i)/L;
    disp('GHIJK');
end
displacement(i);
disp(displacement(i));
end

```

```

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

```

```

function [concestress]=hognestad(fcp,ecc,e100,e85)
% Calculates strength using Hognestad's parabolic portion
% called in capacity
% fcp=cylinder strength, fc'
% fcpp=maximum cylinder strength, fc''=fc'*k3
if ecc<=e100
    concestress=fcp*((2*ecc/e100)-(ecc/e100)^2);
else
    concestress=fcp-(0.15*fcp)/(e85-e100)*(ecc-e100);
end

```

```

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

```

```

function [SA,TCL]=RCS(b,h)
% Creation of the Rectangular Column Strips (RCS)
% CA=Cross Section Area
% SW=Section Width
% SH=Section Height
% TCL=Tension/Compression Location
% SA=Strip Area
% EA=Estimated Area of the section
SH=h/100;
CA=b*h;
SW=b;

```

$$TCL(1)=SH/2;$$
$$SA(1)=SH*SW;$$

```
for i=1:99
```

$$TCL(i+1)=TCL(i)+SH;$$
$$SA(i+1)=SH*SW;$$

end

$$EA=100*SA;$$
[illegible]

```
function [RF]=RRLA(lengthTCLR,NR,DR)
```

% Estimates the locations and the area of the reinforcement

% RRLA=Rectangular Reinforcement Location and Area

% TCL=Tension/Compression Location

% RF=Reinforcement Area per row

%

00000

```
for i=1:lengthTCLR
```

```
RF(i)=NR(i,:)*(pi*DR(i,:)^2)/4;
```

end

[illegible]

```
function [sect]=section
```

```
% Asks the user for a section type
```

```
section=1;%menu('CHOOSE A SECTION','CIRCULAR','RECTANGULAR');
```

```
if section==1
```

sect=1;

else

sect=2

end

[illegible]

```

function [steelstress]=steel(es,ey,eh,eu,fy,fh,fu)
% Estimates the steel stress for the corresponding strain
% es=steel strain
% ey=steel strain at the yield point
% eh=steel strain at the hardening point
% eu=steel strain at the ultimate point
if es<=ey
    steelstress=(fy/ey)*es;
elseif es>ey&es<=eh
    steelstress=fy;
% elseif es>eh&es<=eu
%   term1=fu-fh;
%   term2=(es-eh)/(eu-eh);
%   steelstress=fh+term1*(2*term2-(term2)^2);
elseif es>eh&es<=eu
    steelstress=fh+(fu-fh)/(eu-eh)*(es-eh);
elseif es>eu
    steelstress=fu;
end

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

```

A.3. Dynamic Analysis Main Code

```

clc
clear all;
close all;

datano=1
failure=0;
EQindex=1

[ug,d,FF,displacement_capacity]=loadall_son(datano,EQindex);

```

```

[gama,beta]=method;
% Initial Calculations
[umax,umin,Fmax,Fmin,ke]=kee(d,FF); % Estimates the stiffness for the elastic range
FF=FF*100; % Converts from (N) to (kg.cm/sec^2)
m=input('Input the mass (in kilograms) m=');
p=(-ug)*m;
ksi=0.05;%input('Input the damping ratio ksi=');
c=2*ksi*(ke*m)^(1/2); % Estimation of the damping coefficient
u(1)=0;
du(1)=0;
ddu(1)=(p(1)-c*du(1)-ke*u(1))/m; % Step 1.1 in the main procedure
dt=0.01;%input('Input time increment (in seconds) dt='); % Step 1.2 in the main procedure
a=m/beta/dt+gama*c/beta; % Step 1.3 in the main procedure
b=m/2/beta+dt*c*(gama/2/beta-1); % Step 1.3 in the main procedure
lengthd=length(d);
lengthp=length(p)-1;
flag=0; % A sign which determines the structure to be in the elastic or inelastic range
region=0; % A sign which gives the region of the last displacement and force value
uumin(1)=0; % The minimum displacement vector
uumax(1)=0; % The maximum displacement vector
% Calculations for each time step i
i=1;
while u(i)>=d(1)&u(i)<=d(lengthd)&lengthp>=i
    deltap(i)=p(i+1)-p(i);
    deltapsapka(i)=deltap(i)+a*du(i)+b*ddu(i); % Step 2.1 in the main procedure
    uii=u(i);
    % The beginning for the step 2.2 in the main procedure
    if flag==0
        [kii,Fii,flag,umax,umin,Fmax,Fmin,region]=stiffnesse(uii,ke,flag,d,FF,umax,umin,Fmax,Fmin);
    else
        ui=u(i-1);

```

```

    Fi=F(i-1);

[kii,Fii,umax,umin,Fmax,Fmin,region]=stiffnessi(ui,uii,ke,d,FF,umax,umin,Fmax,Fmin,Fi)
;
    end
    k(i)=kii;
    % The End for the step 2.2 in the main procedure
    F(i)=Fii;
    ksapka(i)=k(i)+gama*c/(beta*dt)+m/(beta*(dt)^2); % Step 2.3 in the main procedure
    deltau(i)=deltapsapka(i)/ksapka(i); % Step 2.4 in the main procedure
    deltadu(i)=gama*deltau(i)/(beta*(dt))-gama*du(i)/beta+dt*ddu(i)*(1-gama/(beta*2)); %
Step 2.5 in the main procedure
    deltaddu(i)=deltau(i)/(beta*(dt)^2)-du(i)/(beta*dt)-ddu(i)/(2*beta);% Step 2.6 in the
main procedure
    u(i+1)=u(i)+deltau(i); % Step 2.7 in the main procedure
    du(i+1)=du(i)+deltadu(i); % Step 2.7 in the main procedure
    ddu(i+1)=ddu(i)+deltaddu(i); % Step 2.7 in the main procedure
    regionv(i)=region;
    uumin(i)=umin;
    uumax(i)=umax;
    i=i+1;
end

umax=max(u);
umin=min(u);
F(i)=F(i-1);
k(i)=k(i-1);
t(1)=0;
for i=2:length(u)
    t(i)=t(i-1)+dt; % Time vector
end
uumin(i)=uumin(i-1);
uumax(i)=uumax(i-1);

```

```

displacement_demand=10*max(abs(u));
capacity_demand_ratio=displacement_capacity/displacement_demand;

CapacityTable(EQindex,datano)=displacement_capacity;

DemandTable(EQindex,datano)=displacement_demand;

CapacityDemandRatioTable(EQindex,datano)=capacity_demand_ratio;

if capacity_demand_ratio<1
    FailureStateTable(EQindex,datano)=1;
    failure=failure+1;
else
    FailureStateTable(EQindex,datano)=0;
end
EQindex
end
ProbabilityOfFailure(datano)=failure/10;
datano
end

ProbabilityOfFailure=ProbabilityOfFailure';

filename=['ProbabilityOfFailure.dat'];
save(filename,'ProbabilityOfFailure','-ascii');

```

A.4. Functions Used in the Dynamic Analysis Code

```

function y=ke(d,FF)
Fmax=0.75*max(FF);
Fmin=(-1)*Fmax;
n=1;
while Fmax>=FF(n)

```

```

n=n+1;
end
umax=d(n-1);
umin=(-1)*umax;
ke=Fmax/umax;

```

```

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

```

```

function [umax,umin,Fmax,Fmin,ke]=kee(d,FF)

```

```

% Evaluates the stiffness for the elastic behaviour.

```

```

% INPUTS:

```

```

%   d: the displacement vector taken from the backbone curve and the values are
%   ordered from
%   the minimum to the maximum value. The units must be in (cm).
%   FF: the force vector taken from the backbone curve and the values are ordered with
%   respect to
%   the displacement vector, d. The units must be in (N).

```

```

% OUTPUTS:

```

```

%   Fmax: a force, which for the elastic behavior is equal to 75 percent of the maximum
%   FF component
%   and for the inelastic behavior is computed with respect to umax. (kg.cm/sec2)
%   Fmin: a force, which for the elastic behavior is equal to 75 percent of the minimum
%   FF component
%   and for the inelastic behavior is computed with respect to umin. (kg.cm/sec2)
%   ke: the stiffness for the elastic behavior. ((kg.cm/sec2)/cm)
%   umax: the maximum displacement obtained up to the time step, i+1. For the elastic
%   behavior of the
%   system its value is obtained with respect to Fmax and for the inelastic behavior it is
%   equal to a
%   displacement, which is greater than the umax obtained up to that time step, i. (cm)
%   umin: the minimum displacement obtained up to the time step, i+1. For the elastic
%   behavior of the

```

```
%    system its value is obtained with respect to Fmin and for the inelastic behavior it is
%    equal to a
%    displacement, which is smaller than the umin obtained up to that time step, i. (cm)
```

```
Fmax=0.75*max(FF);
Fmin=(-1)*Fmax;
n=1;
while Fmax>=FF(n)
    n=n+1;
end
umax=d(n-1);
umin=(-1)*umax;
ke=(fix((fix(Fmax/umax))/5))*5*100;
Fmax=Fmax*100;
Fmin=Fmin*100;
```

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
```

```
function [kii,Fii]=kiiFiineg(d,FF,uii)
```

```
% Evaluates the stiffness when the current displacement, u(i), exceeds
% the minimum displacement, umin, in the backbone curve.
```

```
% INPUTS:
```

```
%    d: the displacement vector taken from the backbone curve and the
%    values are ordered from the minimum to the maximum value. The units
%    must be in (cm).
```

```
%    FF: the force vector taken from the backbone curve and the values
%    are ordered with respect to the displacement vector, d. The units
%    must be in (kg.cm/sec2).
```

```
%    uii: a parameter which corresponds to the current displacement, u(i).
%    (cm)
```

```
% OUTPUTS:
```

```
%    Fii: a parameter which corresponds to the current force, F(i).
%    (kg.cm/sec2)
```

```

%      kii:the stiffness which corresponds to the current stiffness, k(i).
%      ((kg.cm/sec2)/cm)

Fa=0;
Fb=0;
ua=0;
ub=0;
n=1;
while uii>=d(n)
    n=n+1;
end
ua=d(n-1);
Fa=FF(n-1);
ub=d(n);
Fb=FF(n);
kii=abs((Fa-Fb)/(ua-ub));
Fii=((Fa-Fb)/(ua-ub))*uii+Fb-((Fa-Fb)/(ua-ub))*ub;

%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

function [ug,d,FF,displacement_capacity]=loadall_son(datano,EQindex)
% Loads the ground acceleration vector and the displacement and force
% vectors taken from the backbone curve.
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
% INPUTS:
%      Z3_040_Acc.dat: The applied force vector in a text file.(mg)
%      DisplacementData.dat: The displacement vector taken from the
%      backbone curve in a text file.(cm)
%      ForceData.dat: The force vector taken from the backbone curve in
%      a text file.(N) or (kg.m/sec2)
% OUTPUTS:
%      d:      the displacement vector taken from the backbone curve
%      and the values are ordered from the minimum to the maximum

```

```
%      value. (cm).
%      FF:      the force vector taken from the backbone curve and the
%      values are ordered with respect to the displacement
%      vector, d. (N) or (kg.m/sec2)
%      ug:      the ground acceleration (cm/sec2)
% Please locate the applied force vector and displacement and force vectors
% of the backbone curve with respect to the explanations above.
```

```
ug=load(['Z3_040_Acc',num2str(EQindex),'.dat']);
ug=ug*9.81*100;% multiplied by 9.81/10 in order to convert to cm/sn^2
```

```
force=load(['ForceData',num2str(datano),'.dat']);
displacement=load(['DisplacementData',num2str(datano),'.dat']);
```

```
forcedisplacement=[force;displacement]';
```

```
[pgasort,order]=sort(forcedisplacement(:,1));
forcedisplacement=forcedisplacement(order,:);
```

```
displacement_capacity=forcedisplacement((length(force)),2);
```

```
lengthdisplacement=length(displacement);
length_doubledisp=2*lengthdisplacement-1;
for i=1:(lengthdisplacement-1);
    doubledisp(i)=-1*displacement(lengthdisplacement-i+1);
    doubleforce(i)=-1*force(lengthdisplacement-i+1);
end
for i=1:lengthdisplacement;
    doubledisp(lengthdisplacement+i-1)=displacement(i);
    doubleforce(lengthdisplacement+i-1)=force(i);
end
```

```
d=doubledisp/10; % d is converted from mm to cm
```

```
FF=doubleforce;
```

```
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
```

```
function
```

```
[kii,Fii,flag,umax,umin,Fmax,Fmin,region]=stiffnesse(uii,ke,flag,d,FF,umax,umin,Fmax,Fmin)
```

```
% Determinates the stiffness considering that the system behaves linearly.
```

```
% INPUTS:
```

```
% d: the displacement vector taken from the backbone curve and the values
% are ordered from the minimum to the maximum value. The units are in (cm).
% FF: the force vector taken from the backbone curve and the values are
% ordered with respect to the displacement vector, d. The units are in (N).
% flag: a sign that indicates the systems behavior. If its value equals to 0 then
% the system behaves linearly and when its value equals to 1 than the
% system behaves nonlinearly.
% Fmax: a force, which for the elastic behavior is equal to 75 percent of the
% maximum FF component and for the inelastic behavior is computed with
% respect to umax. (kg.cm/sec2)
% Fmin: a force, which for the elastic behavior is equal to 75 percent of the
% minimum FF component and for the inelastic behavior is computed with
% respect to umin. (kg.cm/sec2)
% ke: the stiffness for the elastic behavior. ((kg.cm/sec2)/cm)
% uii: a parameter which corresponds to the current displacement, u(i). (cm)
% umax: the maximum displacement obtained up to the time step, i+1. For the
% elastic behavior of the system its value is obtained with respect to
% Fmax and for the inelastic behavior it is equal to a displacement,
% which is greater than the umax % obtained up to that time step, i.
% (cm)
% umin: the minimum displacement obtained up to the time step, i+1. For the
% elastic behavior of the system its value is obtained with respect to Fmin
% and for the inelastic behavior it is equal to a displacement, which is
```

```

%      smaller than the umin %      obtained up to that time step, i. (cm)
% OUTPUTS:
%      Fii:   a parameter which corresponds to the current force, F(i). (kg.cm/sec2)
%      flag:  a sign that indicates the systems behavior. If its value equals to 0
%             then the system behaves linearly and when its value equals to 1 then
%             the system behaves nonlinearly.
%      Fmax:  a force, which for the elastic behavior is equal to 75 percent of the
%             maximum FF component and for the inelastic behavior is computed with
%             respect to umax. (kg.cm/sec2)
%      Fmin:  a force, which for the elastic behavior is equal to 75 percent of the
%             minimum FF component and for the inelastic behavior is computed with
%             respect to umin. (kg.cm/sec2)
%      kii:   the stiffness which corresponds to the current stiffness, k(i).
%      region: a sign which gives the position of the point, described by the last
%             displacement and force obtained with respect to that displacement,
%             in the coordinate system. The value for the parameter 'region' is
%             obtained by considering the previous %      displacement and force, and
%             the maximum or minimum displacement obtained up to that moment.
%      umax:  the maximum displacement obtained up to the time step, i+1. For the
%             elastic behavior of the system its value is obtained with respect to Fmax
%             and for the inelastic behavior it is equal to a displacement, which is
%             greater than the umax obtained up to that time step, i. (cm)
%      umin:  the minimum displacement obtained up to the time step, i+1. For the
%             elastic behavior of the system its value is obtained with respect to Fmin
%             and for the inelastic behavior it is equal to a displacement, which is
%             smaller than the umin obtained up to that time step, i. (cm)
%      if uii>=0
%          if uii<=umax
%              kii=ke;
%              Fii=kii*uii;
%              flag=0;
%              region=1;
%          end

```

```

    if uii>umax
        [kii,Fii]=kiiFiipos(d,FF,uii);
        umax=uii;
        Fmax=Fii;
        flag=1;
        region=2;
    end
end
if uii<0
    if uii>=umin
        kii=ke;
        Fii=kii*uii;
        flag=0;
        region=3;
    end
    if uii<umin
        [kii,Fii]=kiiFiineg(d,FF,uii);
        umin=uii;
        Fmin=Fii;
        flag=1;
        region=4;
    end
end
end

```

%%%

function

[kii,Fii,umax,umin,Fmax,Fmin,region]=stiffnessi(ui,uii,ke,d,FF,umax,umin,Fmax,Fmin,Fi)

% Determinates the stiffness considering that the system behaves nonlinearly.

% INPUTS:

% d: the displacement vector taken from the backbone curve and the values

% are ordered from the minimum to the maximum value. The units are in (cm).

% FF: the force vector taken from the backbone curve and the values are

```

%      ordered with respect to the displacement vector, d. The units are in (N).
%      Fi:   a parameter which corresponds to the previous force, F(i-1). (kg.cm/sec2)
%      Fmax:  a force, which for the elastic behavior is equal to 75 percent of the
%             maximum FF component and for the inelastic behavior is computed with
%             respect to umax. (kg.cm/sec2)
%      Fmin:  a force, which for the elastic behavior is equal to 75 percent of the
%             minimum FF component and for the inelastic behavior is computed with
%             respect to umin. (kg.cm/sec2)
%      ke:    the stiffness for the elastic behavior. ((kg.cm/sec2)/cm)
%      ui:    a parameter which corresponds to the previous displacement, u(i-1). (cm)
%      uii:   a parameter which corresponds to the current displacement, u(i). (cm)
%      umax:  the maximum displacement obtained up to the time step, i+1. For the
%             elastic behavior of the system its value is obtained with respect to
%             Fmax and for the inelastic behavior it is equal to a displacement, which
%             is greater than the umax obtained up to that time step, i. (cm)
%      umin:  the minimum displacement obtained up to the time step, i+1. For the
%             elastic behavior of the system its value is obtained with respect to
%             Fmin and for the inelastic behavior it is equal to a displacement, which
%             is smaller than the umin obtained up to that time step, i. (cm)
% OUTPUTS:
%      Fii:   a parameter which corresponds to the current force, F(i). (kg.cm/sec2)
%      Fmax:  a force, which for the elastic behavior is equal to 75 percent of the
%             maximum FF component and for the inelastic behavior is computed
%             with respect to umax. (kg.cm/sec2)
%      Fmin:  a force, which for the elastic behavior is equal to 75 percent of the
%             minimum FF component and for the inelastic behavior is computed with
%             respect to umin. (kg.cm/sec2)
%      kii:   the stiffness which corresponds to the current stiffness, k(i).
%      region: a sign which gives the position of the point, described by the last
%             displacement and force obtained with respect to that displacement, in
%             the coordinate system. The value for the parameter 'region' is
%             obtained by considering the previous % displacement and force, and the
%             maximum or minimum displacement obtained up to that moment.

```

```
%      umax: the maximum displacement obtained up to the time step, i+1. For the
%      elastic behavior of the system its value is obtained with respect to
%      Fmax and for the inelastic behavior it is equal to a displacement, which
%      is greater than the umax % obtained up to that time step, i. (cm)
%      umin: the minimum displacement obtained up to the time step, i+1. For the
%      elastic behavior of the system its value is obtained with respect to
%      Fmin and for the inelastic behavior it is equal to a displacement, which
%      is smaller than the umin % obtained up to that time step, i. (cm)
```

```
uf0=ui-(Fi)/ke;
```

```
if ui>0&Fi>0&uii>0&uii<ui&uii>uf0
```

```
    kii=ke;
```

```
    Fii=kii*uii+Fi-kii*ui;
```

```
    region=5;
```

```
end
```

```
if ui>0&Fi>0&uii>0&uii>ui&uii<=umax
```

```
    kii=(Fmax-Fi)/(umax-ui);
```

```
    kii=(floor(kii*10000))/10000;
```

```
    Fii=kii*uii+Fi-kii*ui;
```

```
    region=6;
```

```
end
```

```
if ui>0&Fi>0&uii>0&uii>umax
```

```
    [kii,Fii]=kiiFiipos(d,FF,uii);
```

```
    umax=uii;
```

```
    Fmax=Fii;
```

```
    region=7;
```

```
end
```

```
if ui>0&Fi>0&uii>0&uii<uf0
```

```
    kii=Fmin/(umin-uf0);
```

```
    kii=(floor(kii*10000))/10000;
```

```
    Fii=kii*uii+Fmin-kii*umin;
```

```
    region=8;
```

```
end
```

```

if ui>0&Fi>0&uii<0&uii>=umin
    kii=Fmin/(umin-uf0);
    kii=(floor(kii*10000))/10000;
    Fii=kii*uii+Fmin-kii*umin;
    region=9;
end
if ui>0&Fi>0&uii<0&uii<umin
    [kii,Fii]=kiiFiineg(d,FF,uii);
    umin=uii;
    Fmin=Fii;
    region=10;
end
if ui>0&Fi<0&uii>0&uii>uf0&uii<=umax
    kii=Fmax/(umax-uf0);
    kii=(floor(kii*10000))/10000;
    Fii=kii*uii+Fmax-kii*umax;
    region=11;
end
if ui>0&Fi<0&uii>0&uii>umax
    [kii,Fii]=kiiFiipos(d,FF,uii);
    umax=uii;
    Fmax=Fii;
    region=12;
end
if ui>0&Fi<0&uii>0&uii>ui&uii<uf0
    kii=ke;
    Fii=kii*uii+Fi-kii*ui;
    region=13;
end
if ui>0&Fi<0&uii>0&uii<ui
    kii=(Fmin-Fi)/(umin-ui);
    kii=(floor(kii*10000))/10000;
    Fii=kii*uii+Fi-kii*ui;

```

```

    region=14;
end
if ui>0&Fi<0&uii<0&uii>=umin
    kii=((Fmin-Fi)/(umin-ui));
    kii=(floor(kii*10000))/10000;
    Fii=kii*uii+Fmin-kii*umin;
    region=15;
end
if ui>0&Fi<0&uii<umin
    [kii,Fii]=kiiFiineg(d,FF,uii);
    umin=uii;
    Fmin=Fii;
    region=16;
end
if ui<0&Fi<0&uii>0&uii<=umax
    kii=Fmax/(umax-uf0);
    kii=(floor(kii*10000))/10000;
    Fii=kii*uii+Fmax-kii*umax;
    region=17;
end
if ui<0&Fi<0&uii>umax
    [kii,Fii]=kiiFiipos(d,FF,uii);
    umax=uii;
    Fmax=Fii;
    region=18;
end
if ui<0&Fi<0&uii>ui&uii<uf0
    kii=ke;
    Fii=kii*uii+Fi-kii*ui;
    region=19;
end
if ui<0&Fi<0&uii<ui&uii>=umin
    kii=(Fmin-Fi)/(umin-ui);

```

```

kii=(floor(kii*10000))/10000;
Fii=kii*uii+Fmin-kii*umin;
region=20;
end
if ui<0&Fi<0&uii<umin
[kii,Fii]=kiiFiineg(d,FF,uii);
umin=uii;
Fmin=Fii;
region=21;
end
if ui<0&Fi<0&uii<0&uii>ui&uii>uf0
kii=Fmax/(umax-uf0);
kii=(floor(kii*10000))/10000;
Fii=kii*uii+Fmax-kii*umax;
region=22;
end
if ui<0&Fi>0&uii>0&uii<=umax
kii=(Fmax-Fi)/(umax-ui);
kii=(floor(kii*10000))/10000;
Fii=kii*uii+Fmax-kii*umax;
region=23;
end
if ui<0&Fi>0&uii>umax
[kii,Fii]=kiiFiipos(d,FF,uii);
umax=uii;
Fmax=Fii;
region=24;
end
if ui<0&Fi>0&uii<ui&uii>=umin
kii=Fmin/(umin-uf0);
kii=(floor(kii*10000))/10000;
Fii=kii*uii+Fmin-kii*umin;
region=25;

```

```

end
if ui<0&Fi>0&uii<umin
    [kii,Fii]=kiiFiineg(d,FF,uii);
    umin=uii;
    Fmin=Fii;
    region=26;
end
if ui<0&Fi>0&uii<0&uii>ui
    kii=(Fmax-Fi)/(umax-ui);
    kii=(floor(kii*10000))/10000;
    Fii=kii*uii+Fmax-kii*umax;
    region=27;
end
if ui<0&Fi>0&uii<ui&uii>uf0
    kii=ke;
    Fii=kii*uii+Fi-kii*ui;
    region=28;
end
if uii==uf0
    kii=ke;
    Fii=0;
    region=29;
end

```

%%%

A.5. Functions Used in the Dynamic Analysis Code

beta: coefficient needed for the Newmark's Method. It indicates the method, which is going to be used. For the average acceleration method its value is 1/4 and for the linear acceleration method its value is 1/6.

- c*: the damping coefficient of the system.
- d*: the displacement vector taken from the backbone curve and the values are ordered from the minimum to the maximum value. The units are in (cm).
- deltatau*: it is the difference between the next displacement, $u(i+1)$, and the current displacement $u(i)$. In the main procedure, it corresponds to the step 2.4. (cm)
- deltadu*: it is the difference between the next velocity, $du(i+1)$, and the current velocity $du(i)$. In the main procedure, it corresponds to the step 2.5. (cm/sec)
- deltaddu*: it is the difference between the next acceleration, $ddu(i+1)$, and the current acceleration $ddu(i)$. In the main procedure, it corresponds to the step 2.6.(cm/sec²)
- deltap*: the difference between the next applied force to the current applied force.(kg.cm/sec²)
- deltapsapka*: a formula that in the main procedure it corresponds to step 2.1
- ddu*: the acceleration vector and the response for an earthquake motion. (cm/sec²)
- ddu(i)*: a acceleration value of the acceleration vector, ddu , for the current step, i . (cm/sec²)
- ddu(i+1)*: a acceleration value of the acceleration vector, ddu , for the current step, $i+1$. (cm/sec²)
- dt*: the time increment used for the ground acceleration.(sec).

- du*: the velocity vector and the response for an earthquake motion.
(cm/sec)
- du(i)*: a velocity value of the velocity vector, *du*, for the current step, *i*.
(cm/sec)
- du(i+1)*: a velocity value of the velocity vector, *du*, for the current step, *i+1*.
(cm/sec)
- F*: the force vector obtained with respect to the displacement vector, *u*,
and the stiffness vector, *k*. (kg.cm/sec²)
- F(i)*: a force value of the force vector, *F*, obtained from the current
displacement, *u(i)*. (kg.cm/sec²)
- F(i-1)*: a force value of the force vector, *F*, obtained from the previous
displacement, *u(i-1)*. (kg.cm/sec²)
- FF*: the force vector taken from the backbone curve and the values are
ordered with respect to the displacement vector, *d*. The units are in
(N).
- Fi*: a parameter which corresponds to the previous force, *F(i-1)*. This is
the input value in the function 'stiffnessi'. (kg.cm/sec²)
- Fii*: a parameter which corresponds to the current force, *F(i)*. This is the
output value in the functions 'stiffnesse', 'stiffnessi', 'kiiFiineg'
and 'kiiFiipos'. (kg.cm/sec²)

- flag:* a sign that indicates the systems behavior. If its value equals to 0 than the system behaves linearly and when its value equals to 1 than the system behaves nonlinearly.
- Fmax:* a force, which for the elastic behavior is equal to 75 percent of the maximum FF component and for the inelastic behavior is computed with respect to u_{max} . (kg.cm/sec²)
- Fmin:* a force, which for the elastic behavior is equal to 75 percent of the minimum FF component and for the inelastic behavior is computed with respect to u_{min} . (kg.cm/sec²)
- gama:* coefficient needed for the Newmark's Method. For average and linear acceleration methods its value is 1/2.
- i:* the time step.
- k:* the stiffness vector of the system, obtained from the stiffness of the each time step, i. ((kg.cm/sec²)/cm)
- k(i):* the stiffness for the current displacement, $u(i)$.
- ke:* the stiffness for the elastic behavior. ((kg.cm/sec²)/cm)
- kii:* the stiffness which corresponds to the current stiffness, $k(i)$.
- ksapka:* a formula that in the main procedure corresponds to the step 2.3.((kg.cm/sec²)/cm)
- ksi:* the damping ratio of the system.
- lengthd:* the number of the values in the displacement vector, d.

- lengthp:* the number of the values in the applied force vector, p , minus 1.
- m:* the mass of the system. (kg)
- p:* the applied force vector. It is obtained by multiplying the mass of the system with the ground acceleration. (kg.cm/sec²)
- region:* a sign which gives the position of the point, described by the last displacement and force obtained with respect to that displacement, in the coordinate system. The value for the parameter 'region' is obtained by considering the previous displacement and force, and the maximum or minimum displacement obtained up to that moment.
- regionv:* a vector whose components are the 'region' values obtained for each time step, i .
- u:* the displacement vector and the response for an earthquake motion. (cm)
- u(i):* a displacement value of the displacement vector, u , for the current step, i . (cm)
- u(i-1):* a displacement value of the displacement vector, u , for the current step, $i-1$. (cm)
- u(i+1):* a displacement value of the displacement vector, u , for the current step, $i+1$. (cm)
- ug:* the ground acceleration (cm/sec²)
- ui:* a parameter which corresponds to the previous displacement, $u(i-1)$. This is the input value in the function 'stiffness'. (cm)

- uii:** a parameter which corresponds to the current displacement, $u(i)$. This is the input value in the functions 'stiffnesse', 'stiffnessi', 'kiiFiineg' and 'kiiFiipos'. (cm)
- umax:** the maximum displacement obtained up to the time step, $i+1$. For the elastic behavior of the system its value is obtained with respect to F_{max} and for the inelastic behavior it is equal to a displacement, which is greater than the $umax$ obtained up to that time step, i . (cm)
- umin:** the minimum displacement obtained up to the time step, $i+1$. For the elastic behavior of the system its value is obtained with respect to F_{min} and for the inelastic behavior it is equal to a displacement, which is smaller than the $umin$ obtained up to that time step, i . (cm)
- uumax:** the maximum displacement vector which values are equal to $umax$ that is evaluated for each time step, i . (cm)
- uumin:** the minimum displacement vector which values are equal to $umin$ that is evaluated for each time step, i . (cm)
- t:** the time vector in seconds.

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