SEISMIC PERFORMANCE EVALUATION AND ANALYSIS OF STEEL STRUCTURES WITH SEMI-RIGID CONNECTIONS

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE
OF
MASTER OF SCIENCE
IN
CIVIL ENGINEERING

JUNE 2004

Approval of the Graduate School of Natural and Applied Sciences

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ABSTRACT

SEISMIC PERFORMANCE EVALUATION AND ANALYSIS OF STEEL STRUCTURES WITH SEMI-RIGID CONNECTIONS

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June 2004, 159 Pages

At the design stage, column-beam connections of steel structures are assumed as fully rigid or as hinges, and the design is completed with these assumptions. On the other hand, in practice, steel column-beam connections show neither fully rigid nor fully hinge behaviour, and the characteristic behaviour of the connections lies between these two special cases. Performing realistic calculation of these forces and knowing the behaviour of structures close to reality will decrease life and goods losses to the minimum level in a probable of earthquake to be encountered in the future.

In this study, seismic performance of 2-D steel frames were evaluated by Capacity Spectrum Method proposed in the ATC 40 document published in 1996. A new computer program was developed in order to define all geometric and loading data and to perform nonlinear analysis of rigid and semi rigid steel frames for which the performances will be evaluated.

In case studies, 3-Floor Steel Frames that have different bay numbers were investigated in various forms according to the rigid and different semi rigid connection types. In addition, the performances these frames for various seismic regions and soil conditions were compared.

According to the results, it was observed that semi rigidly connected frames are under the effect of smaller ground acceleration have greater displacement values. As a consequence of this ductile and energy dissipative response, it was seen that the stresses in the members of frame become considerably small, relative to the stresses in the rigid frames'. Furthermore, the performances of semi-rigid frames can be affected negatively beyond such a low rigidity.

Consequently, the most convenient design should be made according to the seismic and soil region where the structure to be constructed by performing the necessary studies on the connection details in order to achieve desired performance, serviceability and optimum member criteria.

Keywords: Semi-Rigid Steel Connections, 2-D Nonlinear Analysis, Pushover Analysis, Capacity Spectrum Method, Seismic Performance

ÖZ

YARI-RİJİT BAĞLANTILI ÇELİK ÇERÇEVELERİN ANALİZLERİ VE SİSMİK PERFORMANLARININ DEĞERLENDİRİLMESİ

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Haziran 2004, 159 Sayfa

Tasarım aşamasında çelik yapıların kolon-kiriş bağlantıları, tam olarak rijit veya mafsal formda varsayılıp, tasarım bu varsayımların ışığında sonuçlandırılmaktadır. Diğer taraftan; pratikte, çelik kolon-kiriş bağlantıları, ne tam olarak rijit ne de tam olarak mafsal davranışı göstermekte olup, bağlantıların karakteristik davranışları gerçekte bu iki özel durum arasında yer almaktadır. Yapının deprem yüklerine karşı gerçeğe yakın davranışının bilinmesi, binanın ileride karşılaşabileceği depremler sonucu oluşabilecek can ve mal kayıplarını minimum seviyeye indirecektir.

Bu çalışmada 2 boyutlu çelik çerçevelerin sismik performansları, 1996 yılında yayımlanan ATC-40 dokümanında önerilen Kapasite Spektrum Modeli ile değerlendirilmiştir. Performans hesabı yapılacak olan rijit ve yarı-rijit çerçevenin, tüm geometric ve yükleme bilgilerini tanımlamak, doğrusal olmayan analizleri gerçekleştirmek için yeni bir bilgisayar programı geliştirilmiştir.

Bu çalışmada 3 katlı farklı açıklıklara sahip çelik çerçeveler rijit ve farklı yarı-rijit bağlantı tiplerine gore değişik formlarda incelenip, farklı deprem ve zemin bölgeleri için sismik performanları karşılaştırılmıştır.

Yapılan çalışmalar sonucunda, yarı-rijit bağlantılı çerçevelerin daha büyük deplasman değerleriyle beraber daha düşük ivme etkisi altında kaldığı görülmüştür. Bu sünek ve enerji emici davranış neticesinde, çerçeve elemanlarında oluşan gerilmelerin, rijit bağlantılı çerçeve elemanlarına gore ciddi şekilde azaldığı görülmüştür. Bununla beraber, belirli bir noktadan sonraki çok düşük rijitliklere sahip çerçeve performanslarının olumsuz yönde etkilendiği kaydedilmiştir.

Sonuç olarak, uygun performans, servis sınırları ve optimum eleman kesitleri için bağlantılar üzerine gerekli çalışmalar yapılarak, yapının inşa edileceği deprem bölgesi ve zemin koşullarına göre en uygun tasarım gerçekleştirilmelidir.

Anahtar Kelimeler: Yarı-Rijit Çelik Bağlantılar, 2 Boyutlu Doğrusal Olmayan Analiz, Statik İtme Analizi, Kapasite Spektrum Metodu, Sismik Performans

ACKNOWLEDGEMENTS

I would like to offer my thanks to Assoc.Prof. Dr. Can BALKAYA for his support for the completion of this study.

Also, I would like to express my appreciation to my mother, father and sister, supporting me throughout my life.

And finally, my special thanks to all my teachers and faculty members from primary school till my university graduation, who encourage me during my education.

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LIST OF SYMBOLS AND ABBREVIATIONS

2D	Two Dimensional
3D	Three Dimensional
ADRS	Acceleration-Displacement Response Spectra
C_i	Curve Fitting Constant in Polynomial Function
d	Overall Depth of the Beam
d_a	Overall Depth of Web Angle
E	Modulus of Elasticity
F_{fi}	Fictitious Load Acting at i'th Storey in the Determination of Fundamental Natural Vibration Period
FEMA	Federal Emergency Management Agency
FR	Fully Restrained
g	Distance from the heel of Web-Angle to the center of the fastener hole
g_2	Distance Between Two Plastic Hinges
H_i	Height of i'th Storey of Building Measured from the Top Foundation Level
K	Standardization Constant in Polynomial Function
l_t	Length of the Top Angle
l_s	Length of the Seat Angle
M	Bending Moment
M_0	Plastic Bending Moment
M_{pt}	Plastic Moment in the Top Angle
M_{py}	Plastic Moment Capacity of Web Angle
PF_1	First Mode Modal Participation Factor
PR	Partially Restrained
S_a	Spectral Acceleration

S_d	Spectral Displacement
SE	Secant stiffness of the connection
SR	Semi-Rigid
SRFP	Semi-Rigid Frame Program
T	Period of the Structure
t_s	Thickness of Seat Angle
t_t	Thickness of Top Angle
t_w	Thickness of Web Angle
V_0	Plastic Shear Force
V_{pt}	Plastic Shear Force in the Vertical Leg
V_{py}	Plastic Shear Force per Unit Height of a Single Web Angle
w_i	Weight of i'th Storey of Building by Considering Live Load Participation Factor
α_1	First Mode Modal Mass Coefficient
ε	Tolerance Limit
ΔM	Change in Moment During a Load Increment
Δ_{roof}	Value for Roof Displacement
$\Delta\theta$	Change in Relative Rotation During the Load Increment
θ_r	Relative Connection Relative Deformation
ϕ_{roof1}	Value of the Fundamental Mode Shape at the Roof

CHAPTER 1

INTRODUCTION

1.1 Purpose and Objectives

Our country is on a very active earthquake zone since it is located at a geological region where important fault lines pass through. Earthquakes occurred in history, where Marmara, Düzce, Bolvadin and Bingöl earthquakes are the recent ones, caused so many life and goods losses. Those earthquakes again confirmed that the seismic design performance of the structure should be the first priority design parameter. Knowing the forces affecting the structures and consequently the behaviour of the structures against possible earthquakes should be the fundamental point on the design and construction process of the structures. In other words, the performance of the structure should be known and investigated before construction.

The main aim of this study is to investigate the general behaviour of planar steel frames with semi-rigid connections, to compute performance of this kind of frames and to investigate the effects of different semi-rigid connection types on the performance of the frame in a most realistic way. As a necessity of this purpose, firstly the general behaviour of planar frames with semi-rigid connections under specific loads needs to be studied. The reason for the selection of steel structures to work on is, the proofs of the worldwide investigations that steel structures' performance are superior than reinforced concrete structures' and to point out the advantages of steel structures in Turkey, where the usage of steel as a construction material is uncommon. Also, the reason for the selection of semi-rigid connection type instead of fixed or hinge connection types is to achieve more realistic analysis results.

This study is generally established on a computer program that can make linear and non-linear analysis of 2D steel frames. And most of the work of the master thesis had been done for this program.

In this study, first of all; the analyses of frames with semi rigid connections are performed. For the fixed geometry and loading, the frame is analysed for rigid, rigid with P-Δ and semi-rigid cases. For this step, the program developed in Fortran language is used in order to get the results that are to be checked for the results in the references. For the next step, the performances of 2D steel frames with rigid and semi-rigid connections are evaluated according to different types of connections, different type of soil conditions, and different types of earthquake regions. Also, the above- mentioned parameters are changed for the fixed frame geometry. Results of these effects of parameters on the performance of the structures are investigated according to the ADRS method described in ATC-40, 1996.

In the next chapter, Chapter 2, general information about semi-rigid connections and commonly used semi-rigid connection types are given.

Chapter 3 tells about the derivation of the computer program that can make above-mentioned analysis. In that study, the analysis of frames for rigid, rigid with P-Δ and semi-rigid cases are told in detail.

In chapter 4, where performance issue comes into the scene, assumptions, derivations and computation of performance according to ATC 40 and FEMA 273 are studied.

In chapter 5, case studies are performed in order to accomplish the whole study.

And finally, in chapter 6, conclusion part, the results of the whole study are concluded and possible future studies are discussed.

CHAPTER 2

SEMI-RIGID CONNECTIONS

2-1 General Information About Semi-Rigid Connections

The steel frame performs as an integrated structure deriving its strength from the individual elements, which are properly located to transmit loads throughout the structure. The connections' behaviour has an effect on the frame performance, because they are the basic elements and integrated part of a steel frame.

In 1994, the Load and Resistance Factor Design (LRFD) specification of the American Institute of Steel Construction (AISC) categorizes two basic types of steel frame construction as follows:

1. Fully restrained (FR) type construction, with the assumption of full continuity and adequate rigidity of beam to column connections, which can retain the initial angles between intersection members.
2. Partially restrained (PR) type construction, with the assumption that a connection possesses moment capacity in between complete fixity and the pin connection (Dhillon and Majid, 1990)

Conventional analysis and design of steel framework are performed using the assumption that the connections are either fully rigid or ideally pinned. No relative rotation of connection occurs and the end moment of the beam is completely transferred to the column in fully rigid connections. Contrary to this type, the pinned connection implies that no restraint for rotation of connection exists and the connection moment is always zero. The available design techniques are based on these idealized conditions. But, in reality most connections are semi-rigid and fall under PR type construction type. Figure 2-1 shows the comparison between connection types.

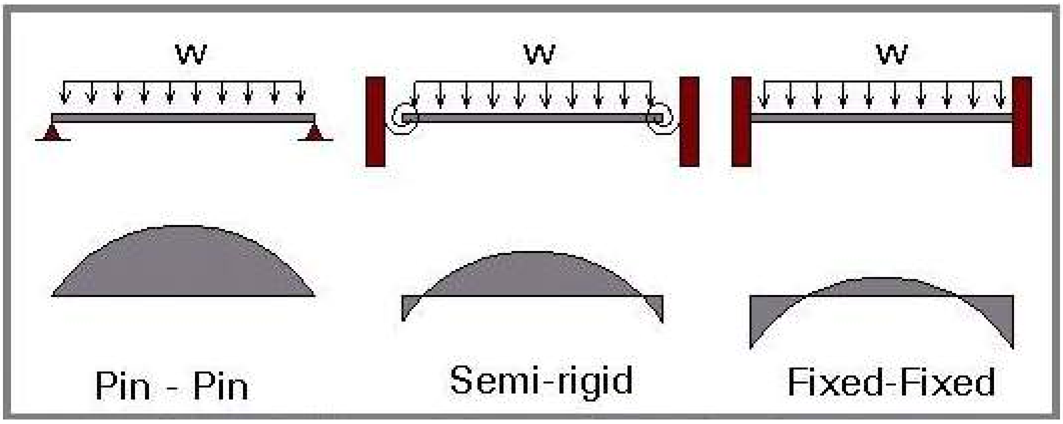


Figure 2-1: Comparison of Semi-Rigid Connections vs. Pinned and Fixed Connections (McGuire, 1995)

Most of the connections that fasten beam to column using angles, plates, welds, and bolts are deformable and perform a non-linear behaviour between fully fixed and perfectly pinned conditions. So it is more reasonable to classify all connections under the classification of semi-rigid, whereas rigid and pinned conditions being special cases. Exact determination of the relative restraint of beam to column connection is important for both strength and serviceability of structural frames. Overestimating the connection restraint can result in underestimating lateral sway and underestimating the connection restraint can lead to underestimating forces developed in the beams and columns. Both conditions can affect structural stability.

The primary distortion of a steel beam to column connection is due to in-plane bending moment, which yields in a rotational deformation. A connection under an applied moment rotates through angle θ_r, which is the angle between beam and column from their original position (Figure 2-2). This moment also causes a destabilization effect on frame stability since additional drift will occur as a result of the decrease in effective stiffness of the members which the connections are attached to. Increased frame drift triggers the P-Δ effect and the overall stability of the frame will be affected.

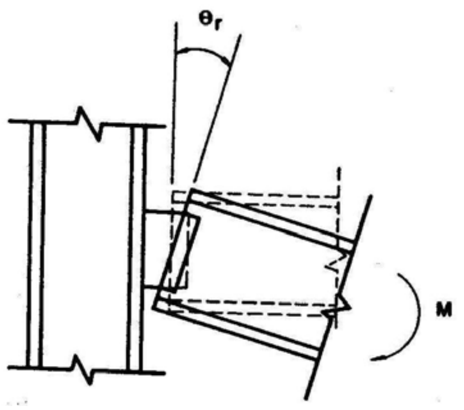


Figure 2-2: Rotation of Connections under Applied Moment (Chen and Toma, 1994)

Figure 2-3 shows the relative moment rotation behaviour of a variety of commonly used semi-rigid connections (Dhillon and O'Malley, 1999). All types of connections exhibit non-linear moment-rotation behaviour that falls between the two extreme cases of ideally pinned and fully rigid connections. The single web-angle connection represents a flexible joint and the T-stub connection represents a rather rigid joint. The moment-rotation curves of all types of connections are non-linear over the entire range of loading. It is difficult to analyze these connections by a hard procedure. As a result of this, the analyses of connection behaviour used in design are approximate with drastic simplifications.

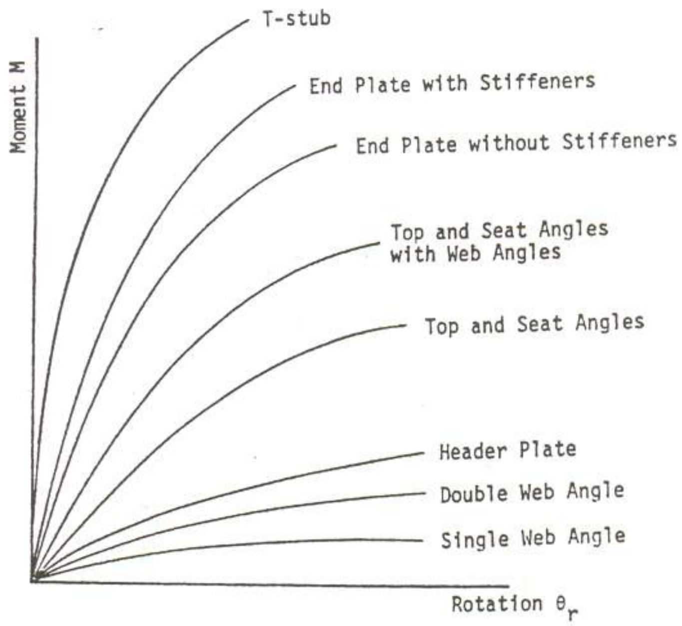


Figure 2-3: Moment-Rotation Behaviour of Commonly Used Semi-Rigid Connections (Dhillon and O'Malley, 1999)

While two types of construction, fully restrained (FR) construction and partially restrained (PR) construction are defined, basic guidelines for the design of PR construction are not given, because it is very difficult to evaluate the actual restraint of semi-rigid connections used in engineering practice. Also, for the majority of designers, design and analysis of frames with PR construction still seems impractical when compared to relative simplicity of traditional FR construction (Bhatti and Hingtgen, 1995). There are several interrelated obstacles that prevent today's designer of steel structures from embracing a semi rigid connection philosophy. A general listing of these concerns are as follows (Semi-Rigid Connections in Steel Frames, 1992):

- *Utilization Classification Concerns*: Traditional rules put frustrating constraints on the designer when semi rigid connections are contemplated.

- *Moment-Rotational Model*: The problem is that this information is scattered worldwide, and is normally in a mathematical formulation that is not comforting to the practicing engineer.

- *Serviceability and Stability Concerns*: Semi rigid connection parts depart from elastic strain limits and this inelasticity gives a soft connection effect, which will leave a residual deflection in the connected beam.

Besides all these obstacles, there are still advantages of semi rigid connections.

- First, the essential inelastic behaviour of the connecting parts prevents high stress points in the connected members themselves, thus allowing more slender cross sections and the elimination of stiffeners, and reducing high stress concentration complications in ductility sensitive designs.

- Second, for inertia-oriented loads such as earthquakes, preliminary research indicates that the energy absorption of inelastic connections actually keeps excessive lateral drift within reason

- Third, the use of plastic design in steel actually represents a higher order of optimisation in its process of developing mechanism failure modes. Most semi rigid frameworks reach their useful limit at a serviceability limit rather than a strength limit. This implies a retrofit benefit in flexibly connected structures after an accident.

- Fourth, the extra technical nature of semi rigid analysis automatically draws the designer more intimately into the design process. This closer engagement is bound to create better designs as a result of this stimulation.

- Fifth, on a philosophical level it is apparent that neither pinned nor rigid connections are actually obtained in real structures. It would seem that as a profession we need to continually drive ourselves closer to reality. Even approximate estimates of frame flexibility are closer to truth than the assumed ideals of nil or full restraint in the connections.

2.2 The Most Common Types of Semi-Rigid Connections

2.2.1 Single Web-Angle Connections and Single Plate Connections

Single web-angle connection consists of an angle either bolted or welded to both the column and the beam web as shown in Figure 2-4. Different from this, single plate connections use the plate instead of angle (Figure 2-5). This connection type requires less material than a single-web angle connection. In the design of this kind of connection, generally, the single web-angle connections have moment rigidity equal to about one-half of the web double web-angle connections (Chen and Toma, 1994)

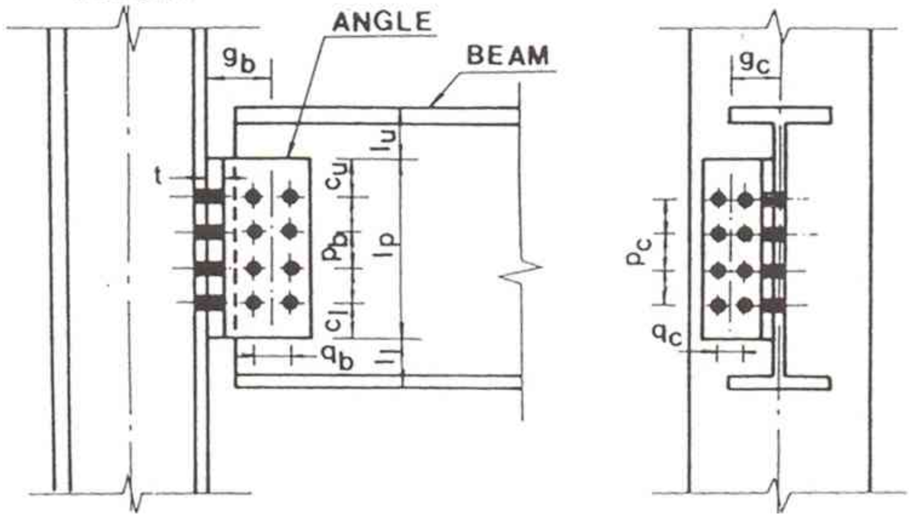


Figure 2-4 : Single Web-Angle Connection

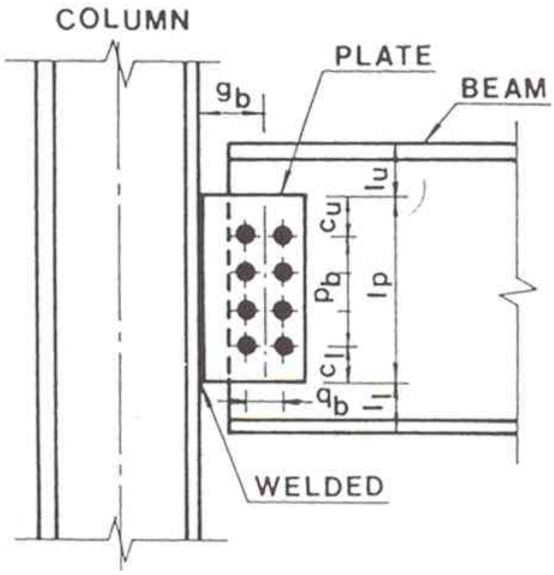


Figure 2-5 : Single Plate Connection

2.2.2 Double Web-Angle Connections

Double web-angle connections consist of two angles either bolted or riveted to both the column and the beam web as shown in Figure 2-6. Today, the double web-angle connections with high-strength bolts are used popularly. The connection rigidity of this type is stiffer than that of the single web-angle and single plate connections.

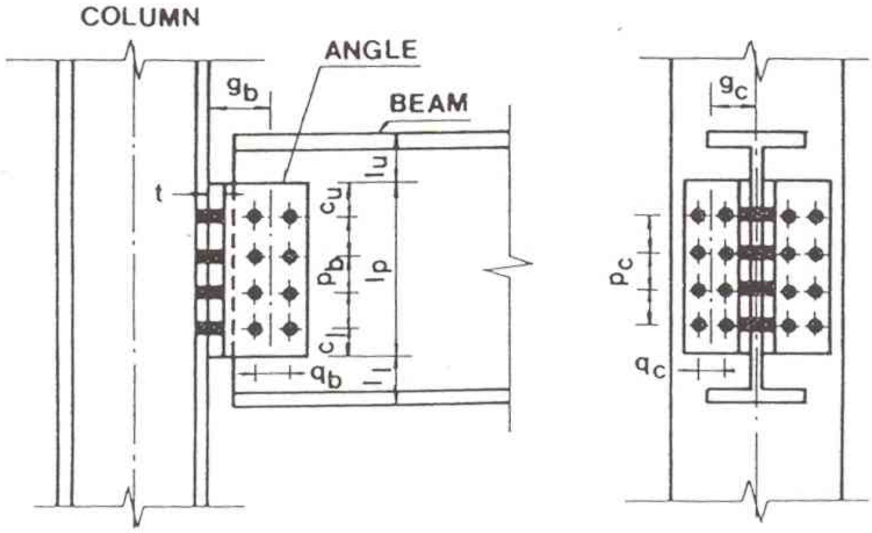


Figure 2-6: Double Web-Angle Connection

2.2.3 Top and Seat Angle Connections with Double Web Angle

A typical top and seat angle connection with double web angle is shown in Figure 2-7. Top and seat angle connection with or without double web angle are semi-rigid connections. Double web angle is used to improve the connection restraint characteristics of top and seat angle connections. In order to evaluate inherent ductility offered by the flexural deformation capacities of both the flange and the web angles in the legs attached to the column under static and earthquake loading (Chen and Toma, 1994).

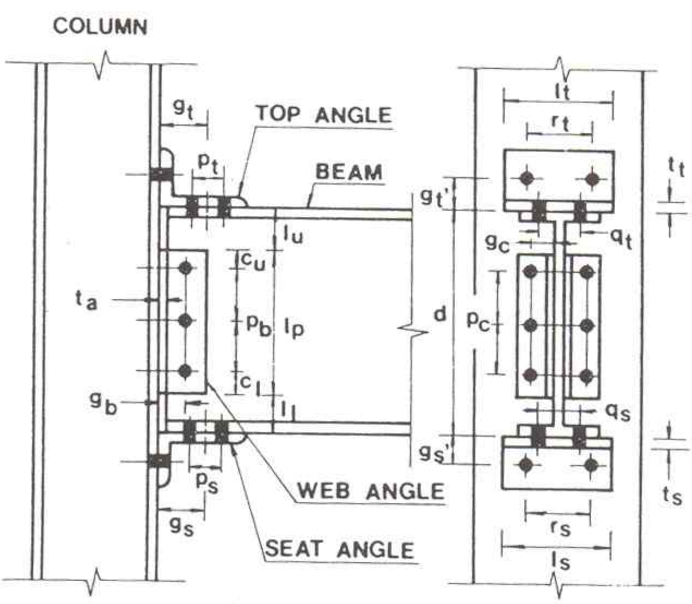


Figure 2-7: Top and Seat Angle Connections with Double Web Angle

2.2.3 Top and Seat Angle Connections

A typical top and seat angle connection is shown in Figure 2-8. In this type, the seat angle transfers only vertical reaction and should not give significant restraining moment on the end of the beam. On the other hand, the top angle is merely for lateral stability and is not considered to carry any gravity loads. However, according to experimental results, these connections will resist some end moment of the beam (Chen and Toma, 1994).

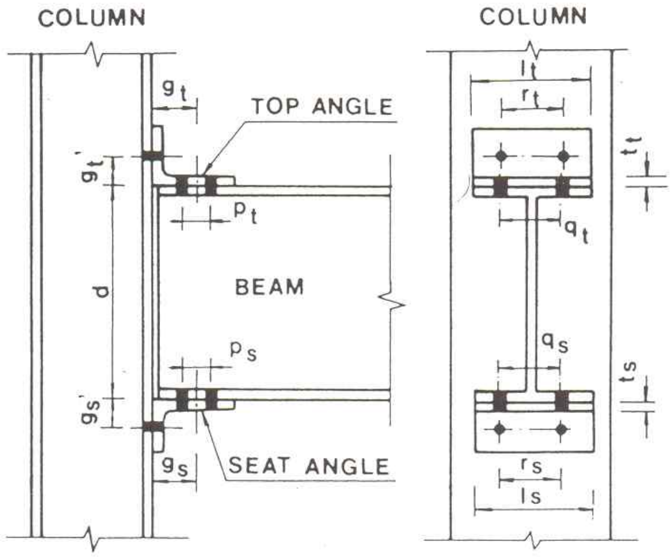


Figure 2-8: Top and Seat Angle Connections

2.2.4 Extended End – Plate Connections and Flush End-Plate Connections

In general, the end-plate connections are welded to they beam end along both the flanges and the web in the fabricator's shop and bolted to the column in the field. The end-plate connection has been used extensively since the late 1960s. The extended end-plate connections are classified into two types – either on the tension side only or on both the tension and the compression sides, as shown in Figure 2-9. A typical flush end-plate connection is shown in Figure 2-10. Since both extended and flush end-plate connections are considered as type FR rather than type PR connections, they have often been used as means of transferring beam end moment to the column).

The extended end-plate connection on the tension side only is commonly used. The extended end-plate connection on both sides is preferred when the frame structure is subjected to moment reversal, as during severe earthquake loading. While the flush end-plate connection is weaker than the extended end-plate connection, this connection type is often used in roof details. The behaviour of the end-plate connections depends on whether the column flange near the connection is stiffened or not. The stiffeners of the column flanges act to prevent the flexural deformation of column flange, thereby influencing the behavior of the plate and fasteners (Semi-Rigid Connections in Steel Frames, 1992) .

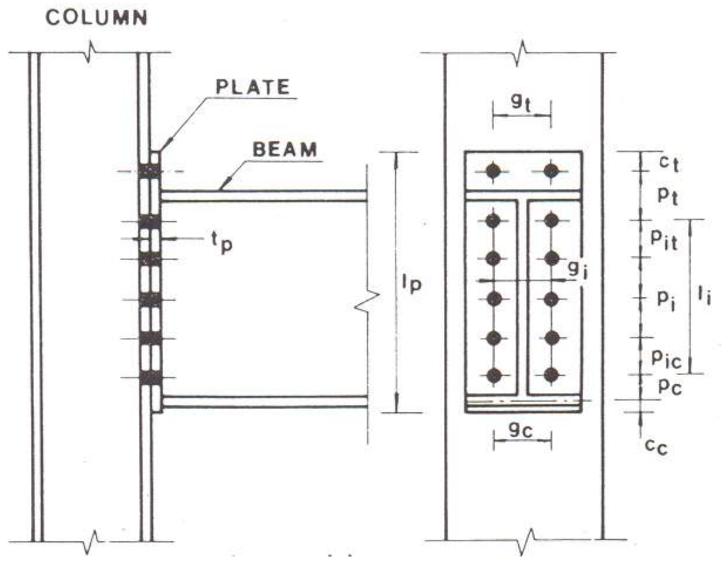


Figure 2-9: Extended End-Plate Connections (Tension Side Only)

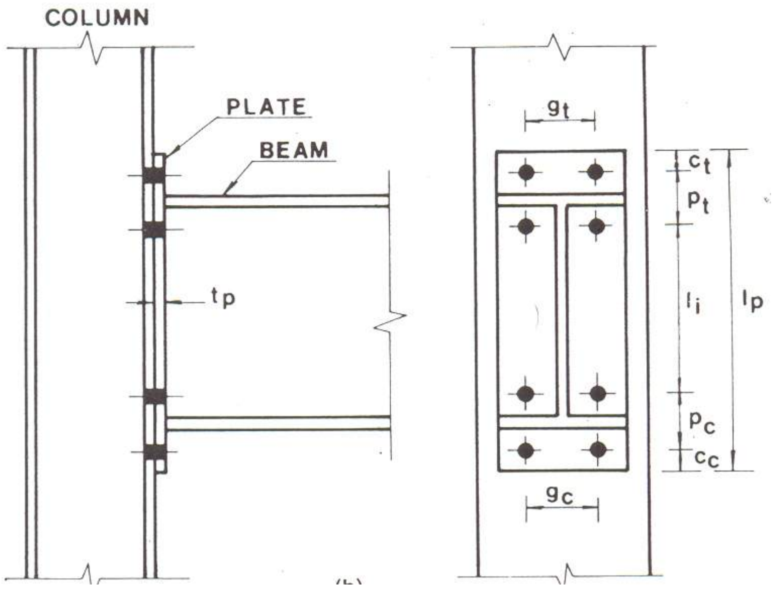


Figure 2-10: Extended End-Plate Connections (Tension and Compression Sides)

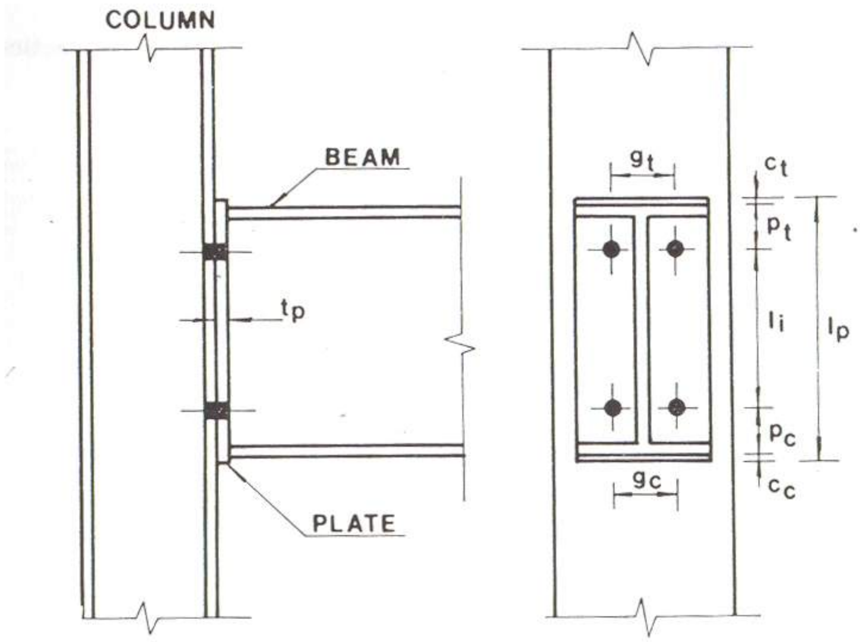


Figure 2-11: Typical Flush End-Plate Connections

2.2.5 Header Plate Connections

A header plate connection consists of an end plate whose length is less than the depth of the beam, welded to the beam web, and bolted to the column, as shown in Fig.A.9. The moment- rotation characteristics of these connections are similar to those of double web-angle connections. Accordingly, a header plate connection is used mainly to transfer the reaction of the beam to the column.

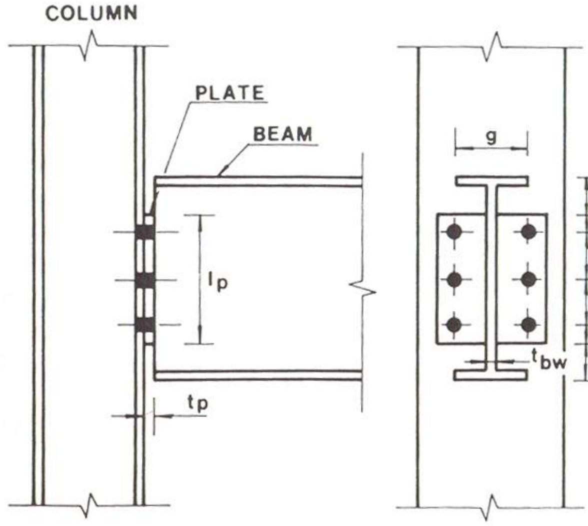


Figure 2-12: Typical Header Plate Connections

Although the above most commonly used types of semi-rigid connections are introduced; top and seat angle connections, web angle connections and top and seat angle connections with double angles will be examined in this study.

2.3 Modelling of Semi-Rigid Connections

Since the 1930s, methods of modelling moment rotation curves of semi-rigid connections have been developed along with experimental studies. There are several representative models proposed in the literature to represent the moment-rotation behaviour. Linear Model, Power model, Exponential model, Cubic B-Spline model and polynomial model are the most popular ones.

In this study, the semi-rigid connections are modelled by using polynomial model.

2.3.1 Polynomial Model

A popular model for structural analysis is the polynomial function proposed by Frye and Morris. The Frye-Morris model was developed based on a procedure by Sommer. They used the method of least squares to determine the constants of the polynomial. The model has the form

$$\theta_r = C_1(KM) + C_2(KM)^3 + C_3(KM)^5$$

where C_i are curve-fitting constants and K is the standardization factor (a dimensionless factor whose value depends on the size parameters for the particular connection considered). The size parameters in the equations for standardization constants in Table 2.1, and eight types of connections, are given in Figure 2-13.

Table 2.1: Standardized Connection Constants (Dhillon and O'Malley, 1999)

Connection Types	Curve-Fitting Constants	Standardization Constants
Single Web-Angle	$C_1=4.28\text{x}10^{-3}$ $C_2=1.45\text{x}10^{-9}$ $C_3=1.51\text{x}10^{-16}$	$\kappa = d_a^{-2.4} t_a^{-1.81} g^{0.15}$
Double Web-Angle	$C_1=3.66\text{x}10^{-4}$ $C_2=1.15\text{x}10^{-6}$ $C_3=4.57\text{x}10^{-8}$	$\kappa = d_a^{-2.4} t_a^{-1.81} g^{0.15}$
Top and Seat Angle with Double Web Angle	$C_1=2.23\text{x}10^{-5}$ $C_2=1.85\text{x}10^{-8}$ $C_3=3.19\text{x}10^{-12}$	$\kappa = d^{-1.287} t^{-1.128} t_c^{-0.415} l_a^{-0.694} g^{1.35}$
Top and Seat Angle without Double Web Angle	$C_1=8.46\text{x}10^{-4}$ $C_2=1.01\text{x}10^{-4}$ $C_3=1.24\text{x}10^{-8}$	$\kappa = d^{-1.5} t^{-0.5} l_a^{-0.7} d_b^{-1.5}$
End Plate without Column Stiffeners	$C_1=1.83\text{x}10^{-3}$ $C_2=1.04\text{x}10^{-4}$ $C_3=1.24\text{x}10^{-8}$	$\kappa = d_g^{-2.4} t_p^{-0.4} d_b^{-1.5}$
End Plate with Column Stiffeners	$C_1=1.79\text{x}10^{-3}$ $C_2=1.76\text{x}10^{-4}$ $C_3=2.04\text{x}10^{-4}$	$\kappa = d_g^{-2.4} t_p^{-0.6}$
T-Stub	$C_1=2.10\text{x}10^{-4}$ $C_2=6.20\text{x}10^{-6}$ $C_3=-7.60\text{x}10^{-9}$	$\kappa = d^{-1.5} t^{-0.5} l_t^{-0.7} d_b^{-1.1}$
Header Plate	$C_1=5.10\text{x}10^{-5}$ $C_2=6.20\text{x}10^{-10}$ $C_3=2.40\text{x}10^{-13}$	$\kappa = d_p^{-2.3} t_p^{-1.6} t_w^{-0.5} g^{1.6}$

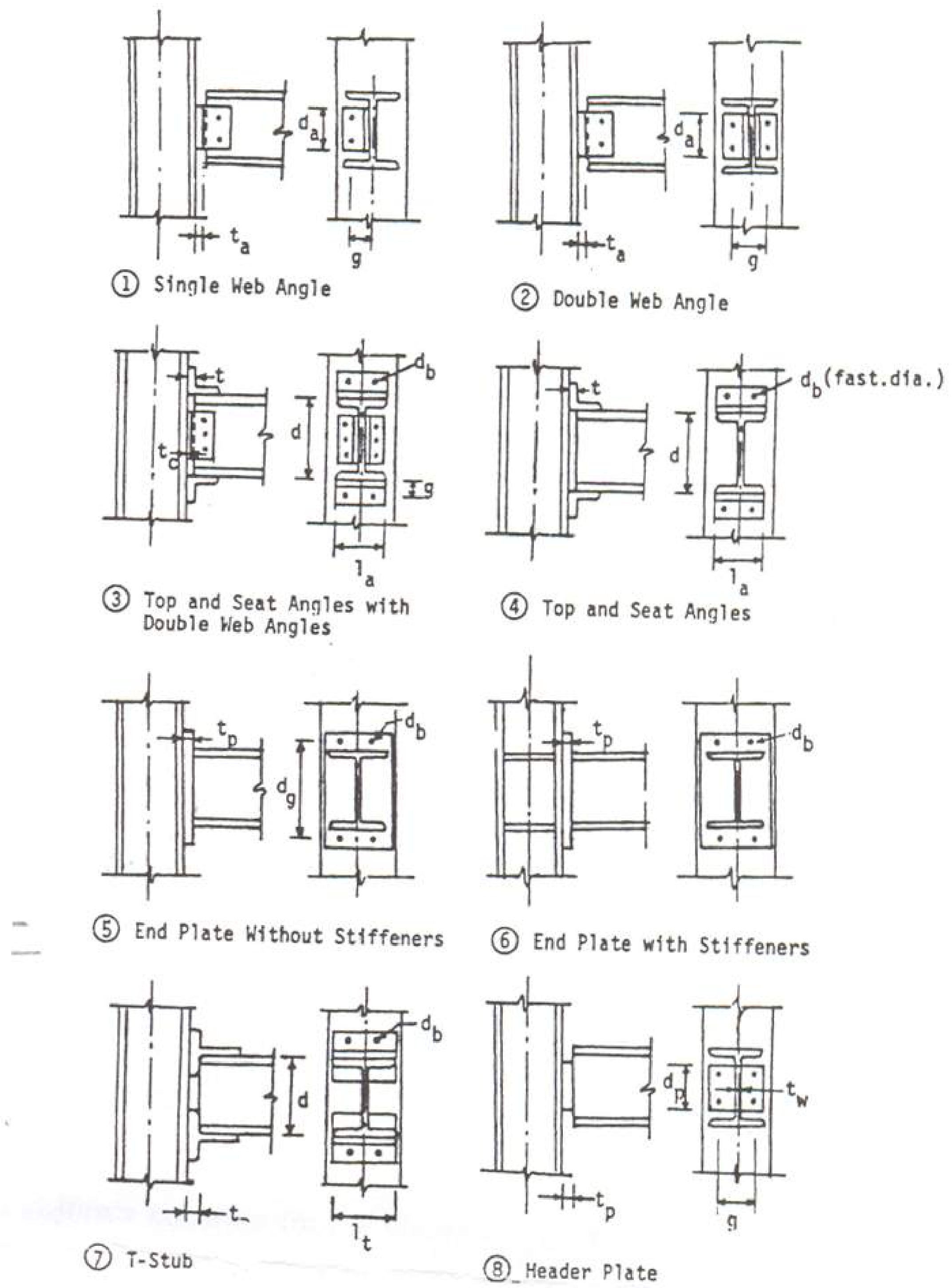


Figure 2-13: Parameters of 8 Commonly Used Type Semi Rigid Connections (Dhillon and O'Malley, 1999)

CHAPTER 3

SEMI-RIGID PLANAR STEEL FRAME ANALYSIS BY A DEVELOPED COMPUTER PROGRAM

3.1 General

In this study, a computer program written in Fortran language is developed. Actually the program consists of two parts; for general frame analysis and performance calculations. This general program is designed to satisfy the multi purpose of the user to analyze multistory frames with orthogonal beam and column members subjected to point, distributed, self and earthquake loads. The second part that is to be used for the performance calculation is a particular case of the above general program. The program can analyze frames with either rigid or semi-rigid connections.

First, the program is general since any traditional rigid frame can be analyzed linearly. On the other hand, the program is particular one, since it is designed to analyze semi rigid frames with different types of connections by considering detailed parameters thereof. It is an iterative and interactive analysis procedure, which accounts for the geometric nonlinearity of the frame members and the nonlinear moment-rotation behavior of commonly used semi-rigid connection types. Also the P-Δ effect can be taken into consideration upon user's request.

The base of first floor columns can either be pinned or totally fixed. The other joints can be assigned as rigid or semi-rigid. All assigned loads are converted to point loads-horizontal, vertical or moment- and assigned at joints for the analysis. Different types of forces can be assigned on a single joint by following the loop in the program. Wind loads are not taken into consideration directly, but upon request of the user, their equivalent forces can be applied on the relevant joints.

The following assumptions are taken into consideration throughout the program:

1. Materials are homogeneous, isotropic and elastic.
2. Structural members are elastically stable.
3. Unless otherwise stated, all joints are rigid.
4. The structure is two-dimensional (All members are in the same plane).
5. All loading is in the plane of the structure.
6. Modulus of elasticity (E) is the same for all members.
7. All members are located on their strong axes.
8. Element lengths are taken as the distance between the axes. (The widths and heights of the elements are not taken into consideration when computing member lengths.)
9. 3 degrees of freedom are considered for each joint.
10. Constraints of base columns are assigned as either rigid or hinge.

3.2 Analytical Method

3.2.1. Analytic Method for Rigid Frame Analysis

Rigid frames are analyzed by conventional matrix structural analysis. This analysis assumes that the deformations are relatively small, and the equilibrium equations can be formulated with respect to initial geometry. For this linear elastic analysis,

$$[P]=[K][d]$$

where [P] = Applied force matrix
[K] = Global stiffness matrix
[d] = Displacement matrix

The first order frame element stiffness matrix $[k]_e$ that form the general stiffness matrix is as follows:

$$[k_e]_i = \begin{bmatrix} \frac{A_X E}{L} & 0 & 0 & \frac{-A_X E}{L} & 0 & 0 \\ 0 & \frac{12EI}{L^3} & \frac{6EI}{L^2} & 0 & \frac{-12EI}{L^3} & \frac{12EI}{L^3} \\ 0 & \frac{12EI}{L^3} & \frac{4EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{2EI}{L} \\ \frac{-A_X E}{L} & 0 & 0 & \frac{A_X E}{L} & 0 & 0 \\ 0 & \frac{-12EI}{L^3} & \frac{-6EI}{L^2} & 0 & \frac{12EI}{L^3} & \frac{-6EI}{L^2} \\ 0 & \frac{12EI}{L^3} & \frac{2EI}{L} & 0 & \frac{-6EI}{L^2} & \frac{4EI}{L} \end{bmatrix}$$

First Order Frame Element Stiffness Matrix

3.2.1. Analytic Method for Rigid Frame Analysis with P-Δ Effect

Like linear analysis, well-known P-Δ analysis technique is applied for rigid frame analysis with P-Δ effects.

The linear elastic analysis assumes that the deformations are relatively small and, the equilibrium equations can be formulated with respect to initial geometry. But on the other hand, increasing applied loads cause significant changes in the geometry of the structure. For this reason, the equilibrium and compatibility equations become nonlinear and the resulting stiffness matrix contains terms that are functions of axial forces and deformations. The stiffness matrix to represent this behavior must include the effect of geometric nonlinearity.

When compared with the columns, the beams have small axial forces. For this reason, it’s better to take the effect of change in the geometry in column members. While the member under compression becomes weaker, the member under tension becomes stronger. In order to take this issue into consideration; in columns, the stiffness matrix of these elements incorporating P-Δ effects are expressed as the sum of two matrices, $[k]_e$ and $[k]_g$. $[k]_e$ is the above-mentioned first order frame element stiffness matrix, and $[k]_g$ is called the geometric stiffness matrix. For a column element *i*, the nonlinear stiffness matrix is written as:

$$[k_i]=[k_e]_i+[k_g]_i$$

where

$$[k_e]_i=\begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{6}{5} & \frac{L}{10} & 0 & \frac{-6}{5} & \frac{L}{10} \\ 0 & \frac{L}{10} & \frac{2L^2}{15} & 0 & \frac{-L}{10} & \frac{-L^2}{30} \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & \frac{-6}{5} & \frac{-L}{10} & 0 & \frac{6}{5} & \frac{-L}{10} \\ 0 & \frac{L}{10} & \frac{-L^2}{30} & 0 & \frac{-L}{10} & \frac{2L^2}{15} \end{bmatrix}$$

Member Geometric Stiffness Matrix

3.2.3. Analytic Method for Semi-Rigid Frame Analysis with P-Δ Effect

As mentioned in the preceding paragraphs, the program can analyze the frame by considering all connections are rigid and analyze the same frame with different types of semi rigid connections. By the help of this function, the user can observe variations in the behavior of the frame due to connection type under same loading.

In PR construction, the moments rotation capability of a given of connection must be established by analytical or experimental means. Most experiments have shown that the connection moment rotation characteristics are nonlinear over the entire range of loading for almost all types of connections. Since the connection flexibility and second order geometric nonlinearity are highly related and simultaneously affect each other, the analysis of the steel frame as a whole must consider a nonlinear approach. (Dhillon and O'Malley, 1999).

In addition, for members with flexible connections at the ends, the stiffness matrix must be modified to account for the effect of connection flexibility. In general, the columns in frames are continuous and do not possess any internal flexible connections. The beams have semi-rigid connections, but have small axial forces with negligible effect of geometric nonlinearity. For simplification, two types of elements will be used in the design of plane frames with semi rigid connections; Beam –Column Element and Semi-Rigid Beam Elements follows (Dhillon and O'Malley, 1999).

3.2.3.1 Beam-Column Element

A Beam Column Element is a plane frame element modified to include geometric nonlinearity; in other words, columns of the frame.

The stiffness matrix of a beam-column element incorporating P-Δ effects can be expressed as the sum of two matrices,

3.2.3.2 Semi Rigid Beam Element

A Semi-Rigid Beam Element is a plane-frame element modified to include end connection flexibility.

Although connections constitute a small portion of the steel frame, their effect is significant to the overall frame performance. While the restraining effect of the connection was recognized, there was no means of incorporating it into the analysis methods because it was not known numerically how much restraint was present. In order to have a better understanding the behavior and strength of the steel frame, more attention was directed towards studying the steel connections.

The connection behavior is primarily shown by the moment-rotation curve, which relates the moment transmitted by the connection to the relative rotation of the intersecting members. The slope of this curve at any particular moment value represents the stiffness of the connection (Figure 3-1). The nonlinear nature of the curves indicates that the stiffness decreases as the loading increases for all semi-rigid connections.

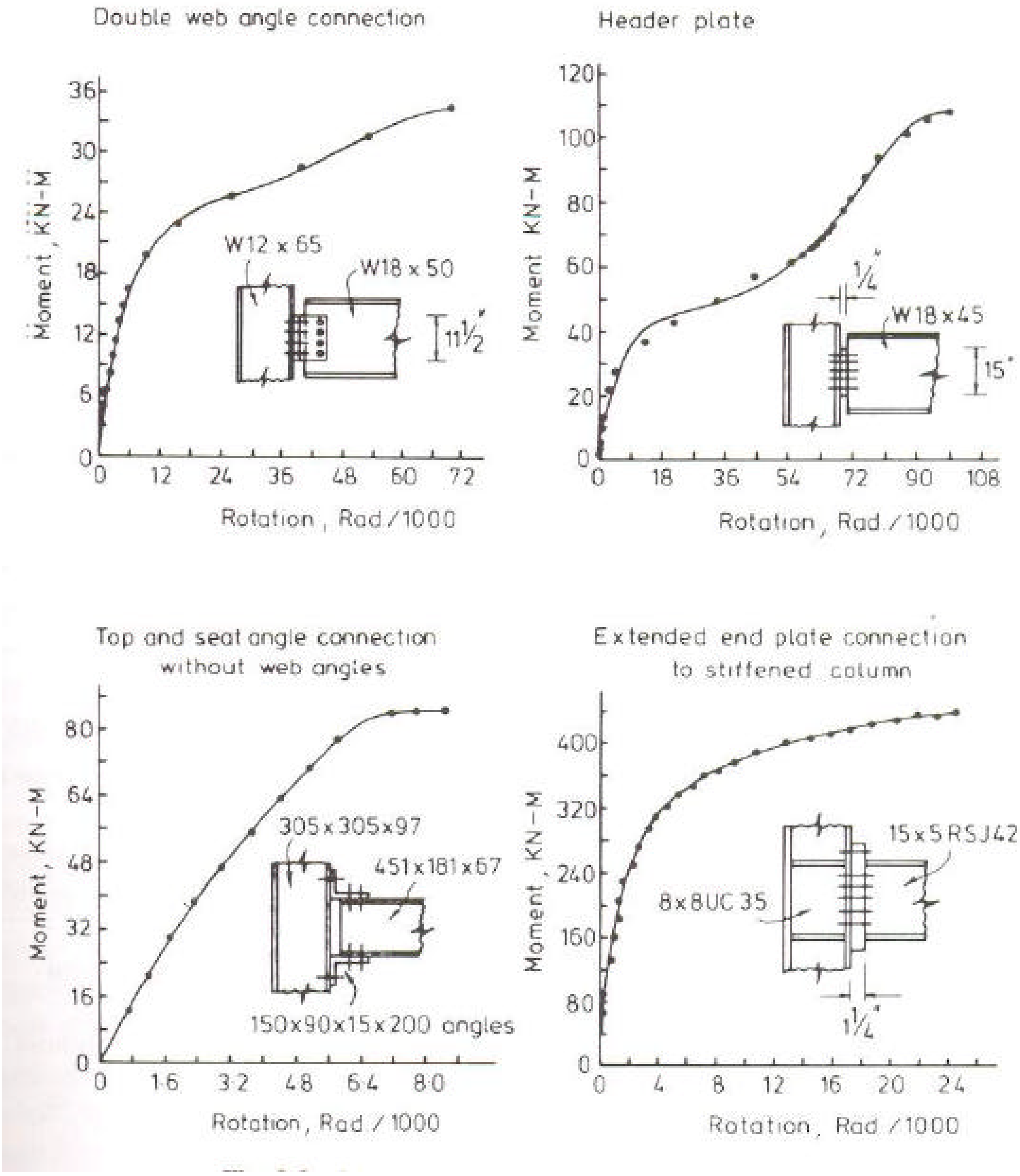


Figure 3-1: Moment-Rotation Curves of Sample Connections

From the available moment-rotation relationship, the stiffness of the connection can be derived as the secant or the tangent stiffness approach, which is sensitive to local variations. In this study the secant stiffness method is used.

The connection element is represented by a rotational spring with stiffness equal to the change in relative connection rotation for the load increment under consideration. The instantaneous stiffness of these connections is computed as follows;

$$SE = \frac{\Delta M}{\Delta q}$$

where

SE = Secant stiffness of the connection

ΔM = Change in moment during a load increment

Δq = Change in relative rotation during the load increment

The effect of flexible connection at the ends of a beam-column element can be incorporated in the analysis method by modifying the standard elastic stiffness matrix of the beam column element. Since the secant stiffness method is adopted and applied to the connection curve in an iterative solution procedure, the stiffness of the connection can be modeled as the constant of the linear spring.

For the modification of member stiffness matrix there are different techniques are investigated. Although the form and the parameters of the techniques are different from each other, they both yield the same result. For this study Monforton and Wu technique is used.

According to this method, two dimensionless parameters, γ_j and γ_k, designated as fixity factors, are introduced

$$g_j = \frac{L}{L + \frac{EI}{(SE)_j}} \qquad g_k = \frac{L}{L + \frac{EI}{(SE)_k}}$$

where the subscripts *j* and *k* refer to the ends of the beam-column element.

The modified stiffness matrix $[k_f]_m$ is the outcome of the multiplication of the beam-column stiffness matrix $[k_e]_m$ by the correction matrix $[C]$

$$[k_e]_m = [k_f]_m + [C]$$

where

$$[C] = \begin{bmatrix} C_{jj} & C_{jk} \\ C_{kj} & C_{kk} \end{bmatrix}$$

and

$$[C_{jj}] = [C_{kj}] = \begin{bmatrix} 1 & 0 & 0 \\ 0 & \dfrac{4g_k - 2g_j + g_j g_k}{4 - g_j g_k} & -2L\left(\dfrac{g_j(1-g_k)}{4 - g_j g_k}\right) \\ 0 & \dfrac{6}{L}\left(\dfrac{g_j - g_k}{4 - g_j g_k}\right) & \dfrac{3g_j(2-g_k)}{4 - g_j g_k} \end{bmatrix}$$

$$[C_{jk}] = [C_{kk}] = \begin{bmatrix} 1 & 0 & 0 \\ 0 & \dfrac{4g_j - 2g_k + g_j g_k}{4 - g_j g_k} & -2L\left(\dfrac{g_k(1-g_j)}{4 - g_j g_k}\right) \\ 0 & \dfrac{6}{L}\left(\dfrac{g_j - g_k}{4 - g_j g_k}\right) & \dfrac{3g_k(2-g_j)}{4 - g_j g_k} \end{bmatrix}$$

Using the equations above, the modified stiffness matrix from eq(33) reduces to the following form:

$$[\overrightarrow{k_f}]_m = \begin{bmatrix} \dfrac{A_X E}{L} & 0 & 0 & \dfrac{-A_X E}{L} & 0 & 0 \\ 0 & \dfrac{12EI}{L^3}\left(\dfrac{A}{H}\right) & \dfrac{6EI}{L^2}\left(\dfrac{B}{H}\right) & 0 & \dfrac{-12EI}{L^3}\left(\dfrac{A}{H}\right) & \dfrac{12EI}{L^3}\left(\dfrac{G}{H}\right) \\ 0 & \dfrac{12EI}{L^3}\left(\dfrac{B}{H}\right) & \dfrac{4EI}{L}\left(\dfrac{C}{H}\right) & 0 & \dfrac{-6EI}{L^2}\left(\dfrac{B}{H}\right) & \dfrac{2EI}{L}\left(\dfrac{F}{H}\right) \\ \dfrac{-A_X E}{L} & 0 & 0 & \dfrac{A_X E}{L} & 0 & 0 \\ 0 & \dfrac{-12EI}{L^3}\left(\dfrac{A}{H}\right) & \dfrac{-6EI}{L^2}\left(\dfrac{B}{H}\right) & 0 & \dfrac{12EI}{L^3}\left(\dfrac{A}{H}\right) & \dfrac{-6EI}{L^2}\left(\dfrac{G}{H}\right) \\ 0 & \dfrac{12EI}{L^3}\left(\dfrac{G}{H}\right) & \dfrac{2EI}{L}\left(\dfrac{F}{H}\right) & 0 & \dfrac{-6EI}{L^2}\left(\dfrac{G}{H}\right) & \dfrac{4EI}{L}\left(\dfrac{D}{H}\right) \end{bmatrix}$$

where

$A = g_j + g_k + g_j g_k$

$B = g_j\,(2 + g_k\,)$

$C = 3g_j$

$D = 3g_k$

$F = 3g_j g_k$

$G = g_k\,(2 + g_j)$

$H = 4 - g_j g_k$

Adding the geometric stiffness and the modified stiffness matrix includes the geometric nonlinearity effect.

The beam-column and semi-rigid beam stiffness matrices are in local or member coordinates. It is necessary to transfer them to global coordinates before the formation of the overall stiffness matrix The resulting stiffness matrix for member i is transferred to the structure coordinates

$$[k]_i = [T]^T[k]_m[T]$$

where

$$[k]_m = [k_f]_m + [k_g]_m$$

$$[T] = \begin{bmatrix} c & s & 0 & 0 & 0 & 0 \\ -s & c & 0 & 0 & 0 & 0 \\ 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & c & s & 0 \\ 0 & 0 & 0 & -s & c & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 \end{bmatrix}$$

in which c and s represent cosine and sine of angle between the member axis and the horizontal axis

3.3 Iterative Numerical Analysis

The nonlinear response of a PR frame to increasing loads is a result of the nonlinear nature of the beam-column connections and secondary effects due to large deflections. The beam-column stiffness matrix includes non linear coefficients which depend on connection stiffness and member axial forces as well as finite changes in the geometry. As a result, the entire structure stiffness matrix formed by superimposing nonlinear beam-column stiffness matrices has a nonlinear nature and an iterative solution process is indispensable.

The applied loads are divided into a number of small-load increments for which the structural stiffness equations are written in the incremental form

$$\{\Delta A\} = [S] + \{\Delta D\}$$

where

$[S] = Ó\ [k]_i$, Structure stiffness matrix
$\{\Delta A\}$= Incremental load vector
$\{\Delta D\}$= Incremental displacement vector

The incremental equations are iteratively solved through a sequence of steps. The secant approach is used for evaluating the connection stiffness. For each load increment, the stiffness matrix is calculated at the start of each iterative cycle. It requires the calculation of the secant stiffness at the beginning of each cycle, and changing the reference geometry and member forces using the information from the previous cycle. Convergence is obtained when the difference between joint displacements of two consecutive cycles reaches a specific tolerance.

Modification of the connection secant stiffness at each cycle for the first load increment is shown in Figure 3-2. For the first cycle, the connections are assumed as fixed. The member end moment, *M*, is obtained from this analysis and the corresponding connection rotation is obtained using Equation 3-1. The connection secant stiffness is calculated as follows:

$$(SE)_1 = \frac{M_1 - 0}{q_1 - 0} \qquad \text{(Eq: 3-1)}$$

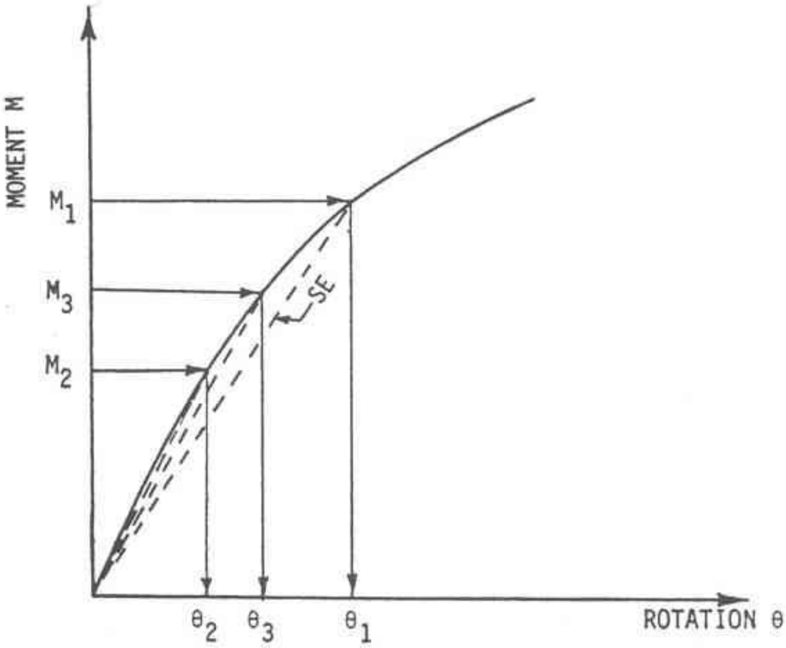


Figure 3-2: Connection Secant Stiffness Modification Through One Increment

The initial secant connection stiffness, $(SE)_1$, is used to modify the beam column stiffness coefficients and the solution of the stiffness equation is performed using the updated matrices. A new connection moment M_2 is obtained and the same procedure is repeated until convergence is achieved.

A convergent solution of a load increment forms an initial estimate for the next iteration and the iterative solution procedure continues until all load increments are considered. The convergent solutions for all load increments are accumulated to obtain the total nonlinear response.

The connection secant stiffness corresponding to all load increments is shown in Figure 3-3.

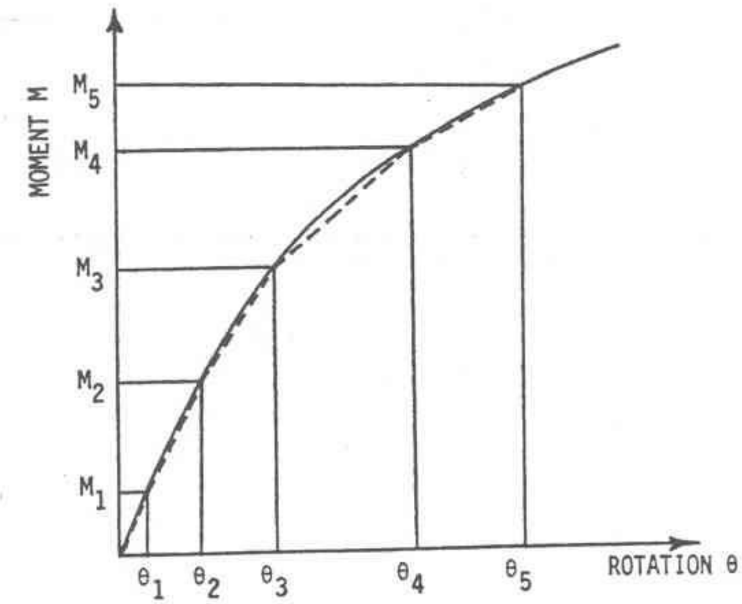


Figure 3-3: Connection Secant Stiffness Through Load Increment

The procedure for the analysis of frames with semi-rigid connections by the developed computer program summarized below;

1. Divide applied loads into a series of small load increments.
2. Calculate the load increment vector $\{\Delta A\}$.
3. Calculate the modified beam column stiffness matrix $[k]_I$, in the structure coordinate system.
4. Assemble the modified beam column stiffness matrices to form the overall structure stiffness matrix [S].
5. Solve the incremental stiffness Equation 3-1 to obtain incremental displacement vector $\{\Delta D\}$.
6. Determine member end actions.
7. Check convergence. If convergence is achieved, go to step 11.
8. Calculate connection secant stiffness.
9. Update the nonlinear terms in the stiffness matrices using the latest connection stiffness and the latest member axial force and geometry.

10. Repeat steps 3 through 9 until convergence is obtained.
11. At convergence, add the incremental deformations and incremental member end actions to the corresponding accumulated values.
12. If the increments applied equal the total load increments, analysis is completed. If not go to step 3, and take the convergent solution for the initial estimation of the nonlinear terms in the next load increment.

3.4 Frame Specifications

The member stiffness calculation is based on the length of the member (l), the cross sectional area (A), moment of inertia (I), modulus of elasticity (E) and end connection types. All these parameters are assigned by the user and inputted in the input data in terms of ton and meter.

3.5 Connection Information

In addition to the fixed and pinned end conditions, eight types of connections can be assigned in the program. The base constraints of the first story columns are assigned as either fixed or pinned. Each of these connection types has an identification number and the condition size parameters are given in Table 3-1. The values of the connection size parameters are expressed in inches and can be obtained corresponding to the dimensions of the column, beam, angles and fastener spacing and diameter as shown in Figure 2-13.

3.6 Applied Loads

Point loads, uniform loads, death and earthquake loads can be assigned by the user. Any other kind of loads can be converted these types of loads and applied to the frame. At the end of assigning the loads, the total loading matrix $[P]$ is formed. In the iterative analysis process, the load increment amount is defined by the user and this incremental increase in loads shows the non linearity effect more visible. The more load incremental amount, the less number of cycles.

Table 3-1: Connection Parameters

Connection No:	Connection Type	Connection Parameters:
1	Single Web-Angle	d_a, t_a, g
2	Double Web-Angle	d_a, t_a, g
3	Top and Seat Angle with Double Web Angle	d, t, t_c, l_a, g
4	Top and Seat Angle without Double Web Angle	$d, t,, l_a, d_b,$
5	End Plate without Column Stiffeners	d_g, t_p, d_b
6	End Plate with Column Stiffeners	d_g, t_p
7	T-Stub	d, t, l_t, d_b
8	Header Plate	d_p, t_p, t_w, g

3.6.1 Point Loads

There are three types of point loads can be applied to the joints. They can be assigned to x, y and z-axis corresponding horizontal, vertical loads and moment respectively. The number and the magnitude of point loads can be adjusted by following the loop in the program.

3.6.2 Distributed Loads

Applied distributed loads are applied as point loads by converting them into the conventional fix end forces. Their conversion is as follows:

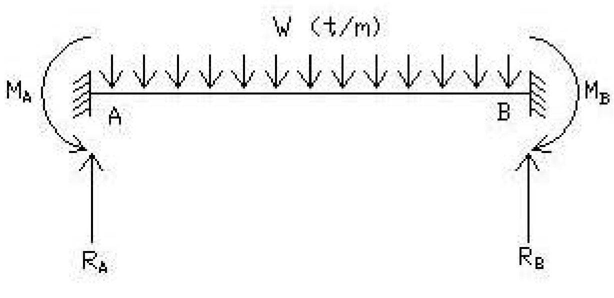


Figure 3-4: Member Fixed-End Forces

M_A= (-)W x L^2 / 12

M_B=W x L^2 / 12

R_A=(-)W x L / 2

R_B=(-)W x L / 2

3.7 Convergence Criteria

As mentioned before, an iterative solution procedure is required in order to take nonlinear nature of the overall frame into consideration. The analysis is performed through a number of cycles and iterations through which the nonlinear terms of the stiffness matrix are updated using the latest information obtained from the previous cycle of iteration.

Each iteration considers one load increment and after a convergent solution is obtained, the next iteration of load increment is applied. Convergence criteria works by comparing all displacements obtained in the current cycle with those obtained in the previous cycle. If the difference in the relative displacement for all degrees of freedom between the previous and current cycles is less than a specific tolerance, convergence is assumed to have occurred. The convergence criterion used in the program is as follows:

$$\frac{\left|D_{j,i} - D_{j-1,i}\right|}{\left|D_{j,i}\right|} \leq e$$

where

i= Degree of freedom identification number

j= Cycle number

D = Joint displacements

e = Tolerance limit (0.0001)

3.8 Example for Checking of the Program Execution

In order to verify the accuracy of the program, a sample frame taken from Bhatti and Hingtgen (1995) is used (Figure 3-5). Originally the sample question is in terms of feet and kips and the loading and specification of the frame are as follows:

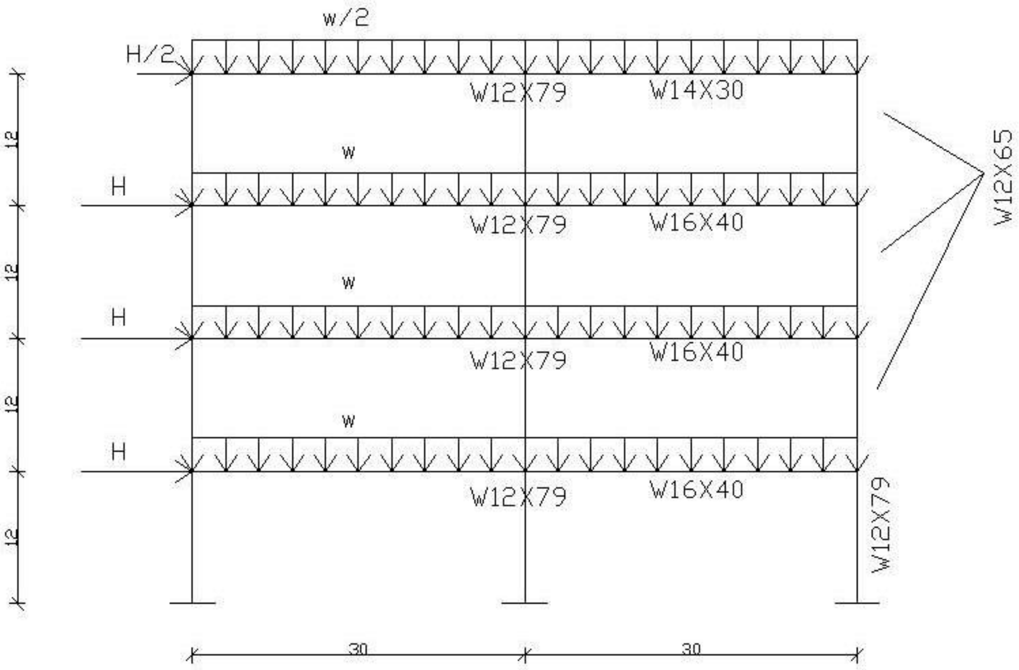


Figure 3-5: Sample Frame

Number of Floors	: 4
Number of Bays	: 2
Floor Height	: 12 feets (3,658 m)
Bay Width	: 30 feets (9.144 m)
Area and Inertia for W12x79	: 0,015m^2, 0,0002755m^4
Area and Inertia for W12x65	: 0,0123m^2, 0,0002219m^4
Area and Inertia for W16x40	: 0,007613m^2, 0,0002156m^4
Area and Inertia for W14x30	: 0,00571m^2, 0,0001211m^4
Lateral Load (H)	: 7,0 kips (3,175 t)
Distributed Load (W)	: 0,15 k/in (2,678 t/m)
Connection Type	: Jenkins Bolted Extended End Plate
Connection Type Number	: 6
Connection Parameters	: d = 19 in., t = 0,7875 in.

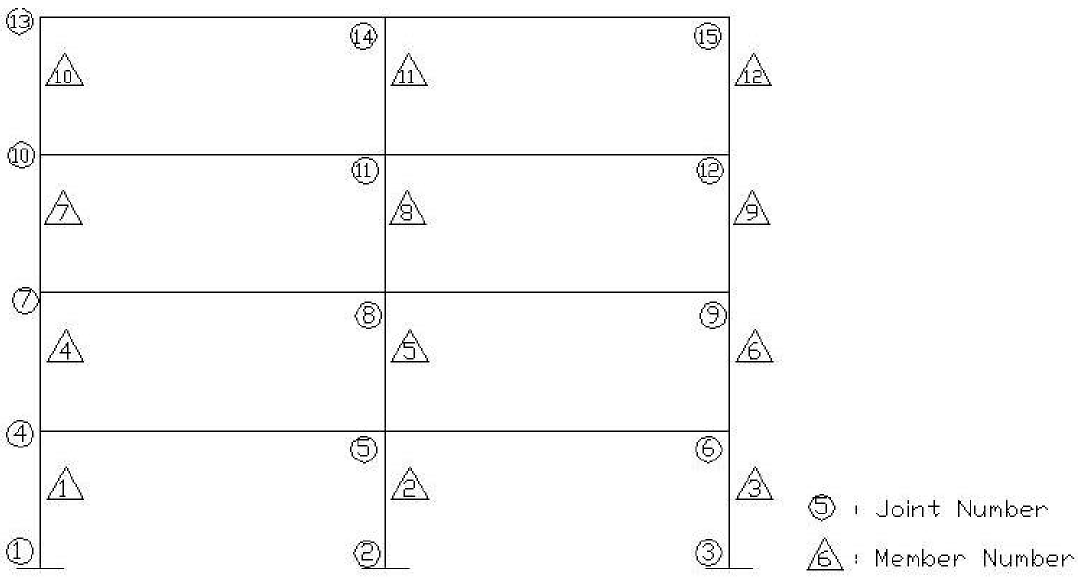


Figure 3-6: Frame Element and Node Numbers

In this verification example, the frame is analyzed assuming rigid and flexible connections, with and without P-Delta effects. For the analysis E was assumed to be 20.389.020 t/m^2 and total number of iterations is selected as 20. In other words, the overall load is applied to the frame in 20 steps. Detailed input data for this frame can be found in the Appendix A.

3.8.2 Comparison of Results

Lateral displacements and absolute bending moments are tabulated in Table 3-2 and 3-3 respectively for the comparison with Flex Frame and King & Chen methods (Bhatti and Hingtgen, 1995). The results obtained from the developed computer program are indicated as “SFRP”.

Table 3-2: Comparison of Lateral Displacements of Sample Frame

Node #	Rigid Connection SFRP (cm)	Rigid Connection with P-Ä Effect (cm)			Semi-Rigid Connection with P-Ä Effect (cm)		
		SRFP	Flex Frame	King & Chen	SRFP	Flex Frame	King & Chen
4	0,664	0,687	0,683	0,685	0,792	0,772	1,016
7	1,656	1,718	1,684	1,676	2,053	1,958	2,718
10	2,360	2,447	2,466	2,387	2,996	2,835	4,089
13	2,782	2,879	2,817	2,819	3,576	3,373	4,953

According to the results given in Table 3-2, it’s seen that for the rigid connection with P-Ä analysis; the values of SRFP are slightly greater than the others’. This is basically due to assigning the mid-columns of 2nd, 3rd and 4th floors as W12x65 instead of W12x79, which yields in a decrease in the stiffness.

In the Flex Frame Analysis, the connection stiffness is assumed to remain constant at its initial calculated value. In SRFP the connection stiffness is initially taken as rigid just for the 1st iteration. However, the nonlinear change in the connection stiffness is taken into consideration starting with the 2nd iteration. Consequently, the decrease in the stiffness is faced with earlier, which yields in greater sway values.

Table 3-3: Comparison of Absolute Maximum Bending Moments

Elem. #	**Rigid Connection SFRP (t.m)**	**Rigid Connection with P-Ä Effect (t.m)**			**Semi-Rigid Connection with P-Ä Effect (t.m)**		
		SRFP	**Flex Frame**	**King & Chen**	**SRFP**	**Flex Frame**	**King & Chen**
1	5,837	6,108	6,152	6,152	6,864	7,131	9,712
2	10,905	11,196	11,025	11,036	11,944	11,705	13,479
3	13,508	13,802	13,848	13,848	14,820	14,447	16,094
4	5,177	5,014	5,242	5,242	5,169	4,654	3,352
5	6,932	7,214	7,569	7,557	6,896	7,477	6,866
6	12,660	12,499	12,695	12,684	12,942	12,258	12,027
7	7,177	7,156	7,085	7,085	7,611	6,808	6,440
8	5,009	5,174	5,467	5,449	5,347	5,714	6,244
9	11,855	11,964	11,855	11,855	12,412	11,578	11,474
10	8,170	8,185	8,167	8,143	8,635	7,915	8,122
11	2,186	2,238	2,304	2,304	2,563	2,592	3,606
12	9,430	9,457	9,447	9,424	9,784	9,355	9,746

CHAPTER 4

SEISMIC PERFORMANCE EVALUATION

4.1 Introduction

The analysis method should be selected carefully, since that analysis gives crucial information about the structure under loads. A good design should respect the behaviour of the structure as close as possible to the actual physical response in order not to cause uneconomical over-design and underestimating the forces, which results in collapse of the structure.

At the beginning of 20^{th} century, most of the steel building design had been based on the use of linear analysis methods. Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, it cannot predict failure mechanisms and account for the redistribution of forces during progressive yielding. The use of inelastic procedures for design and evaluation is attempts to help engineers better understand how structures behave when subjected major earthquakes, where the elastic capacity of structures is exceeded.

Recently, the direction of building design in seismic regions has been towards Performance Based Seismic Design. The purpose of this design method is to accurately predict the performance of a building during any intensity of earthquake ground motion that may occur at the building site during the design life of the building.

In most of the analysis, the effects of semi-rigid connections are not taken into consideration. However, these effects considerably affect the response and seismic performance of the steel frame.

Design and evaluation procedure following the Performance Based Seismic Design Methodology enables the structural engineer to quantify the probability that a building design satisfies specified seismic performance objectives. Also, as can be seen in the preceding chapter, a good performance based analysis can result in optimum frame sections. Two other key terms, demand and capacity, come into scene with the matter of performance.

In short demand is a representation of the earthquake ground motion. This motion produces complex horizontal displacement patterns in structures that may vary with time. For a given structure and ground motion, the displacement demand is an estimate of the maximum expected response of the structure during the ground motion (ATC-40, 1996)

On the other hand, capacity is a representation of structure's ability to resist seismic demand. The overall capacity of a structure depends on the strength and deformation capacities of the individual components of the structure. In order to determine capacities beyond the elastic limits some form of nonlinear analysis, such as Pushover Procedure, is required (ATC-40, 1996). The detail of this procedure is given in the following pages but in short this procedure uses series of sequential elastic analysis each contributes to reflect the nonlinear response of the structure.

The performance is dependent on the manner that the capacity is able to handle the demand. In other words, the structure must have the capacity to resist the demands of the earthquake such that the performance of the structure is compatible with the objectives of the design. Once a capacity curve and demand displacement are defined, a performance check can be done. This check verifies that structural and nonstructural components are not damaged beyond the acceptable limits of the performance objective for the forces and displacements implied by the displacement demand.

Nonlinear analysis methods are the fundamental tools for accurately evaluating building performance objectives. As stated earlier, nonlinear static "pushover" analysis methods are accepted as convenient methods forestimating building response quantities and predicting building performance in a Performance

Based Seismic Design. The documents related with Performance Based Seismic Design Methodology are ATC-40 and FEMA 273. ATC-40 was published in 1996 by Applied Technology Council (ATC), as a report entitled "Seismic EvaIuation of Retrofit of Concrete Buildings" (ATC, 1996) and FEMA 273 was published in 1997 by Federal Emergency Management Agency, as a report entitled “NEHRP Guidelines for the Seismic Rehabilitation of Buildings”.

4.2 Performance Levels

Both ATC-40 and FEMA 273 propose different performance levels for the seismic rehabilitation of a building structures. Different performance levels according to FEMA 273 are described as follows:

Immediate Occupancy Performance Level (S-1): Structural Performance Level S-1, Immediate occupancy, means the post-earthquake damage state in which only very limited structural damage has occurred. The basic vertical-, and lateral-force-resisting systems of the building retain nearly all of their pre-earthquake strength and stiffness. The risk of life-threatening injury as a result of structural damage is very low, and although some minor structural repairs may be appropriate, these would generally not be required prior to reoccupancy.

Life Safety Performance Level (S-3): Structural Performance Level S-3, Life Safety, means the post-earthquake damage state in which significant damage to the structure has occurred, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged, but this has not resulted in large falling debris hazards, either within or outside the building. Injuries may occur during the earthquake; however, it is expected that the overall risk of life-threatening injury as a result of structural damage is low. It should be possible to repair the structure; however, for economic reasons this may not be practical. While the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupancy.

Collapse Prevention Performance Level (S-5): Structural Performance Level S-5, Collapse Prevention, means the building is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and—to a more limited extent—degradation in vertical-load-carrying capacity. However, all significant components of the gravity-load- resisting system must continue to carry their gravity load demands. Significant risk of injury due to falling hazards from structural debris may exist. The structure may not be technically practical to repair and is not safe for reoccupancy, as aftershock activity could induce collapse.

Damage Control Performance Range (S-2): Structural Performance Range S-2, Damage control, means the continuous range of damage states that entail less damage than that defined for the Life Safety level, but more than that defined for the Immediate Occupancy level. Design for Damage Control performance may be desirable to minimize repair time and operation interruption; as a partial means of protecting valuable equipment and contents; or to preserve important historic features when the cost of design for Immediate Occupancy is excessive. Acceptance criteria for this range may be obtained by interpolating between the values provided for the Immediate Occupancy (S-1) and Life Safety (S-3) levels.

Limited Safety Performance Range (S-4): Structural Performance Range S-4, Limited Safety, means the continuous range of damage states between the Life Safety and Collapse Prevention levels. Design parameters for this range may be obtained by interpolating between the values provided for the Life Safety (S-3) and Collapse Prevention (S-5) levels.

In this study, Life Safety Performance Level is taken as reference in the analysis of steel frames.

4.3. Pushover Analysis

In Pushover Analysis of a building, a nonlinear analytical model of the building is subjected to monotonically increasing lateral forces until reaching a predetermined ultimate displacement. The most common way to plot the force-displacement curve is by tracking the base shear and the roof displacement. The ultimate displacement represents the maximum displacement that will be experienced by the building during a particular earthquake and can be calculated using any method that accounts for the effect of nonlinear behaviour on the displacement of the building. The lateral loads are applied incrementally and stiffness properties of the building components are renewed after each load increment. The result of pushover analysis is the calculation of force and deformation demands on the building at the ultimate displacement.

The steps of pushover analysis used in this study can be explained as follows:

4.3.1. Nonlinear Model of the Structure

Frames are modeled in accordance with the procedure of the computer program stated in Chapter 3. Since connections affect the behaviour of the frame considerably, the crucial point in the modeling is the modeling of them. In order to observe the yielding criteria and to orient the pushover analysis, it was needed to know the moment capacities of the connections. The calculation of ultimate moment capacities of some fundamental semi-rigid connections according Semi-Rigid Connections in Steel Frames, 1992 are described as follows:

4.3.1.a Predicted Plastic Moment Capacity for Top and Seat Angle Type Connection:

The collapse mechanism on the top angle is modeled with the development of two plastic hinges. One is located along the line of the nut edge of the fastener, the other along the line of the toe of the fillet as shown in Figure 4-1. The work equation for the top angle at the collapse state is given by

$$2M_{pt}\theta = V_{pt}g_2\theta$$

where M_{pt} = Plastic moment in the top angle

V_{pt} = Plastic shear force in the vertical leg

g_2 = Pistance between two plastic hinges

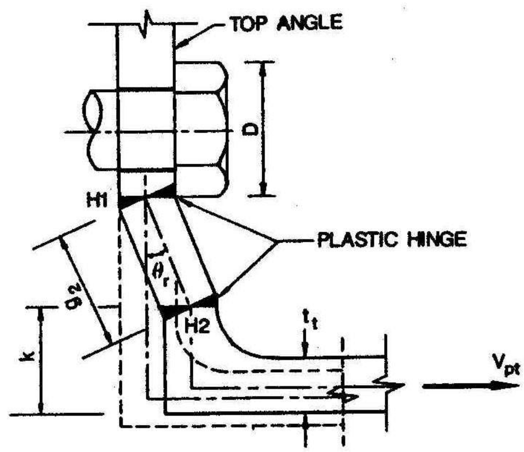


Figure 4-1: Mechanism Formed in Top Angle (Semi-Rigid Connections in Steel Frames, 1992)

The bending-shear interaction formula for the yielding state proposed by Drucker (1956) is used,

$$\frac{M_p}{M_0} + \left(\frac{V_p}{V_0}\right)^4 = 1$$

where M_0 and V_0 are the plastic bending moment and the plastic shear force. And according to Tresca's yielding criterion (Semi-Rigid Connections in Steel Frames, 1992),

$$M_0 = \frac{\sigma_y l_t (t_t)^2}{4}$$

$$V_0 = \frac{\sigma_y l_t t_t}{2}$$

where σ_y is the yield stress and l_t the length of the top angle. The above equations can be combined as,

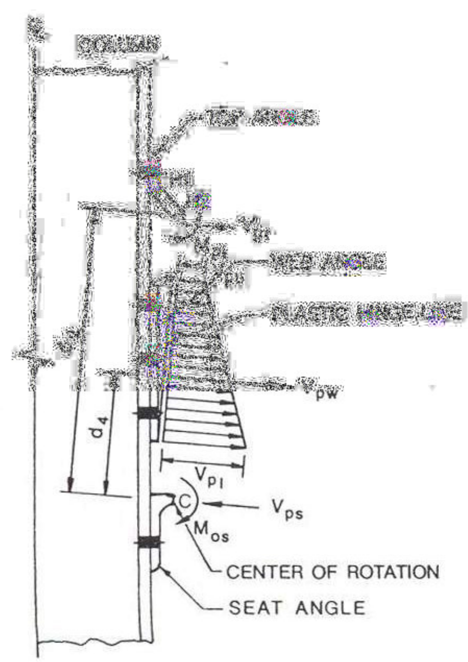
$$\left(\frac{V_{pt}}{V_0}\right)^4 + \frac{g_2}{t_t}\left(\frac{V_{pt}}{V_0}\right) = 1$$

Following a simple iterative procedure, V_{pt} and consequently M_{pt} can be determined. The mechanism moment capacity contributed by the top angle is obtained by taking the moment about the center of rotation C (Figure 4-2).

$$M_{ut} = M_{pt} + V_{pt} d_2$$

where $d_2 = d + \frac{t_s}{2} + k$

k = distance from heel to toe of fillet of angle



$$M_0 = \frac{\sigma_y l_s (t_s)^2}{4}$$

in which l_s and t_s are the length and thickness of the seat angle respectively. There is a considerable contribution of the depth of the beam in the capacity of top angle, when compared with the moment capacity of seat angle.

The overall Capacity of top and seat angle connections can be stated as follows:

$$M_{ut} = M_{0s} + M_{pt} + V_{pt} d_2 \qquad \text{(Eq: 4-1)}$$

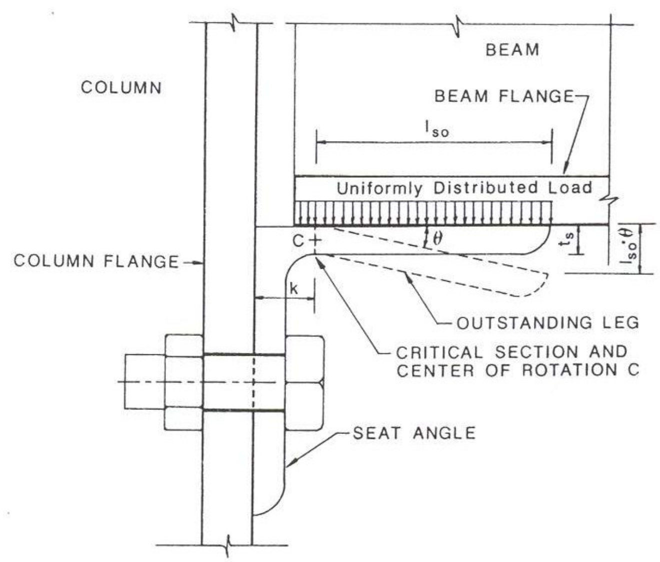


Figure 4-3: Cantilever Beam Model of Seat Angle (Semi-Rigid Connections in Steel Frames,1992)

4.3.1.b Predicted Plastic Moment Capacity for Double Web Angle Type Connection:

The collapse mechanism on the web angle is shown in Figure 4-4, where the two plastic hinge lines formed along the height of the web angle are assumed. One is located along the toe of the fillet, the other is located as an inclined line along the angle height. In this kind of connection, the effect of the shear force on the flexural bending capacity must be considered. According to Figure 4-4, the work equation for the mechanism at an arbitrary horizontal section y can be expressed as,

$$2M_{py}\theta = V_{py} g_y \theta$$

in which M_{py} is the plastic moment capacity and V_{py} the plastic shear force per unit height of a single web angle; g_y is the distance between the two plastic hinges at section y.

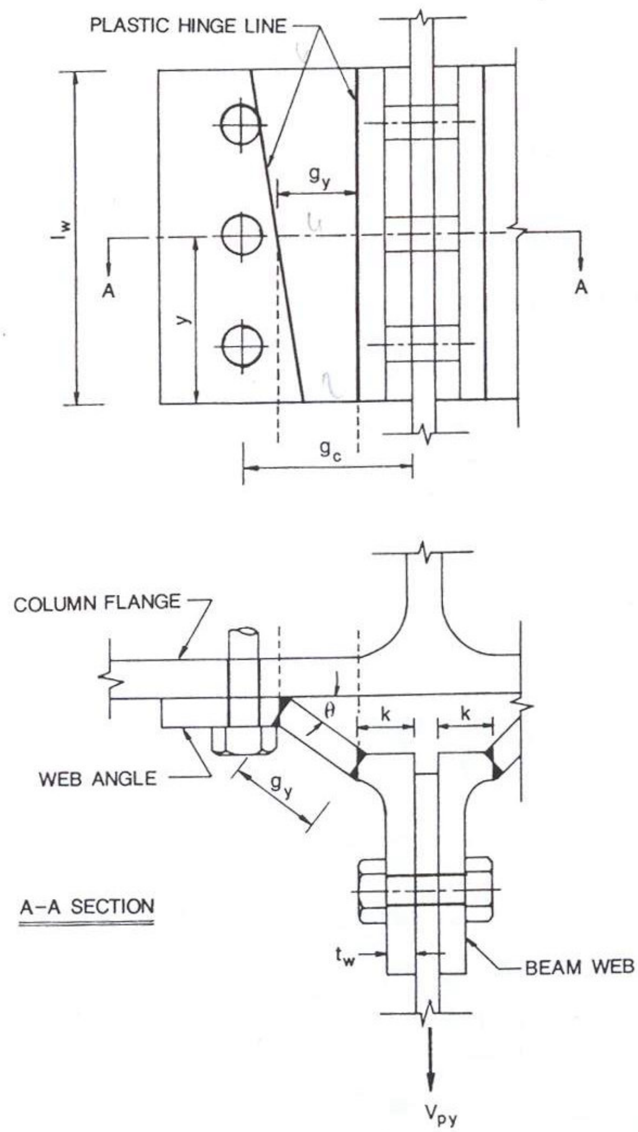


Figure 4-4: Mechanism Formed in Web Angle
(Semi-Rigid Connections in Steel Frames,1992)

Using the same equations for the top and seat angle connections, the relationship between M_0 and V_0 a fourth order equation for calculating the plastic shear force V_{py} is obtained,

$$\left(\frac{V_{py}}{V_0}\right)^4 + \frac{G_y}{t_w}\left(\frac{V_{py}}{V_0}\right) = 1$$

The solution of plastic shear force V_{py} from the above equation has a nonlinear distribution along the height of the web angle as shown in Figure 4-5. The maximum value V_{pl} is at lower edge $y = 0$, and the minimum value V_{pu} is at upper edge $y = 1$. For simplicity, the variation of V_{py} is assumed to be a bilinear distribution in Figure 4-5. The resultant plastic shear force is as follows:

$$V_{pw} = \frac{l_w}{4}\left(V_{pu} + 2V_{pm} + V_{pl}\right)$$

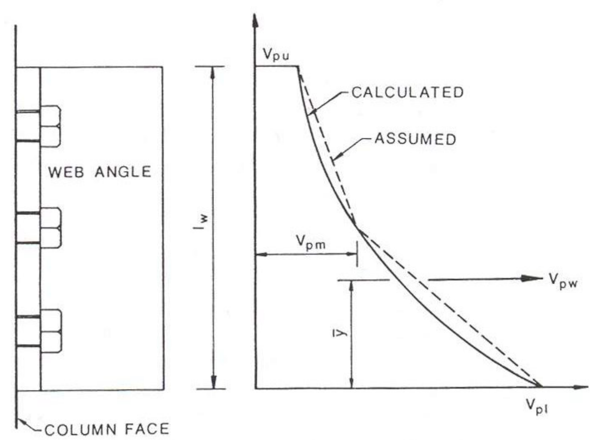


Figure 4-5: Distribution of Plastic Shear Force in the Web Angle (Semi-Rigid Connections in Steel Frames,1992)

The mechanism moment capacity contributed by the double web angle connection is obtained by taking the moment of the shear force V_{pw} around the center of rotation C.

$$M_{uw} = 2V_{pw} d_4 \qquad \text{(Eq: 4-2)}$$

where $d_4 = \overline{y} + l_l + \frac{t_s}{2}$

$\overline{y} =$ height that shear force V_{pw} acted on, measured from lower edge of web angle

4.3.1.c Predicted Plastic Moment Capacity for Top and Seat Angle Connections with Double Web Angle

The mechanism moment capacity of the top and seat angle connection with double web angle is the sum of above equations,

$$M_{ut} = \underbrace{M_{0s} + M_{pt} + V_{pt} d_2}_{TopandSeatAngle} + \underbrace{2V_{pw} d_4}_{DoubleWebAngle} \qquad \text{(Eq: 4-3)}$$

4.3.2. Application of Gravity Loads

In order to reflect the actual situation of the connections before the application of lateral loads, gravity loads are applied incrementally in the analysis. Besides, dead load and live load, any other distributed or point load can be assigned to the structure. The number of steps for incremental gravity load can be defined in the program. By this step, instantaneous situation of the connections can be achieved before starting the application of actual pushover lateral loads.

4.3.2. Application of Lateral Loads

After assigning gravity loads, lateral loads are assigned to the structure according to load coefficients in Equation 4-4 as specified in Specification for Structures to be Built in Disaster Areas (AY,1998).

$$F_{fi} = \frac{w_i H_i}{\sum_{j=1}^{N} (w_j H_j)} \qquad \text{(Eq: 4-4)}$$

where,

F_{fi} = Fictitious load acting at i'th storey in the determination of fundamental natural vibration period

w_i = Weight of i'th storey of building by considering live load participation factor

H_i = Height of i'th storey of building measured from the top foundation level

Computed lateral loads were applied equally to the joints at the relevant storey.

4.3.3 Construction of Capacity Curve of Steel Frame's Response

Capacity curve according to first mode shape of the elastic model of the structure can be constructed until the control point at the roof storey exceeds the predetermined displacement or formation of yield mechanisms in the critical frame members.

The capacity curve is generally constructed to represent the first mode response of the structure for the buildings with fundamental period of vibration up to about one second. In the computation of first mode shape, the deformations of the frame due to the above mentioned lateral load coefficients were taken into consideration. These deformations were normalized according to roof displacement. Mode shapes were adjusted for each load increment in accordance with the change in deformations and member stiffnesses.

There were two predetermined control points for proceeding the construction of capacity curve, top displacement and yield of critical member sections. Top drift was limited up to 2.5% for steel moment frames as mentioned in FEMA-273 in accordance with the Life Safety Performance Level (S-3). On the other hand, pushover analysis was continued until the formation of yield mechanism in one of the base storey columns due to the exceeding of plastic moment capacity of the section.

When one of the connections was faced with greater moments than its ultimate moment capacity, stiffness of this connection was taken as zero in the next load increment. This means, that joint has no more resistance to rotation. However, it keep its role in the contribution to the axial capacity of the relevant beam.

Construction of capacity curve was continued until one of the stated limitations surpasses. Where deformation governs for semi-rigid flexible connections, yielding of base column governs for more rigid structures.

Base shear was computed by adding up the shear forces of base floor columns. By this manner capacity curve was constructed like in Figure 4-6.

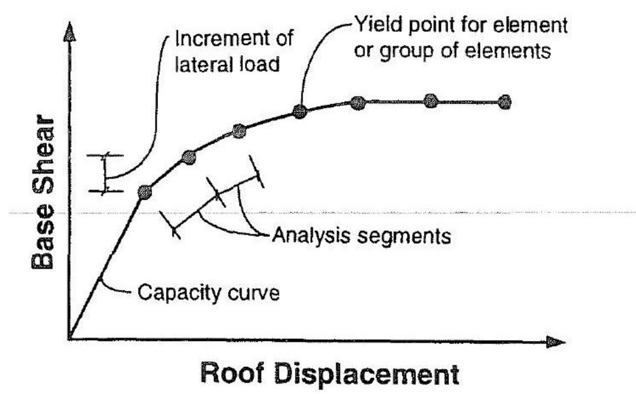


Figure 4-6: Capacity Curve (ATC-40)

4.4. Capacity Spectrum Method According to Acceleration-Displacement Response Spectra (ADRS)

The demand displacement in the capacity spectrum method occurs at a point on the capacity spectrum called performance point. This performance point represents the condition for which the seismic capacity of the structure is equal to the seismic demand imposed on the structure by the specified ground motion. (ATC-40)

The earthquake force capacity of a building represented by a force displacement curve is obtained from pushover analysis. The earthquake demands are represented by response spectra curves. The capacity curve, which is in terms of base shear and top displacement, obtained from the pushover analysis is transformed into equivalent spectral coordinates. This conversion is known as Acceleration-Displacement Response Spectra.

By taking into the consideration only the first mode, the base shear and spectral acceleration S_a are related by the Equation 8.3 in ATC-40,

$$S_a = (V/W)/\alpha_1$$

Where V is the value of base shear and α_1 is the first mode modal mass coefficient, which is given as Equation 8.2. in ATC-40.

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i1})/g\right]^2}{\left[\sum_{i=1}^{N} (w_i / g)\right]\left[\sum_{i=1}^{N} (w_i \phi_{i1}^2)/g\right]}$$

Again, taking into consideration only the first mode of the building, the roof displacement S_d is related by equation 8.4, in ATC-40.

$$S_d = \frac{\Delta_{roof}}{PF_1 \phi_{roof\,1}}$$

Where Δ_{roof} is a value for roof displacement from capacity curve, ϕ_{roof1} is the value of the fundamental mode shape at the roof and PF_1 is the first mode modal participation factor, equation 8.1 in ATC-40.

$$PF_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i1}) / g \right]}{\left[\sum_{i=1}^{N} (w_i \phi_{i1}{}^2) / g \right]}$$

The participation factor and the modal mass coefficient can vary according to the relative interstory displacement over the height of the building.

Like given in the earthquake spectra, most engineers are more familiar with the traditional S_a vs. T representation rather than S_a vs. S_d (ADRS) representation. In order to compare the capacity and demand diagrams, earthquake spectra should convert into S_a and S_d. This can be done with the equation,

$$S_d = \frac{T_i^2}{4\pi^2} S_a g$$

Also for any point on the ADRS spectrum, the period T, can be calculated by using the relationship,

$$T = 2\pi \left(\frac{S_d}{S_a} \right)^{1/2}$$

CHAPTER 5

CASE STUDIES FOR SEISMIC PERFORMANCE EVALUATION OF STEEL FRAMES WITH RIGID AND SEMI-RIGID CONNECTIONS

5.1 General

Essentially 3 planar steel frames with different connection types are investigated as case studies. The number of stories is selected as 3 for each frame and the number of bays is changed as 1, 3 and 5. In order to make a good comparison, geometric and loading parameters are kept as similar as possible. Taking into the consideration the actual conditions and applications, the typical floor height was selected as 4m, where the typical bay width was 6m. For the effective span load calculation, the spacing of sample planar frame was taken as 5m.

Since the earthquake spectra for all kind of earthquake zones are drawn as demand curves, there is no earthquake zone defined for the case study frames. Also, performances of steel frames according to different soil types defined in Specification for Structures to be Built in Disaster Areas (AY, 1998) are drawn in order to compare the performances of the steel frames which are in the same EQ Zone but have different soil conditions.

I-shaped sections were selected as beams, where wide flange I-Shaped sections were assigned as columns. Equal leg angles were used for the connection of beams and columns. Column sections were selected in accordance with the rule of strong-column weak-beam principle, which proposes minimum 20% stronger columns than beams. All steel sections are ST-37 grade structural steel sections.

Overall live load was taken as 200 kg/m2 and this load was applied as distributed load of 1,0 t/m2 considering the contribution of 5 m. spacing. Also this distributed load was applied as half for the roof floor. There was no other load was assigned to the frames except their dead loads which is computed automatically.

5.2 Modelling of Frame

Case frames were modeled in accordance with the procedure of the computer program stated in aforementioned chapters. In case studies, 3 types of semi-rigid connection types were assigned for the beam-column connections. Double web angle (Figure 2-6), top & seat angle (Figure 2-8) and top & seat angle with double web angle connections (Figure 2-7) are the types used in the case studies.

5.2.1. Numerical Computation of the Capacities of Top and Seat Angle Connections Used in Case Studies

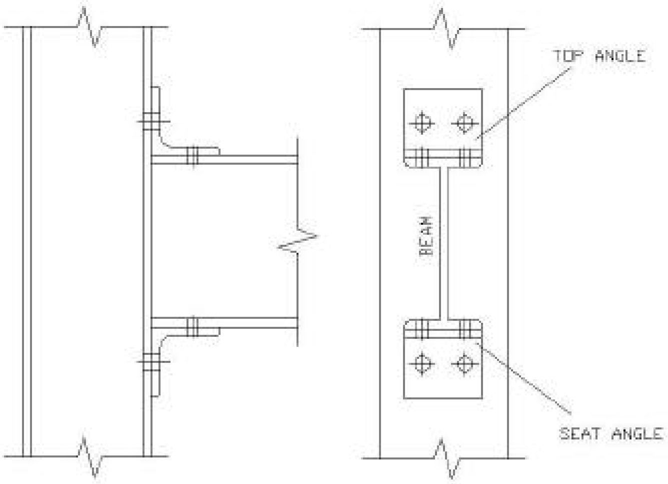


Figure 5-1: Connection 1 as Top and Seat Angle

Connection 1-1:

Top Angle Section: L 100/100/10
Length of Top Angle: 9 cm.
Seat Angle Section: L 100/100/10
Length of Seat Angle: 9 cm.
Relevant beam Section: I 200

$$M_0 = \frac{2.4x9x1}{4} \Rightarrow 0{,}054tm.$$

$$g_2 = 5 - 2 - \frac{2.85}{2} - 0.5 \Rightarrow 1{,}075cm$$

$$V_0 = \frac{2.4x9x1^2}{2} \Rightarrow 10{,}8t.$$

$$\left(\frac{V_{pt}}{10{,}8}\right)^4 + \frac{1{,}075}{1}\left(\frac{V_{pt}}{10{,}8}\right) = 1$$

After an iterative procedure,

$$\Rightarrow V_{pt} = 7{,}59t$$

According to bending-shear interaction formula $M_{pt} = 0{,}04tm.$

$$d_2 = 20 + \frac{1}{2} + 2 \Rightarrow 22{,}5cm$$

and overall plastic capacity of top and seat angle yields in,

$$M_{ut} = 0{,}054 + 0{,}04 + 7{,}59x0{,}225 \Rightarrow 1{,}802tm.$$

Connection 1-2:

Top Angle Section: L 100/100/10

Length of Top Angle: 9 cm.

Seat Angle Section: L 100/100/10

Length of Seat Angle: 9 cm.

Relevant beam Section: I 220

Only difference in the capacity calculation is change of d_2 due to increase in beam depth.

$M_0 = 0{,}054tm.$ $V_0 = 10{,}8t.$ $V_{pt} = 7{,}59t.$ $M_{pt} = 0{,}04tm.$

$$d_2 = 22 + \frac{1}{2} + 2 \Rightarrow 24{,}5cm$$

$$M_{ut} = 0{,}054 + 0{,}04 + 7{,}59x0{,}245 \Rightarrow 1{,}954tm.$$

Connection 1-3:

Top Angle Section: L 80/80/8

Length of Top Angle: 9 cm.

Seat Angle Section: L 80/80/8

Length of Seat Angle: 9 cm.

Relevant beam Section: I 200

$$M_0 = \frac{2,4x9x0,8}{4} \Rightarrow 0,043tm.$$

$$g_2 = 4,5 - 2 - \frac{2,85}{2} - 0,5 \Rightarrow 0,575cm$$

$$V_0 = \frac{2,4x9x(0,8)^2}{2} \Rightarrow 6,912t.$$

$$\left(\frac{V_{pt}}{6,912}\right)^4 + \frac{0,575}{0,8}\left(\frac{V_{pt}}{6,912}\right) = 1$$

After an iterative procedure,

$$\Rightarrow V_{pt} = 5,57t$$

According to bending-shear interaction formula $M_{pt} = 0,025tm.$

$$d_2 = 20 + \frac{1}{2} + 2 \Rightarrow 22,5cm$$

and overall plastic capacity of top and seat angle yields in,

$$M_{ut} = 0,043 + 0,025 + 5,57x0,225 \Rightarrow 1,321tm.$$

5.2.2. Numerical Computation of the Capacities of Double Web Angle Connections Used in Case Studies

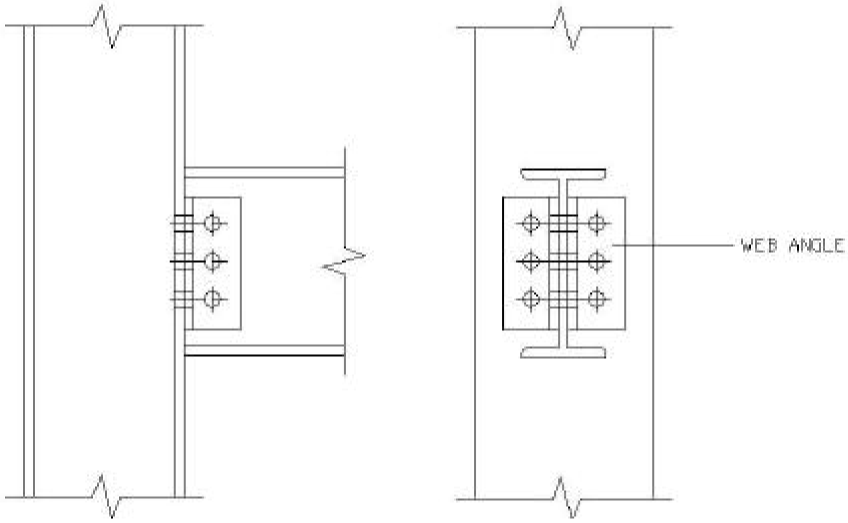


Figure 5-2: Connection Type 2 as Double Web Angle

Connection no: 2-1

Web Angle Section: L 80/80/8

Length of Web Angle: 15 cm.

Relevant beam Section: I 200

$$V_0 = \frac{2{,}4x1x0{,}8}{2} \Rightarrow 0{,}96t.$$

for $g_y = 5$ $\left(\frac{V_{py}}{0{,}96}\right)^4 + \frac{5}{0{,}8}\left(\frac{V_{py}}{0{,}96}\right) = 1 \Rightarrow V_{py} = 0{,}154t.$

for $g_y = 2$ $\left(\frac{V_{py}}{0{,}96}\right)^4 + \frac{2}{0{,}8}\left(\frac{V_{py}}{0{,}96}\right) = 1 \Rightarrow V_{py} = 0{,}375t.$

for $g_y = 3{,}5$ $\left(\frac{V_{py}}{0{,}96}\right)^4 + \frac{3{,}5}{0{,}8}\left(\frac{V_{py}}{0{,}96}\right) = 1 \Rightarrow V_{py} = 0{,}219t.$

$$\Rightarrow V_{pw} = \frac{15}{4}(0{,}154 + 0{,}375 + 2x0{,}219)$$

$$V_{pw} = 3{,}626t$$

$$M_{uw} = 2x3{,}626x0{,}10 \Rightarrow 0{,}7252$$

Connection no: 2-2

Web Angle Section: L 80/80/14

Length of Web Angle: 15cm.

Relevant beam Section: I 200

$$V_0 = \frac{2{,}4x1x1{,}4}{2} \Rightarrow 1{,}68t.$$

for $g_y = 5$ $\quad \left(\frac{V_{py}}{1{,}68}\right)^4 + \frac{5}{1{,}4}\left(\frac{V_{py}}{1{,}68}\right) = 1 \Rightarrow V_{py} = 0{,}47t.$

for $g_y = 2$ $\quad \left(\frac{V_{py}}{1{,}68}\right)^4 + \frac{2}{1{,}4}\left(\frac{V_{py}}{1{,}68}\right) = 1 \Rightarrow V_{py} = 1{,}02t.$

for $g_y = 3{,}5$ $\quad \left(\frac{V_{py}}{1{,}68}\right)^4 + \frac{3{,}5}{1}\left(\frac{V_{py}}{1{,}68}\right) = 1 \Rightarrow V_{py} = 0{,}66t.$

$$\Rightarrow V_{pw} = \frac{10}{4}\left(0{,}47 + 1{,}02 + 2x0{,}66\right)$$

$$V_{pw} = 10{,}538t$$

$$M_{uw} = 2x10{,}538x0{,}10 \Rightarrow 2{,}108$$

<u>Connection 2-3:</u>

Web Angle Section: L 80/80/14

Length of Web Angle: 10cm.

Relevant beam Section: I 160

Following the same precedure,

$$\Rightarrow V_{pw} = \frac{10}{4}\left(0{,}47 + 1{,}02 + 2x0{,}66\right)$$

$$V_{pw} = 7{,}025t$$

$$M_{uw} = 2x7{,}025x0{,}08 \Rightarrow 1{,}124$$

5.2.3. Numerical Computation of the Capacities of Top and Seat Angle Connections With Double Web Angle Used in Case Studies

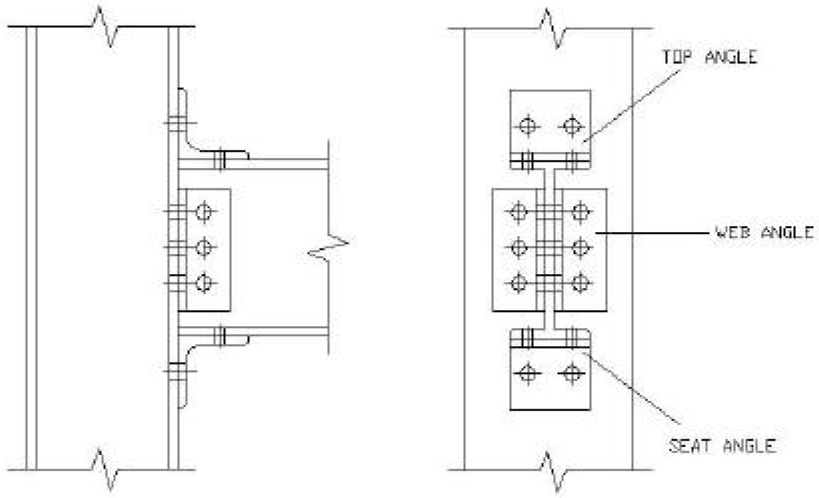


Figure 5-3: Connection Type 2 as Top and Seat Angle with Double Web Angle

Connection 3-1:

This type is the combination of connection 1-2 and Connection 2-2 and has a plastic capacity of,

$$M_{ut} = 1{,}954 + 2{,}108 \Rightarrow 4.062tm.$$

Connection 3-2:

This type is the combination of connection 1-1 and Connection 2-2 and has a plastic capacity of,

$$M_{ut} = 1{,}802 + 2{,}108 \Rightarrow 3{,}910tm.$$

Connection 3-1:

This type is the combination of connection 1-3 and Connection 2-1 and has a plastic capacity of,

$$M_{ut} = 1{,}321 + 0{,}7252 \Rightarrow 2{,}046tm.$$

5.3 Performance Calculations for Case Study #1

As a case study #1, a 3 floor-5 bay planar frame was considered. This frame was studied for rigid and semi-rigid cases in order to compare the assumed and real performances. General geometry and loading data of the frame are given in Figure 5-4. In order to obtain the responses for different connections for different frame sections, frame was investigated for its 8 different forms

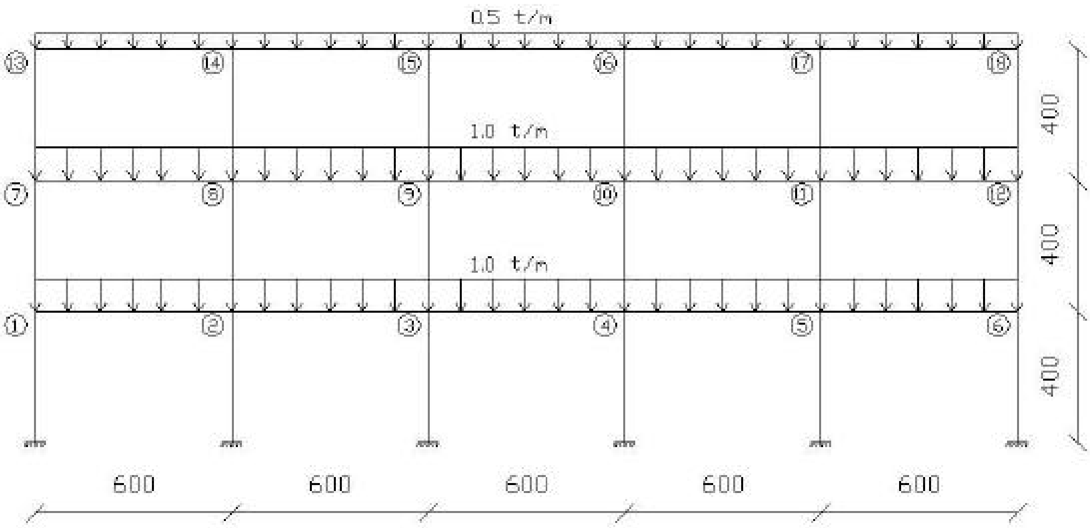


Figure 5-4: Loading and Geometry Data for Case Study #1

5.3.1 Case Study #1, Form 1

As a first form, relatively stronger sections were used for frame elements. Initially, the frame was considered as all connections are rigid. HE 200 AA and HE 180 AA sections were selected as column sections. On the other hand, I200 and I160 steel sections were used for the beams. With the %30 contribution of live load, the masses for first-two and roof floors were computed as 10,478 t and 5,608 t respectively. 30 cm top deflection and exceeding of the plastic capacity of any one of the base column was taken as the limit criteria for performance analysis. 30 cm top deflection limit was found according to the 2,5% drift allowance in FEMA-273.

All relevant output data for performance calculation, and constructed performance graph can be seen at Table 5-1 and Figure 5-5 respectively. In Table 5-1, it is seen that, within the defined limits, the frame faced with 15,6 tons of lateral load. Also, exceeding of plastic capacity of base columns is found to be the governing criteria in construction of capacity spectra.

Table 5-1: Data for Performance Analysis for Case Study 1-Form1

Number of Floors : 3	**Connections :** Rigid		
Number of Bays : 5	**P-Delta effect :** Yes	**Load Increment :** 400 kg	

Top Disp. *(meter)*	**B. Shear** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	**PF1**	α_1	S_d *(meter)*	S_a *(meter/s²)*
0,0000	0,0000	10,7650	10,7650	5,7260				0,0000	0,0000	0,0000	0,0000
0,0000	0,0000	10,7650	10,7650	5,7260	0,3249	0,1215	1,0000	0,5036	0,0653	0,0000	0,0000
0,0027	0,4000	10,7650	10,7650	5,7260	0,3539	0,7746	1,0000	1,3208	0,8661	0,0020	0,0169
0,0054	0,8000	10,7650	10,7650	5,7260	0,3773	0,7971	1,0000	1,3029	0,8781	0,0041	0,0334
0,0081	1,2000	10,7650	10,7650	5,7260	0,3855	0,8050	1,0000	1,2965	0,8820	0,0062	0,0499
0,0108	1,6000	10,7650	10,7650	5,7260	0,3897	0,8090	1,0000	1,2932	0,8839	0,0083	0,0664
0,0135	2,0000	10,7650	10,7650	5,7260	0,3922	0,8114	1,0000	1,2912	0,8851	0,0104	0,0829
0,0162	2,4000	10,7650	10,7650	5,7260	0,3939	0,8130	1,0000	1,2898	0,8858	0,0125	0,0994
0,0189	2,8000	10,7650	10,7650	5,7260	0,3951	0,8142	1,0000	1,2889	0,8864	0,0146	0,1159
0,0216	3,2000	10,7650	10,7650	5,7260	0,3960	0,8151	1,0000	1,2882	0,8868	0,0167	0,1324
0,0243	3,6000	10,7650	10,7650	5,7260	0,3967	0,8158	1,0000	1,2876	0,8871	0,0188	0,1489
0,0270	4,0000	10,7650	10,7650	5,7260	0,3973	0,8163	1,0000	1,2871	0,8874	0,0209	0,1654
0,0297	4,4000	10,7650	10,7650	5,7260	0,3977	0,8168	1,0000	1,2868	0,8876	0,0230	0,1819
0,0324	4,8000	10,7650	10,7650	5,7260	0,3981	0,8171	1,0000	1,2865	0,8877	0,0251	0,1984
0,0350	5,2000	10,7650	10,7650	5,7260	0,3985	0,8174	1,0000	1,2862	0,8879	0,0272	0,2149
0,0377	5,6000	10,7650	10,7650	5,7260	0,3987	0,8177	1,0000	1,2860	0,8880	0,0294	0,2314
0,0404	6,0000	10,7650	10,7650	5,7260	0,3990	0,8180	1,0000	1,2858	0,8881	0,0315	0,2479
0,0431	6,4000	10,7650	10,7650	5,7260	0,3992	0,8182	1,0000	1,2856	0,8882	0,0336	0,2644
0,0458	6,8000	10,7650	10,7650	5,7260	0,3994	0,8183	1,0000	1,2855	0,8883	0,0357	0,2809
0,0485	7,2000	10,7650	10,7650	5,7260	0,3996	0,8185	1,0000	1,2853	0,8884	0,0378	0,2974
0,0512	7,6000	10,7650	10,7650	5,7260	0,3997	0,8186	1,0000	1,2852	0,8884	0,0399	0,3139
0,0539	8,0000	10,7650	10,7650	5,7260	0,3998	0,8188	1,0000	1,2851	0,8885	0,0420	0,3303
0,0566	8,4000	10,7650	10,7650	5,7260	0,4000	0,8189	1,0000	1,2850	0,8886	0,0441	0,3468
0,0593	8,8000	10,7650	10,7650	5,7260	0,4001	0,8190	1,0000	1,2849	0,8886	0,0462	0,3633
0,0620	9,2000	10,7650	10,7650	5,7260	0,4002	0,8191	1,0000	1,2848	0,8886	0,0483	0,3798
0,0647	9,6000	10,7650	10,7650	5,7260	0,4003	0,8192	1,0000	1,2848	0,8887	0,0504	0,3963
0,0674	10,0000	10,7650	10,7650	5,7260	0,4004	0,8193	1,0000	1,2847	0,8887	0,0525	0,4128
0,0701	10,4000	10,7650	10,7650	5,7260	0,4004	0,8193	1,0000	1,2846	0,8888	0,0546	0,4293
0,0728	10,8000	10,7650	10,7650	5,7260	0,4005	0,8194	1,0000	1,2846	0,8888	0,0567	0,4458
0,0755	11,2000	10,7650	10,7650	5,7260	0,4006	0,8195	1,0000	1,2845	0,8888	0,0588	0,4623
0,0782	11,6000	10,7650	10,7650	5,7260	0,4006	0,8195	1,0000	1,2845	0,8889	0,0609	0,4788
0,0809	12,0000	10,7650	10,7650	5,7260	0,4007	0,8196	1,0000	1,2844	0,8889	0,0630	0,4953
0,0836	12,4000	10,7650	10,7650	5,7260	0,4008	0,8197	1,0000	1,2844	0,8889	0,0651	0,5118
0,0863	12,8000	10,7650	10,7650	5,7260	0,4008	0,8197	1,0000	1,2843	0,8889	0,0672	0,5283
0,0890	13,2000	10,7650	10,7650	5,7260	0,4009	0,8198	1,0000	1,2843	0,8889	0,0693	0,5448
0,0916	13,6000	10,7650	10,7650	5,7260	0,4009	0,8198	1,0000	1,2842	0,8890	0,0714	0,5613
0,0943	14,0000	10,7650	10,7650	5,7260	0,4009	0,8198	1,0000	1,2842	0,8890	0,0735	0,5778
0,0970	14,4000	10,7650	10,7650	5,7260	0,4010	0,8199	1,0000	1,2842	0,8890	0,0756	0,5943
0,0997	14,8000	10,7650	10,7650	5,7260	0,4010	0,8199	1,0000	1,2841	0,8890	0,0777	0,6108
0,1024	15,2000	10,7650	10,7650	5,7260	0,4011	0,8199	1,0000	1,2841	0,8890	0,0798	0,6273
0,1051	15,6000	10,7650	10,7650	5,7260	0,4011	0,8200	1,0000	1,2841	0,8891	0,0819	0,6438

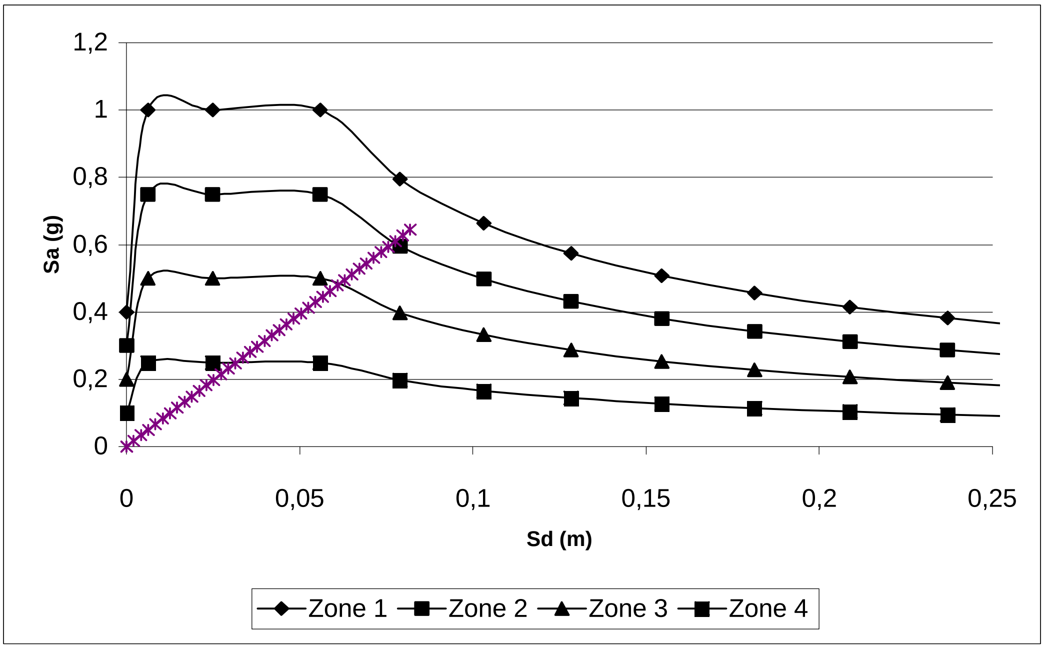


Figure 5-5: Capacity vs. Demand Spectra for Rigid Case (Form 1, Soil Type:Z1)

By looking at Figure 5-5, the frame seems to be capable of resisting to a probable Earthquake that would be in Zone-2. Corresponding S_a and S_d values are found as 0,61 *g* and 7,8 cm. for this rigid case. Location of hinge(s) can be seen in Figure 5-6.

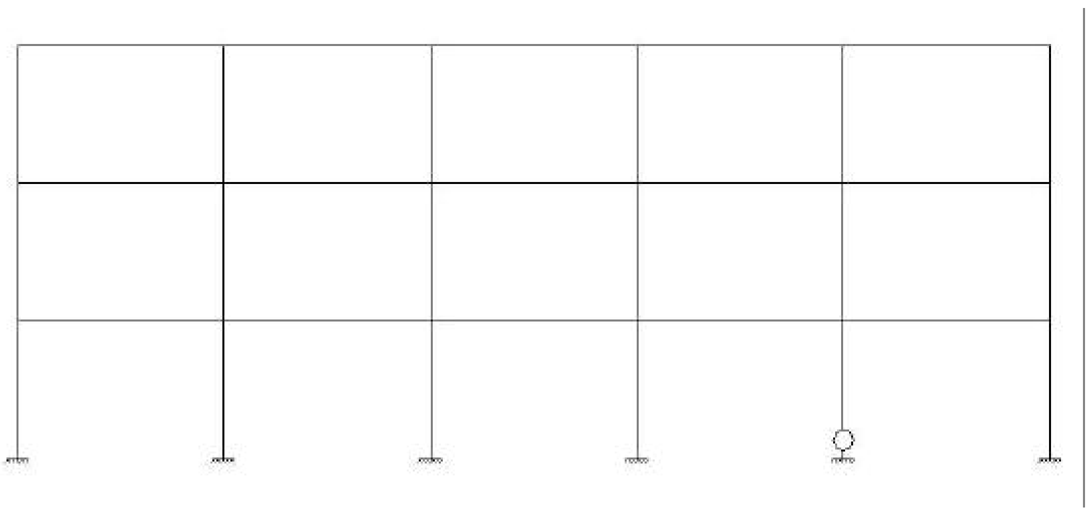


Figure 5-6: Hinge Formation for Form 1, Soil Type:Z1

(Base Shear: 15,6 t, Displacement: 10,5 cm)

Performance of the frame for Soil Type 2, 3 ,4 can be found in Figure 5-7, Figure 5-8 and Figure 5-9 respectively

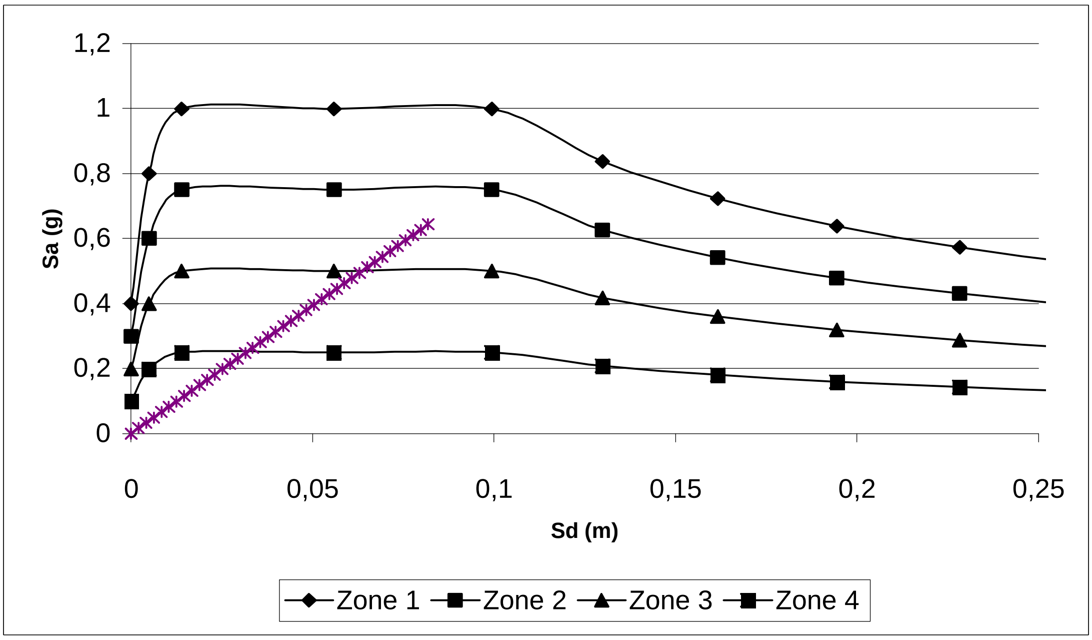


Figure 5-7: Capacity vs. Demand Spectra for Rigid Case (Form 1, Soil Type:Z2)

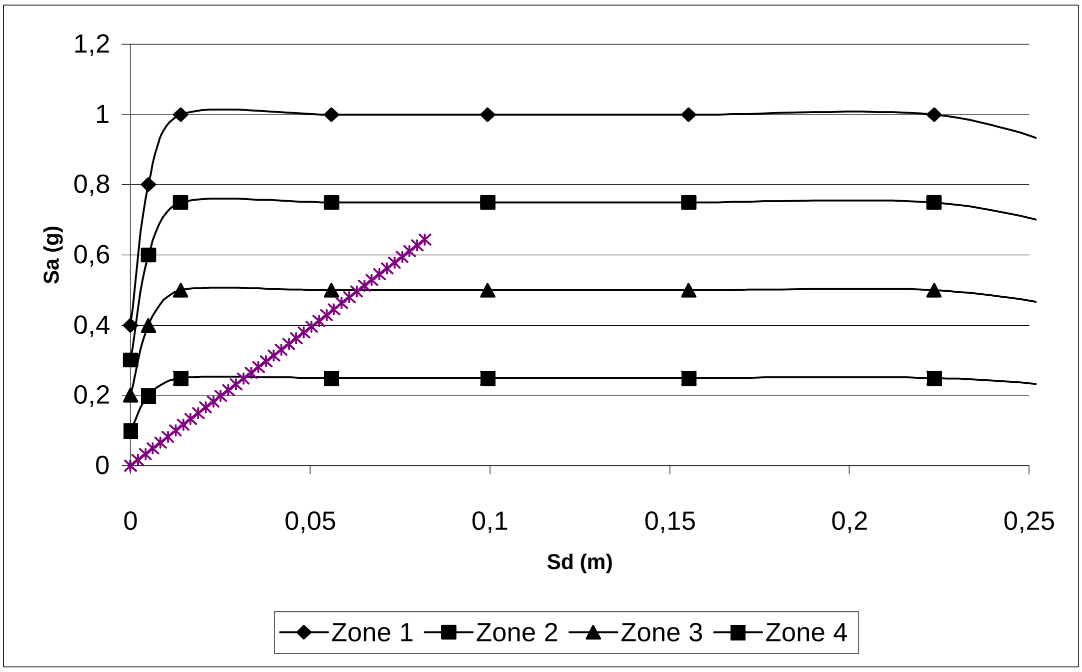


Figure 5-8: Capacity vs. Demand Spectra for Rigid Case (Form 1, Soil Type:Z3)

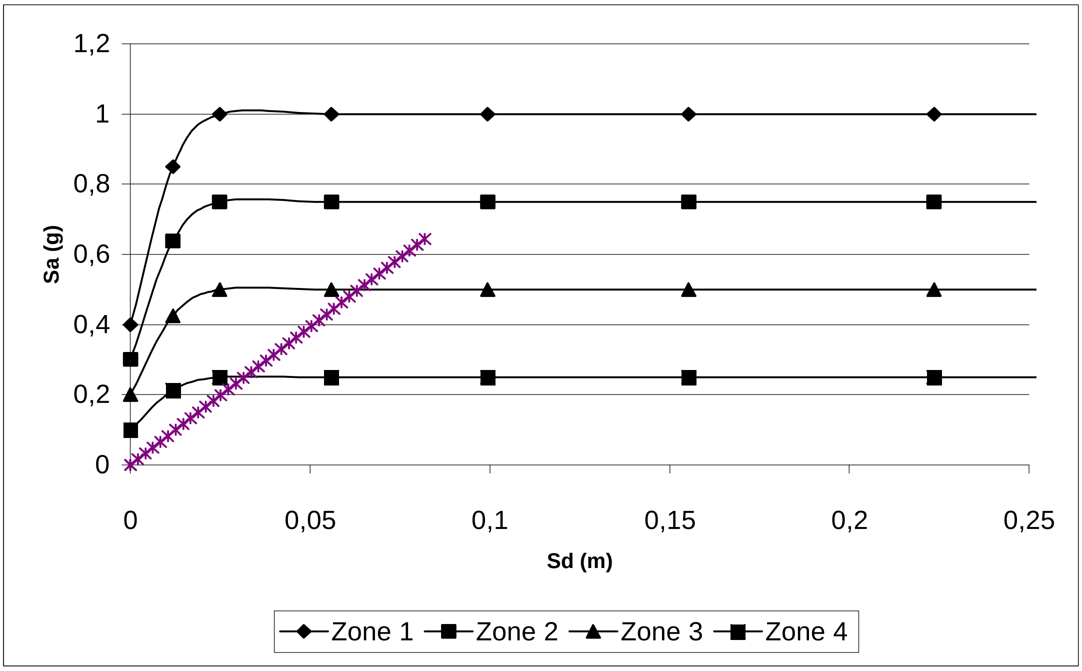


Figure 5-9: Capacity vs. Demand Spectra for Rigid Case (Form 1, Soil Type:Z4)

5.3.2 Case Study #1, Form 2

As a next form, connections of the case frame were assigned as semi-rigid. Location and types of semi-rigid connections are stated Figure 5-10.

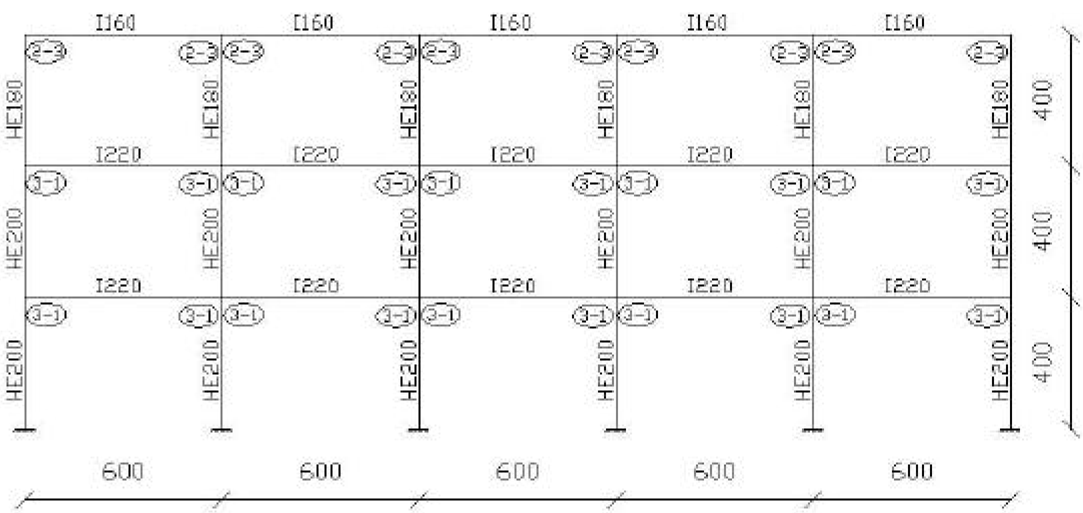


Figure 5-10: Semi Rigid Connection Types

Initially, gravity loads were applied in 20 load increment steps in order to obtain nonlinear response of the connections before lateral loads were applied. Then with a 400 kg of lateral load increment, pushover analysis was performed.

When looking at Table 5-2, it is seen that the total lateral load amount decreases from 15,8 tons to 12,8 tons. This is mainly due to the softening of frame, which causes an increase in lateral drift and consequently an increase in the base column moments. Location of hinges can be seen in Figure 5-11.

Table 5-2: Data for Performance Analysis for Case Study 1-Form 2

Number of Floors : 3			Connections :		Semi-Rigid		Load Increment :		400 kg		
Number of Bays : 5			P-Delta effect :		Yes						
Top Disp. (meter)	B. Shear (ton)	W_1 (ton)	W_2 (ton)	W_3 (ton)	f_1	f_2	f_3	PF1	α_1	S_d (meter)	S_a (meter/s^2)
0,0000	0,0000	10,7650	10,7650	5,7260				0,0000	0,0000	0,0000	0,0000
0,0032	0,4000	10,7650	10,7650	5,7260	0,2645	0,7406	1,0000	1,3361	0,8111	0,0024	0,0181
0,0065	0,8000	10,7650	10,7650	5,7260	0,3497	0,7604	1,0000	1,3324	0,8641	0,0049	0,0340
0,0098	1,2000	10,7650	10,7650	5,7260	0,3562	0,7673	1,0000	1,3270	0,8676	0,0073	0,0507
0,0130	1,6000	10,7650	10,7650	5,7260	0,3595	0,7708	1,0000	1,3242	0,8693	0,0098	0,0675
0,0164	2,0000	10,7650	10,7650	5,7260	0,3613	0,7729	1,0000	1,3225	0,8703	0,0124	0,0843
0,0197	2,4000	10,7650	10,7650	5,7260	0,3625	0,7744	1,0000	1,3213	0,8709	0,0149	0,1011
0,0232	2,8000	10,7650	10,7650	5,7260	0,3632	0,7755	1,0000	1,3204	0,8712	0,0175	0,1179
0,0267	3,2000	10,7650	10,7650	5,7260	0,3636	0,7763	1,0000	1,3198	0,8714	0,0202	0,1347
0,0302	3,6000	10,7650	10,7650	5,7260	0,3638	0,7769	1,0000	1,3193	0,8715	0,0229	0,1515
0,0339	4,0000	10,7650	10,7650	5,7260	0,3639	0,7774	1,0000	1,3189	0,8716	0,0257	0,1684
0,0377	4,4000	10,7650	10,7650	5,7260	0,3638	0,7777	1,0000	1,3186	0,8715	0,0286	0,1852
0,0416	4,8000	10,7650	10,7650	5,7260	0,3635	0,7780	1,0000	1,3184	0,8713	0,0315	0,2021
0,0456	5,2000	10,7650	10,7650	5,7260	0,3631	0,7781	1,0000	1,3182	0,8711	0,0346	0,2190
0,0497	5,6000	10,7650	10,7650	5,7260	0,3626	0,7782	1,0000	1,3182	0,8709	0,0377	0,2359
0,0540	6,0000	10,7650	10,7650	5,7260	0,3620	0,7781	1,0000	1,3182	0,8705	0,0410	0,2529
0,0585	6,4000	10,7650	10,7650	5,7260	0,3612	0,7779	1,0000	1,3183	0,8701	0,0444	0,2699
0,0632	6,8000	10,7650	10,7650	5,7260	0,3604	0,7776	1,0000	1,3186	0,8696	0,0479	0,2869
0,0681	7,2000	10,7650	10,7650	5,7260	0,3593	0,7771	1,0000	1,3189	0,8691	0,0516	0,3040
0,0732	7,6000	10,7650	10,7650	5,7260	0,3581	0,7765	1,0000	1,3194	0,8684	0,0554	0,3211
0,0785	8,0000	10,7650	10,7650	5,7260	0,3568	0,7757	1,0000	1,3200	0,8677	0,0595	0,3383
0,0841	8,4000	10,7650	10,7650	5,7260	0,3553	0,7747	1,0000	1,3207	0,8669	0,0637	0,3555
0,0899	8,8000	10,7650	10,7650	5,7260	0,3536	0,7735	1,0000	1,3217	0,8660	0,0681	0,3728
0,0961	9,2000	10,7650	10,7650	5,7260	0,3518	0,7721	1,0000	1,3227	0,8650	0,0727	0,3902
0,1026	9,6000	10,7650	10,7650	5,7260	0,3497	0,7704	1,0000	1,3240	0,8639	0,0775	0,4077
0,1095	10,0000	10,7650	10,7650	5,7260	0,3475	0,7685	1,0000	1,3255	0,8627	0,0826	0,4253
0,1167	10,4000	10,7650	10,7650	5,7260	0,3450	0,7662	1,0000	1,3272	0,8613	0,0879	0,4430
0,1243	10,8000	10,7650	10,7650	5,7260	0,3424	0,7637	1,0000	1,3291	0,8599	0,0936	0,4608
0,1324	11,2000	10,7650	10,7650	5,7260	0,3395	0,7609	1,0000	1,3312	0,8583	0,0995	0,4788
0,1410	11,6000	10,7650	10,7650	5,7260	0,3365	0,7578	1,0000	1,3336	0,8565	0,1058	0,4969
0,1502	12,0000	10,7650	10,7650	5,7260	0,3332	0,7544	1,0000	1,3362	0,8547	0,1124	0,5151
0,1599	12,4000	10,7650	10,7650	5,7260	0,3297	0,7506	1,0000	1,3390	0,8527	0,1194	0,5336
0,1703	12,8000	10,7650	10,7650	5,7260	0,3261	0,7466	1,0000	1,3420	0,8505	0,1269	0,5522

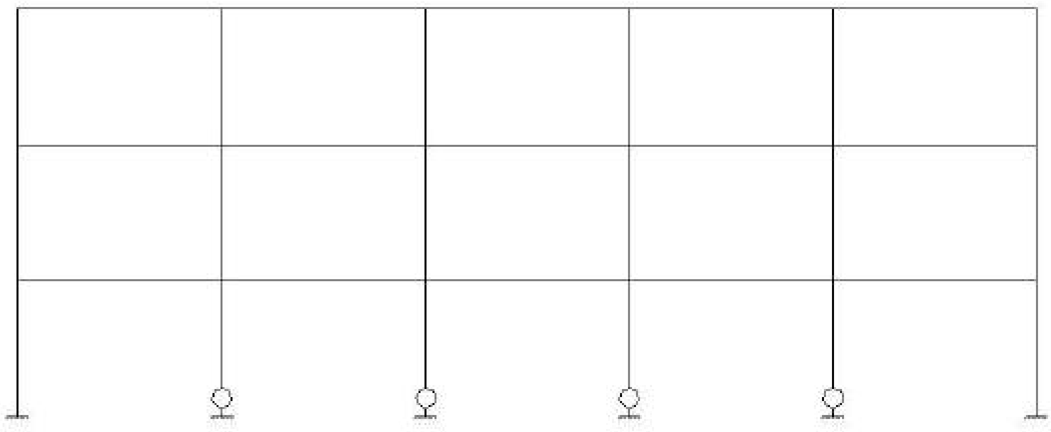


Figure 5-11: Hinge Formation for Form 2, Soil Type:Z1
(Base Shear: 12,8 t, Displacement: 17 cm)

With semi-rigid connections, again the frame is nearly able to resist to an earthquake in Zone-2. However, there is a considerable decrease in the S_a value with the increasing S_d. This difference is also due to the difference in the energy dissipation of two forms. Frame with semi-rigid connections dissipates nearly 20% more energy.

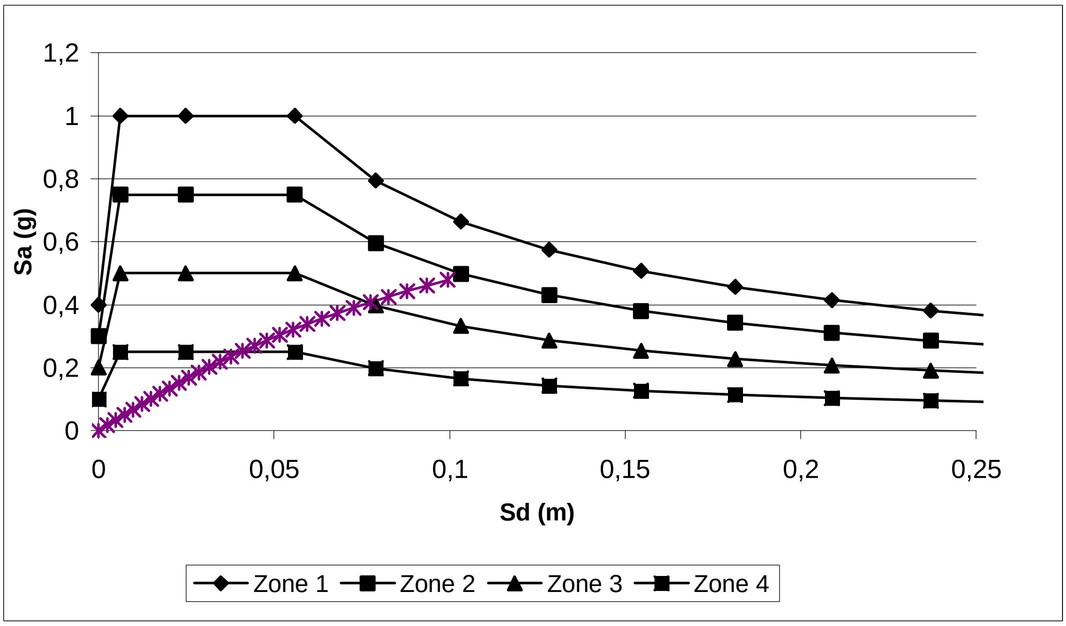


Figure 5-12: Capacity vs. Demand Spectra for Semi-Rigid Case (Form 2)

Performance of the frame for Soil Type 2, 3 ,4 can be found in Figure 5-13, Figure 5-14 and Figure 5-15 respectively

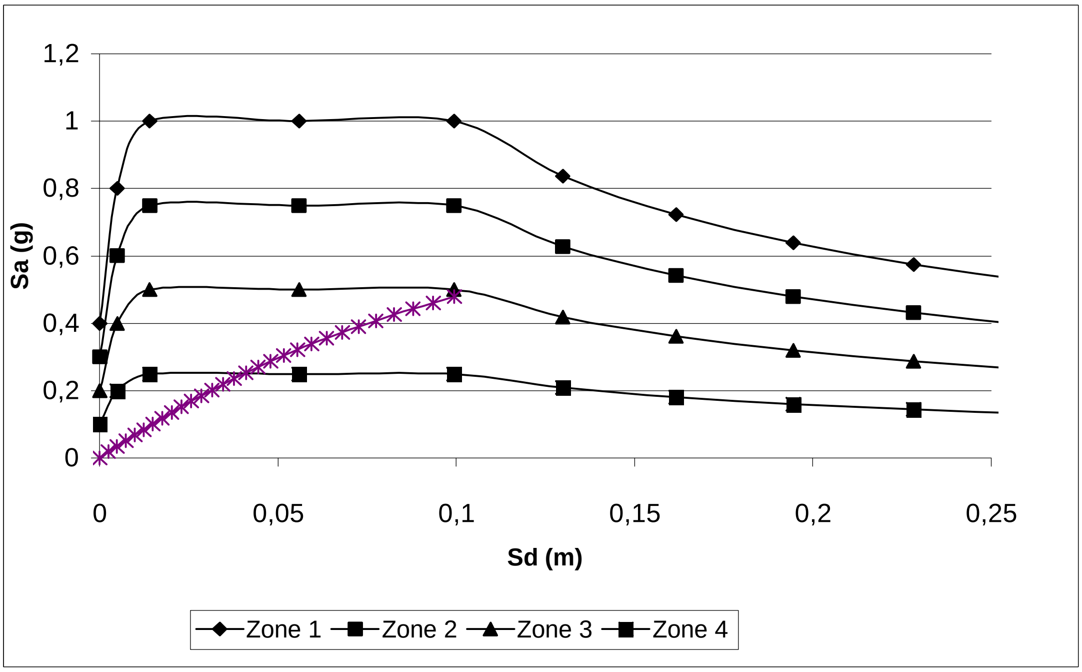


Figure 5-13: Capacity vs. Demand Spectra for Form 2, Soil type:Z2

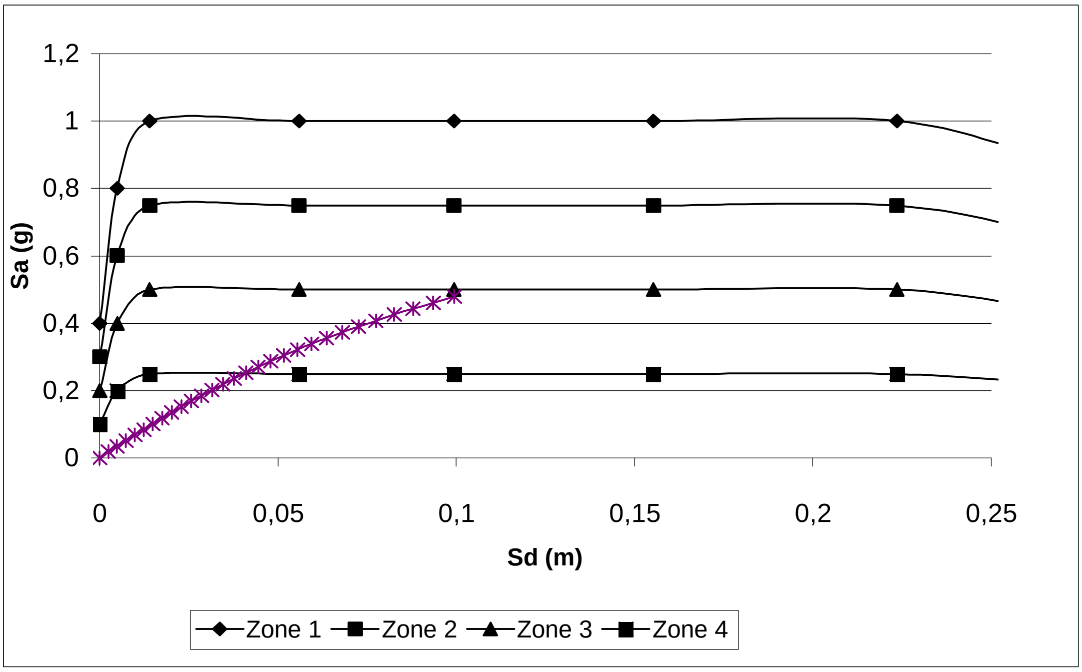


Figure 5-14: Capacity vs. Demand Spectra for Form 2, Soil Type:Z3

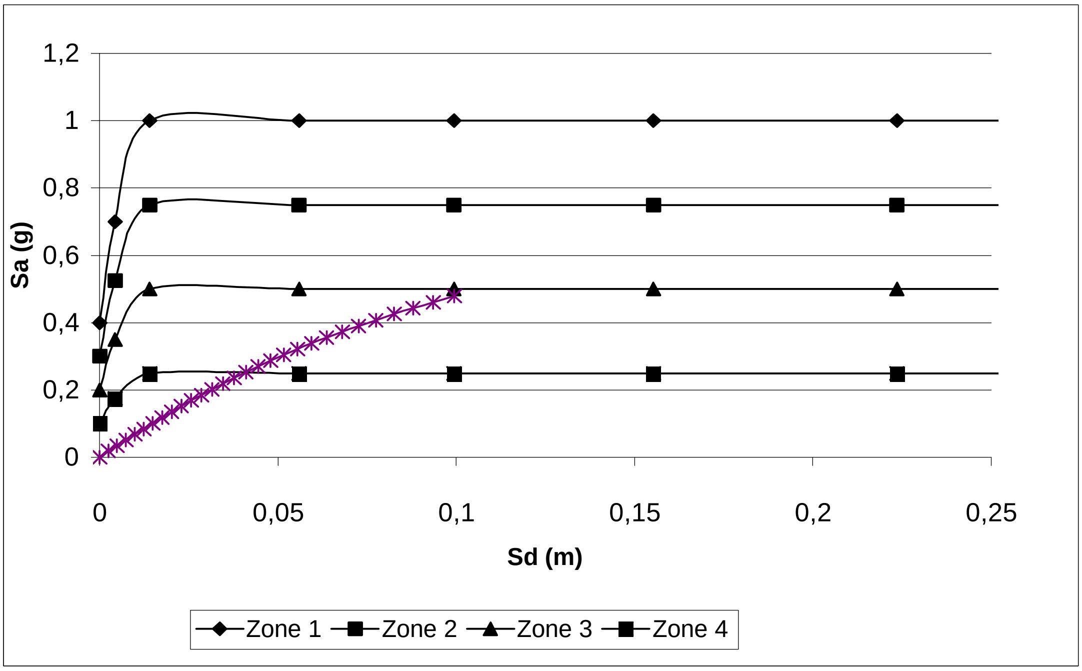


Figure 5-15: Capacity vs. Demand Spectra for Form 2, Soil Type:Z4

5.3.3 Case Study #1, Form 3

As Form 3, the sections of frame members were taken as HE180 and HE160 for columns and I 200 and I160 for beams. However loading was kept same as the previous forms. At first, with these sections, the frame was analyzed with rigid connections

Relevant performance calculation data for rigidly connected frame is given in Table 5-3. When compared with Form 1, it is seen that ultimate top deflection values are nearly same. However due to decreased section moment capacity, frame can resist up to 10.8 tons of lateral load. Location of hinges can be seen in Figure 5-16. The reflection of this capacity decrease is reflected in capacity demand curve in Figure 5-17. As expected, the performance of this rigid frame is less than the one's in the Form 1.

Table 5-3: Data for Performance Analysis for Case Study 1-Form 3

Number of Floors : 3 **Connections :** Fixed **Load Increment :** 400 kg

Number of Bays : 5 **P-Delta effect :** Yes

Top Disp. *(meter)*	***B. Shear*** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	***PF1***	α_1	S_d *(meter)*	S_a *(meter/s²)*
0,00000	0,00000	10,478	10,478	5,608				0,00000	0,00000	0,00000	0,00000
0,00002	0,00000	10,478	10,478	5,608	0,26455	0,20104	1,00000	1,55015	0,61194	0,00001	0,00000
0,00407	0,40000	10,478	10,478	5,608	0,38044	0,81203	1,00000	1,28995	0,87907	0,00315	0,01713
0,00814	0,80000	10,478	10,478	5,608	0,39775	0,82524	1,00000	1,27921	0,88715	0,00636	0,03395
0,01220	1,20000	10,478	10,478	5,608	0,40367	0,82976	1,00000	1,27547	0,88980	0,00957	0,05077
0,01627	1,60000	10,478	10,478	5,608	0,40666	0,83204	1,00000	1,27356	0,89112	0,01278	0,06759
0,02034	2,00000	10,478	10,478	5,608	0,40846	0,83342	1,00000	1,27241	0,89191	0,01599	0,08441
0,02441	2,40000	10,478	10,478	5,608	0,40967	0,83434	1,00000	1,27163	0,89243	0,01920	0,10124
0,02848	2,80000	10,478	10,478	5,608	0,41053	0,83500	1,00000	1,27108	0,89281	0,02240	0,11806
0,03255	3,20000	10,478	10,478	5,608	0,41118	0,83549	1,00000	1,27066	0,89309	0,02561	0,13488
0,03661	3,60000	10,478	10,478	5,608	0,41168	0,83588	1,00000	1,27034	0,89331	0,02882	0,15171
0,04068	4,00000	10,478	10,478	5,608	0,41208	0,83618	1,00000	1,27008	0,89348	0,03203	0,16853
0,04475	4,40000	10,478	10,478	5,608	0,41242	0,83644	1,00000	1,26986	0,89362	0,03524	0,18536
0,04882	4,80000	10,478	10,478	5,608	0,41269	0,83665	1,00000	1,26969	0,89374	0,03845	0,20218
0,05289	5,20000	10,478	10,478	5,608	0,41292	0,83682	1,00000	1,26954	0,89384	0,04166	0,21900
0,05696	5,60000	10,478	10,478	5,608	0,41312	0,83698	1,00000	1,26941	0,89393	0,04487	0,23583
0,06102	6,00000	10,478	10,478	5,608	0,41330	0,83711	1,00000	1,26929	0,89400	0,04808	0,25265
0,06509	6,40000	10,478	10,478	5,608	0,41345	0,83723	1,00000	1,26920	0,89406	0,05129	0,26947
0,06916	6,80000	10,478	10,478	5,608	0,41358	0,83733	1,00000	1,26911	0,89412	0,05449	0,28630
0,07323	7,20000	10,478	10,478	5,608	0,41370	0,83742	1,00000	1,26903	0,89417	0,05770	0,30312
0,07730	7,60000	10,478	10,478	5,608	0,41381	0,83750	1,00000	1,26896	0,89422	0,06091	0,31995
0,08136	8,00000	10,478	10,478	5,608	0,41390	0,83757	1,00000	1,26890	0,89426	0,06412	0,33677
0,08543	8,40000	10,478	10,478	5,608	0,41399	0,83764	1,00000	1,26885	0,89430	0,06733	0,35359
0,08950	8,80000	10,478	10,478	5,608	0,41407	0,83770	1,00000	1,26880	0,89433	0,07054	0,37042
0,09357	9,20000	10,478	10,478	5,608	0,41414	0,83775	1,00000	1,26875	0,89436	0,07375	0,38724
0,09764	9,60000	10,478	10,478	5,608	0,41421	0,83780	1,00000	1,26871	0,89439	0,07696	0,40407
0,10171	10,00000	10,478	10,478	5,608	0,41427	0,83785	1,00000	1,26867	0,89441	0,08017	0,42089
0,10577	10,40000	10,478	10,478	5,608	0,41432	0,83789	1,00000	1,26863	0,89444	0,08338	0,43771
0,10984	10,80000	10,478	10,478	5,608	0,41437	0,83793	1,00000	1,26860	0,89446	0,08659	0,45454

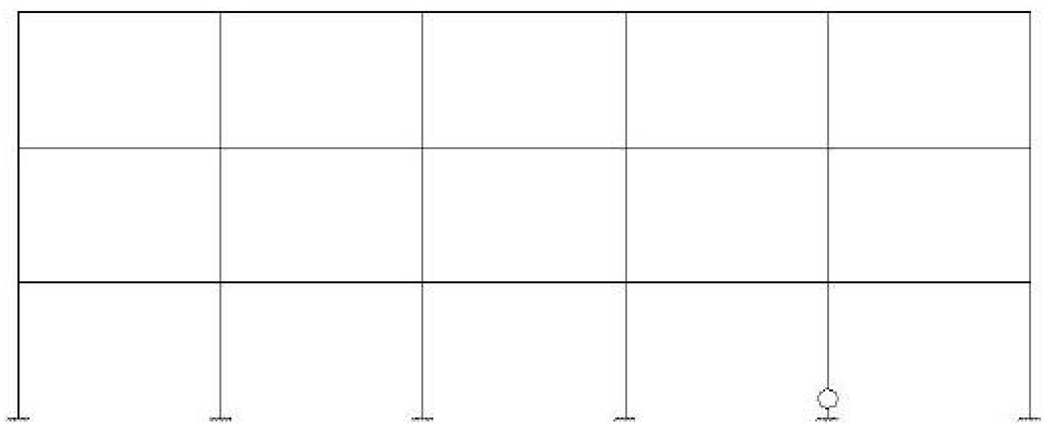


Figure 5-16: Hinge Formation for Form 3, Soil Type:1

(Base Shear: 10,8 t, Displacement: 10,1 cm)

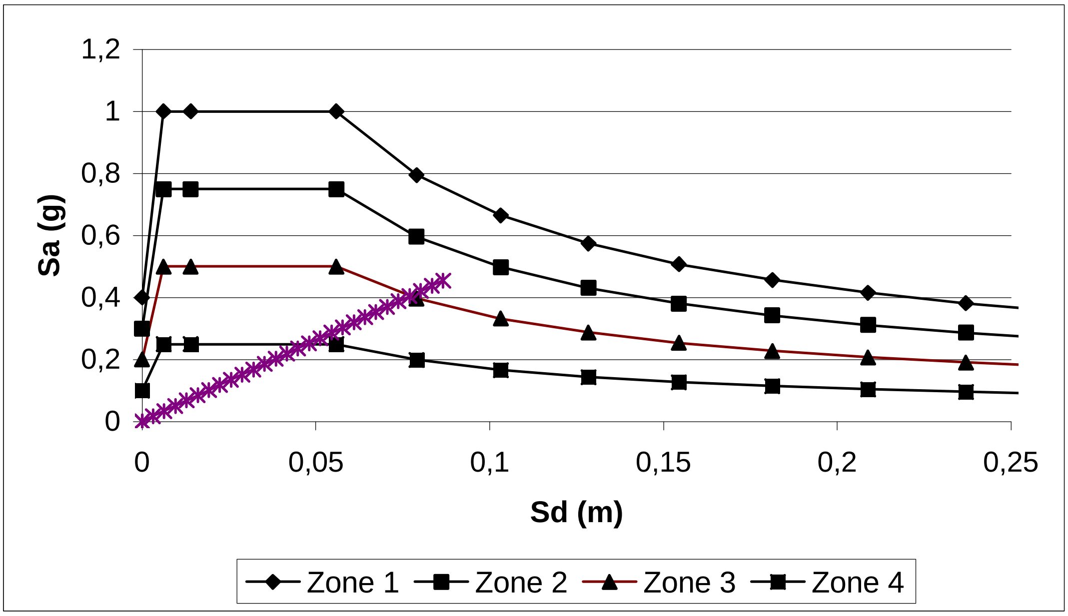


Figure 5-17: Capacity vs. Demand Spectra for Rigid Case (Form 3)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-18, Figure 5-19 and Figure 5-20 respectively

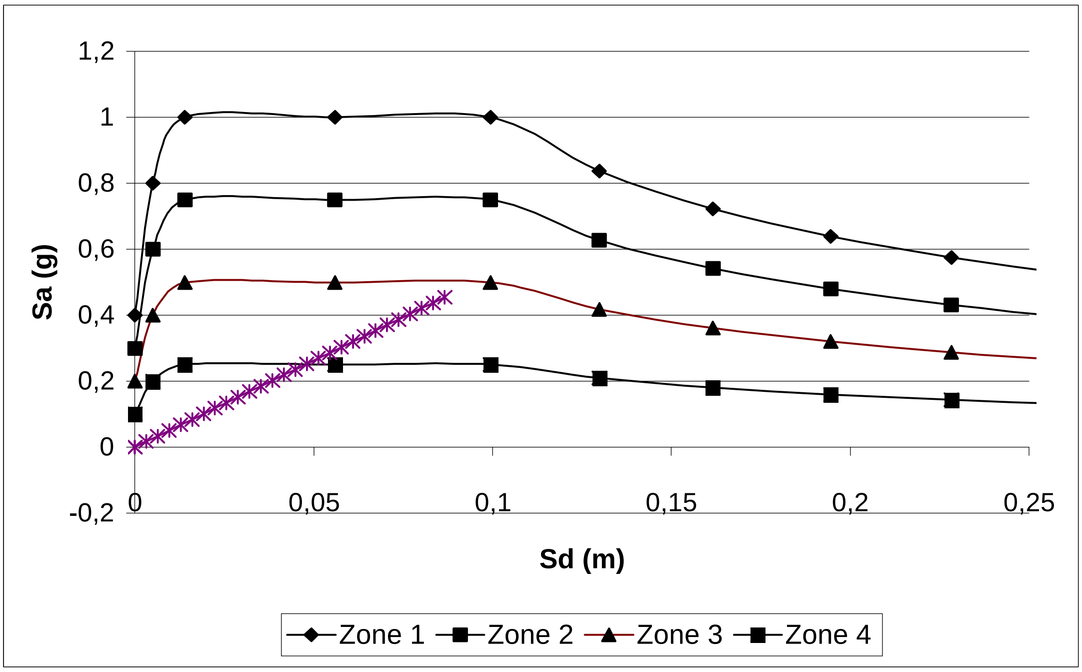


Figure 5-18: Capacity vs. Demand Spectra for Form 3, Soil Type:Z2

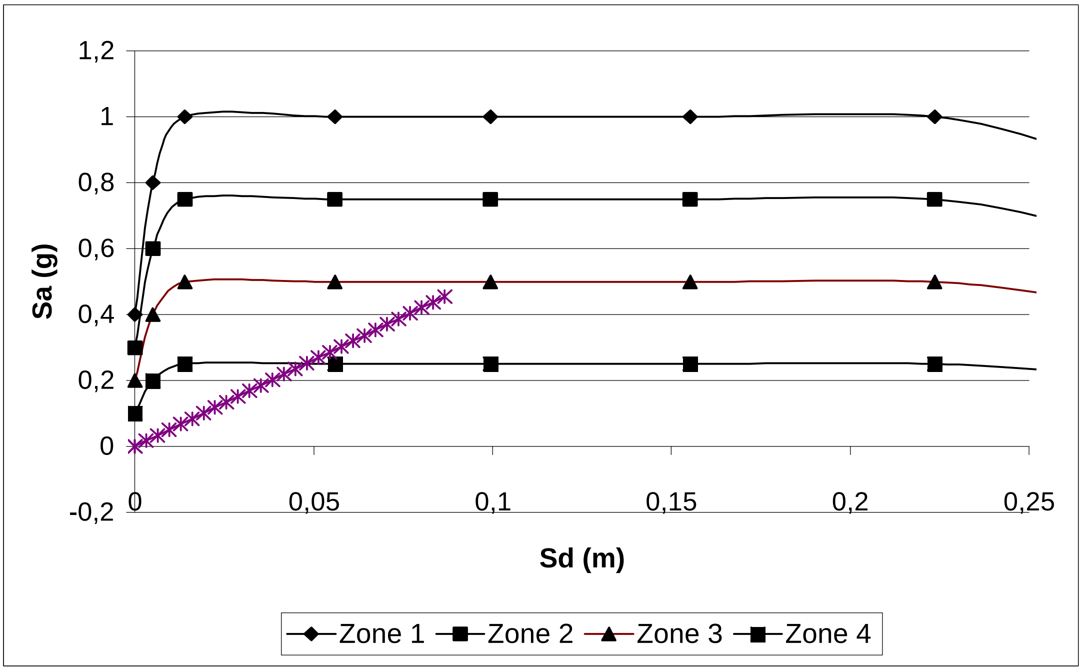


Figure 5-19: Capacity vs. Demand Spectra for Form 3, Soil Type:Z3

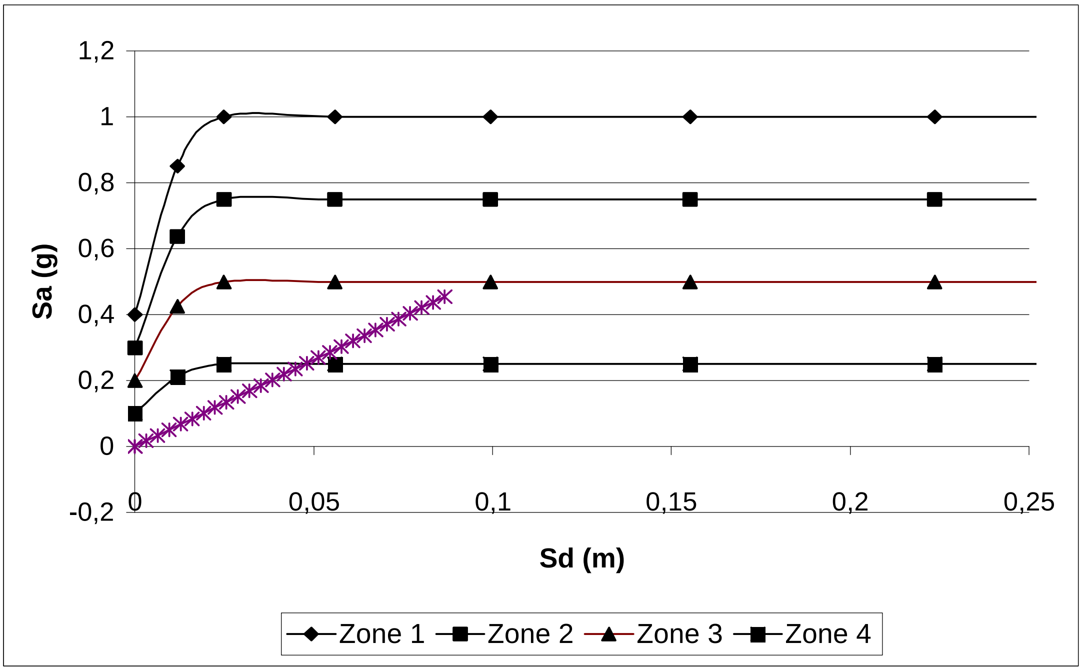


Figure 5-20: Capacity vs. Demand Spectra for Form 3, Soil Type:Z4

5.3.4 Case Study #1, Form 4

In Form 4, semi-rigid connections were assigned in the beam-column connections as in Figure 5-6.

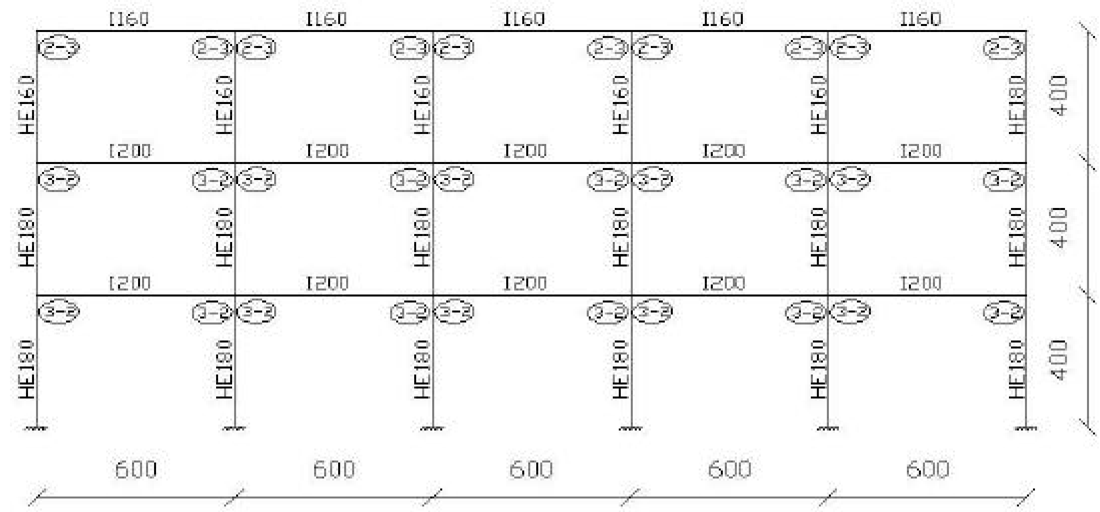


Figure 5-21: Semi Rigid Connection Types (Form 4)

Table 5-4: Data for Performance Analysis for Case Study 1-Form 4

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 400 kg

Number of Bays : 5 **P-Delta effect :** Yes

Top Disp. *(meter)*	***B. Shear*** *(ton)*	***W_1*** *(ton)*	***W_2*** *(ton)*	***W_3*** *(ton)*	f_1	f_2	f_3	***PF1***	α_1	***S_d*** *(meter)*	***S_a*** *(meter/s²)*
0,0000	0,0000	10,478	10,478	5,6080				0,0000	0,0000	0,0000	0,0000
0,0048	0,4000	10,478	10,478	5,6080	0,2645	0,7780	1,0000	1,3034	0,8112	0,0036	0,0186
0,0095	0,8000	10,478	10,478	5,6080	0,3722	0,7909	1,0000	1,3071	0,8756	0,0073	0,0344
0,0143	1,2000	10,478	10,478	5,6080	0,3771	0,7954	1,0000	1,3035	0,8780	0,0110	0,0514
0,0192	1,6000	10,478	10,478	5,6080	0,3794	0,7977	1,0000	1,3017	0,8792	0,0147	0,0685
0,0241	2,0000	10,478	10,478	5,6080	0,3807	0,7991	1,0000	1,3005	0,8798	0,0185	0,0856
0,0291	2,4000	10,478	10,478	5,6080	0,3815	0,8001	1,0000	1,2997	0,8802	0,0224	0,1026
0,0342	2,8000	10,478	10,478	5,6080	0,3819	0,8008	1,0000	1,2992	0,8803	0,0263	0,1197
0,0394	3,2000	10,478	10,478	5,6080	0,3821	0,8013	1,0000	1,2987	0,8804	0,0303	0,1368
0,0448	3,6000	10,478	10,478	5,6080	0,3821	0,8018	1,0000	1,2983	0,8804	0,0345	0,1539
0,0503	4,0000	10,478	10,478	5,6080	0,3819	0,8021	1,0000	1,2981	0,8803	0,0388	0,1711
0,0561	4,4000	10,478	10,478	5,6080	0,3816	0,8023	1,0000	1,2979	0,8801	0,0432	0,1882
0,0621	4,8000	10,478	10,478	5,6080	0,3811	0,8025	1,0000	1,2978	0,8799	0,0478	0,2054
0,0683	5,2000	10,478	10,478	5,6080	0,3805	0,8025	1,0000	1,2977	0,8795	0,0527	0,2226
0,0749	5,6000	10,478	10,478	5,6080	0,3797	0,8025	1,0000	1,2978	0,8791	0,0577	0,2398
0,0817	6,0000	10,478	10,478	5,6080	0,3788	0,8022	1,0000	1,2979	0,8787	0,0630	0,2571
0,0890	6,4000	10,478	10,478	5,6080	0,3777	0,8019	1,0000	1,2982	0,8781	0,0685	0,2744
0,0966	6,8000	10,478	10,478	5,6080	0,3764	0,8013	1,0000	1,2987	0,8774	0,0744	0,2917
0,1047	7,2000	10,478	10,478	5,6080	0,3749	0,8005	1,0000	1,2993	0,8767	0,0806	0,3092
0,1132	7,6000	10,478	10,478	5,6080	0,3732	0,7995	1,0000	1,3001	0,8758	0,0871	0,3267
0,1223	8,0000	10,478	10,478	5,6080	0,3712	0,7982	1,0000	1,3011	0,8748	0,0940	0,3443
0,1320	8,4000	10,478	10,478	5,6080	0,3690	0,7966	1,0000	1,3024	0,8737	0,1014	0,3619
0,1424	8,8000	10,478	10,478	5,6080	0,3664	0,7946	1,0000	1,3039	0,8724	0,1092	0,3797
0,1534	9,2000	10,478	10,478	5,6080	0,3636	0,7922	1,0000	1,3058	0,8710	0,1175	0,3976

With semi-rigid connections, again the exceeding of plastic moment capacity is the governing criteria. When compared with the rigid case, there is a considerable decrease in the S_a value with the increasing S_d (Figure 5-23). This difference is also due to the difference in the energy dissipation of two forms. Frame with semi-rigid connections dissipates nearly 20% more energy. Location of hinge(s) can be seen in Figure 5-22.

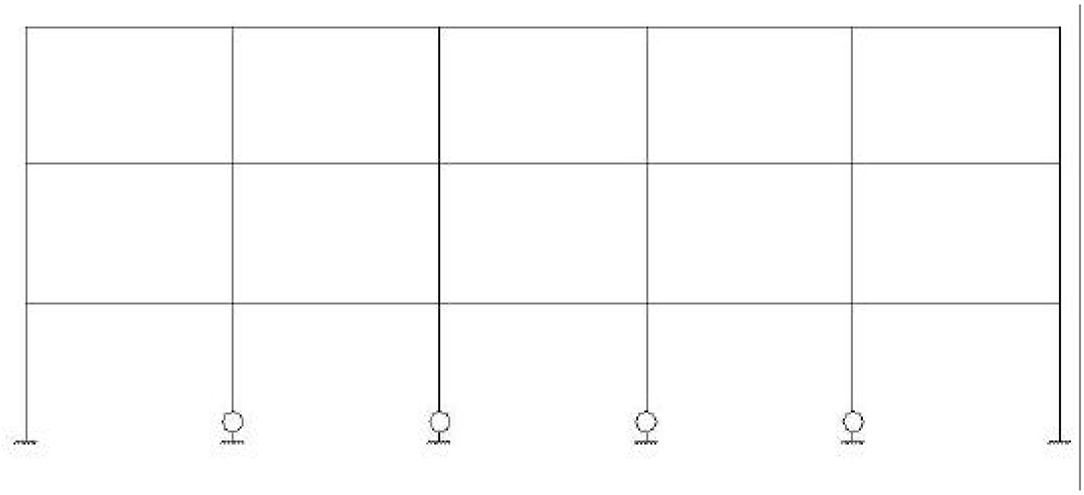


Figure 5-22: Hinge Formation for Form 4, Soil Type:Z1
(Base Shear: 9,2 t, Displacement: 15,3 cm)

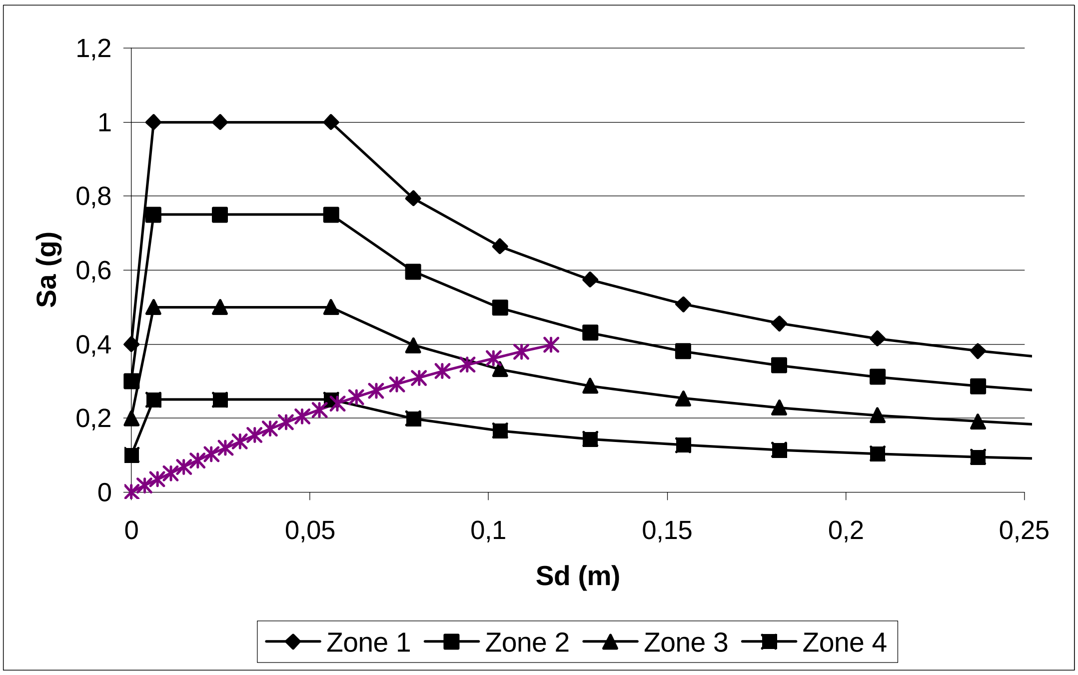


Figure 5-23: Capacity vs. Demand Spectra for Semi-Rigid Case (Form 4)

Performance of the frame for Soil Type 2, 3 ,4 can be found in Figure 5-24, Figure 5-25 and Figure 5-26 respectively.

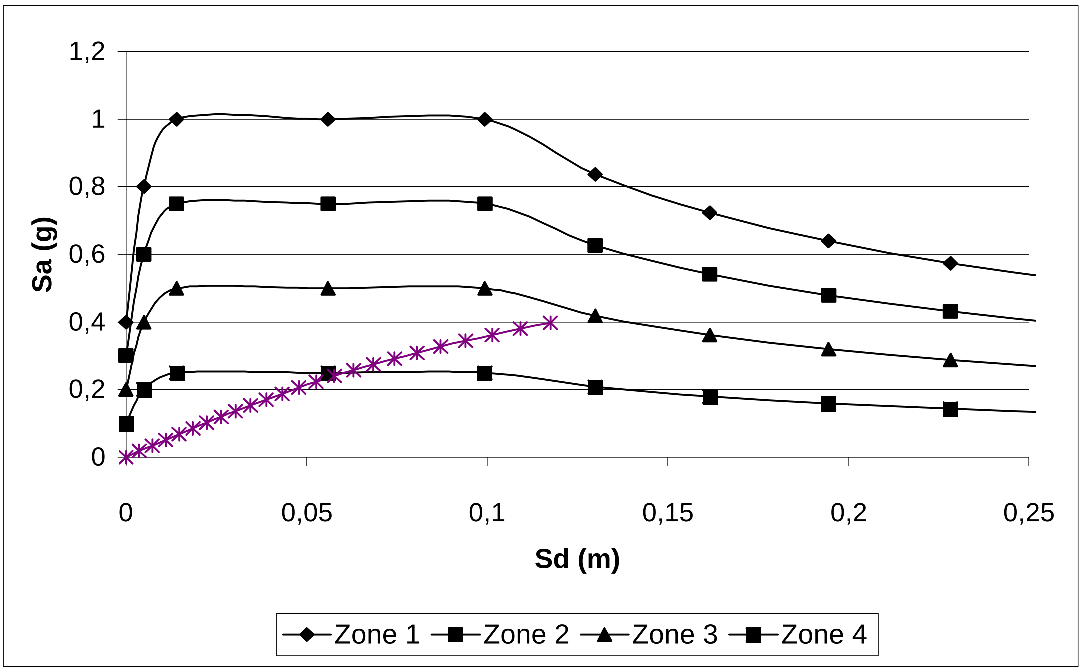


Figure 5-24: Capacity vs. Demand Spectra for Form 4, Soil Type:Z2

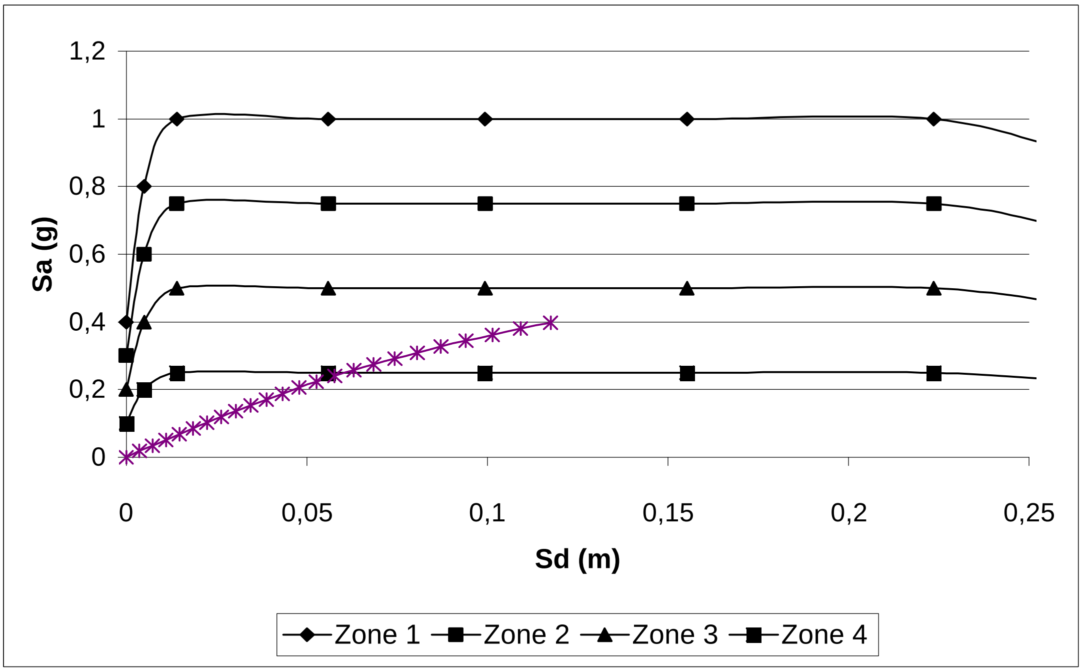


Figure 5-25: Capacity vs. Demand Spectra for Form 4, Soil Type:Z3

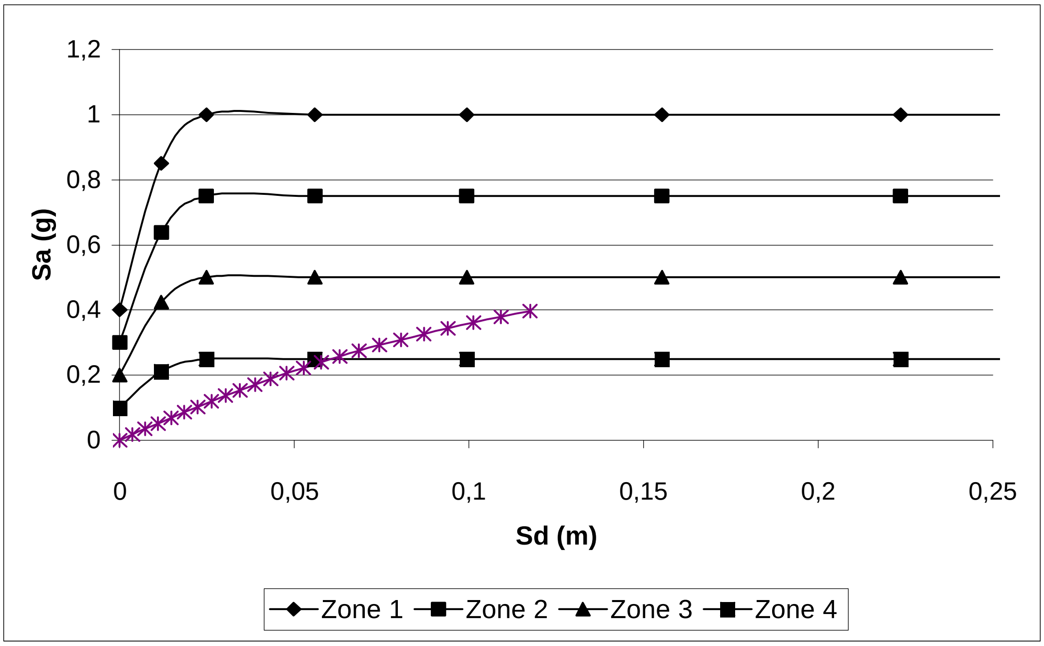


Figure 5-26: Capacity vs. Demand Spectra for Form 4, Soil Type:Z4

5.3.5 Case Study #1, Form 5

In case Study#1 Form 5, relatively weaker top and seat angle with double web angle connections were used. The location and types of connections are stated in Figure 5-8.

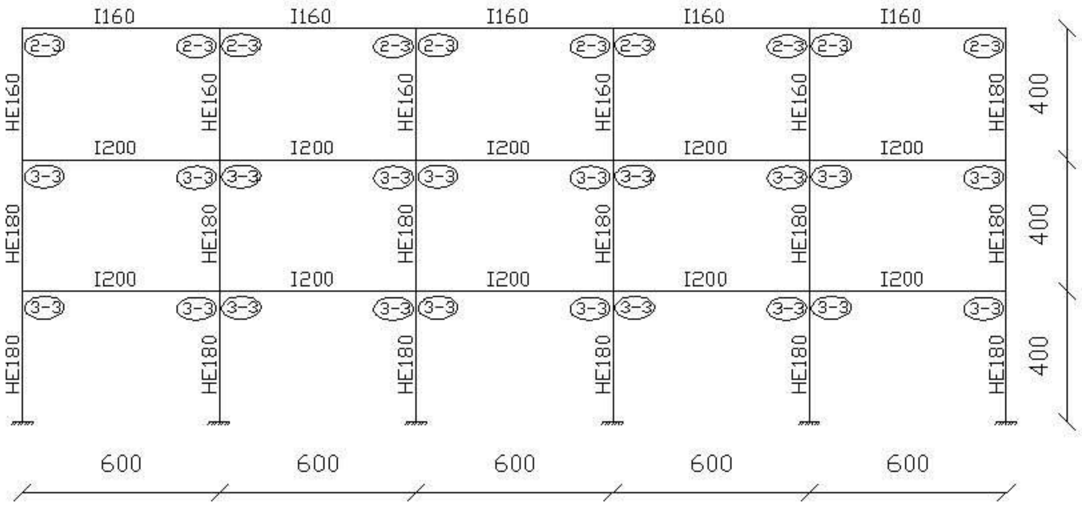


Figure 5-27: Semi Rigid Connection Types (Form 5)

Although the base column yielding was the governing criteria in the performance analysis, lateral sway value was approaching the limit state of 30 cm (Table 5-5). With the decrease in the global stiffness of frame, lateral drift value increases where S_a value is smaller than Form 4 when compared the performances in EQ Zone-3 (Figure 5-28). Location of hinge(s) can be seen in Figure 5-29.

Table 5-5: Data for Performance Analysis for Case Study 1-Form 5

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 400 kg

Number of Bays : 5 **P-Delta effect :** Yes

Top Disp. *(meter)*	**B. Shear** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	**PF1**	α_1	S_d *(meter)*	S_a *(meter/s^2)*
0,0000	0,0000	10,478	10,478	5,6080				0,0000	0,0000	0,0000	0,0000
0,0052	0,4000	10,478	10,478	5,6080	0,2645	0,7760	1,0000	1,3051	0,8112	0,0040	0,0186
0,0104	0,8000	10,478	10,478	5,6080	0,3644	0,7877	1,0000	1,3096	0,8716	0,0080	0,0346
0,0158	1,2000	10,478	10,478	5,6080	0,3686	0,7917	1,0000	1,3064	0,8737	0,0121	0,0517
0,0213	1,6000	10,478	10,478	5,6080	0,3704	0,7939	1,0000	1,3046	0,8746	0,0163	0,0689
0,0271	2,0000	10,478	10,478	5,6080	0,3711	0,7952	1,0000	1,3036	0,8749	0,0208	0,0861
0,0333	2,4000	10,478	10,478	5,6080	0,3710	0,7960	1,0000	1,3029	0,8748	0,0256	0,1033
0,0399	2,8000	10,478	10,478	5,6080	0,3704	0,7965	1,0000	1,3025	0,8745	0,0306	0,1205
0,0471	3,2000	10,478	10,478	5,6080	0,3693	0,7965	1,0000	1,3024	0,8739	0,0362	0,1379
0,0549	3,6000	10,478	10,478	5,6080	0,3676	0,7961	1,0000	1,3027	0,8730	0,0422	0,1552
0,0636	4,0000	10,478	10,478	5,6080	0,3652	0,7952	1,0000	1,3034	0,8718	0,0488	0,1727
0,0732	4,4000	10,478	10,478	5,6080	0,3623	0,7936	1,0000	1,3046	0,8702	0,0561	0,1903
0,0838	4,8000	10,478	10,478	5,6080	0,3587	0,7913	1,0000	1,3063	0,8684	0,0642	0,2081
0,0957	5,2000	10,478	10,478	5,6080	0,3544	0,7882	1,0000	1,3086	0,8661	0,0732	0,2260
0,1091	5,6000	10,478	10,478	5,6080	0,3495	0,7843	1,0000	1,3116	0,8635	0,0832	0,2441
0,1243	6,0000	10,478	10,478	5,6080	0,3439	0,7796	1,0000	1,3152	0,8604	0,0945	0,2625
0,1415	6,4000	10,478	10,478	5,6080	0,3376	0,7737	1,0000	1,3195	0,8570	0,1072	0,2811
0,1612	6,8000	10,478	10,478	5,6080	0,3305	0,7667	1,0000	1,3247	0,8530	0,1217	0,3001
0,1842	7,2000	10,478	10,478	5,6080	0,3225	0,7583	1,0000	1,3309	0,8484	0,1384	0,3195
0,2114	7,6000	10,478	10,478	5,6080	0,3135	0,7480	1,0000	1,3384	0,8429	0,1580	0,3394

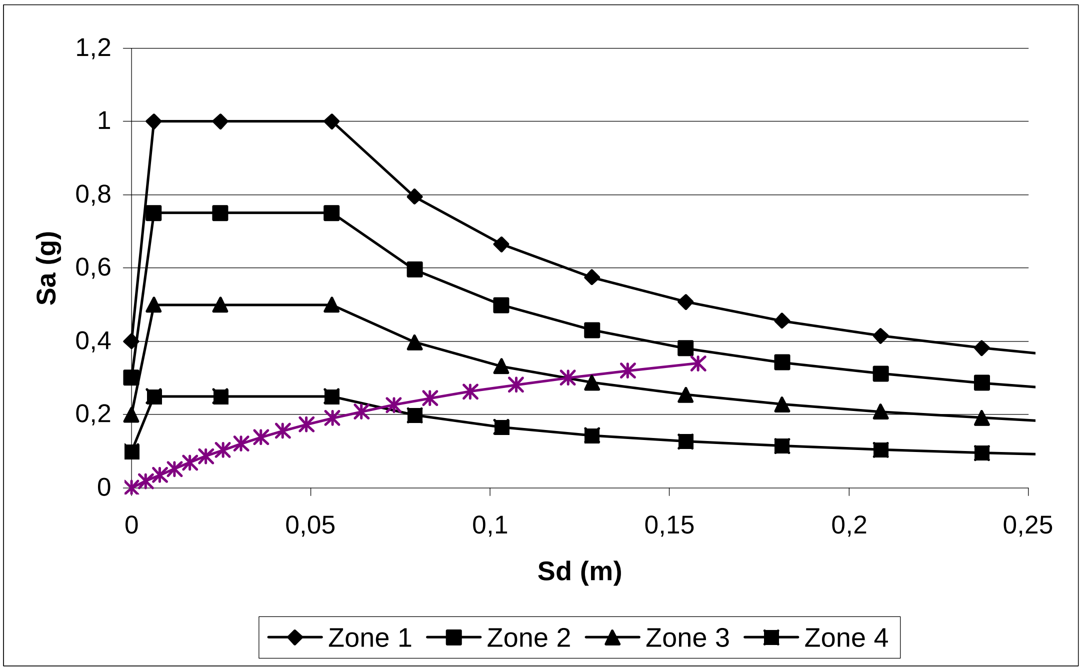


Figure 5-28: Capacity vs. Demand Spectra for Semi-Rigid Case (Form 5)

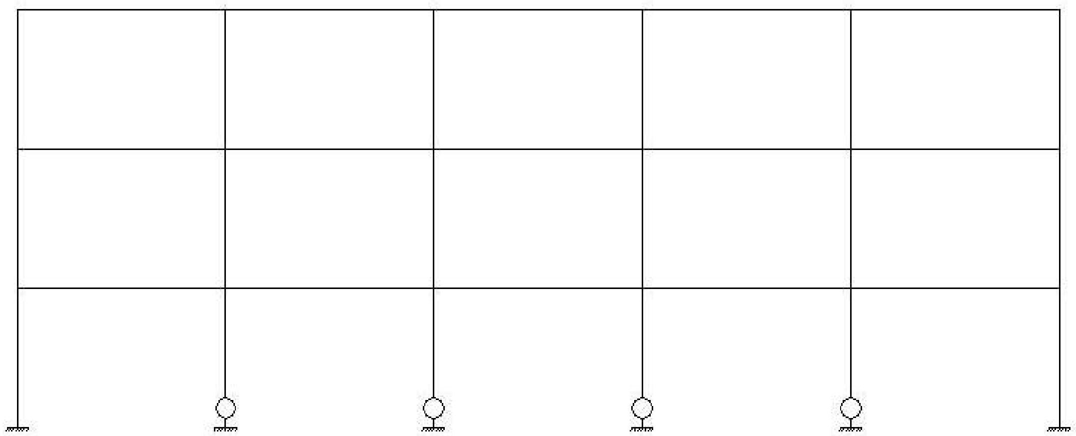


Figure 5-29: Hinge Formation for Form 5, Soil Type:Z1

(Base Shear: 7,6 t, Displacement: 21,1 cm)

Performance of the frame for Soil Type 2, 3 ,4 can be found in Figure 5-30, Figure 5-31 and Figure 5-32 respectively.

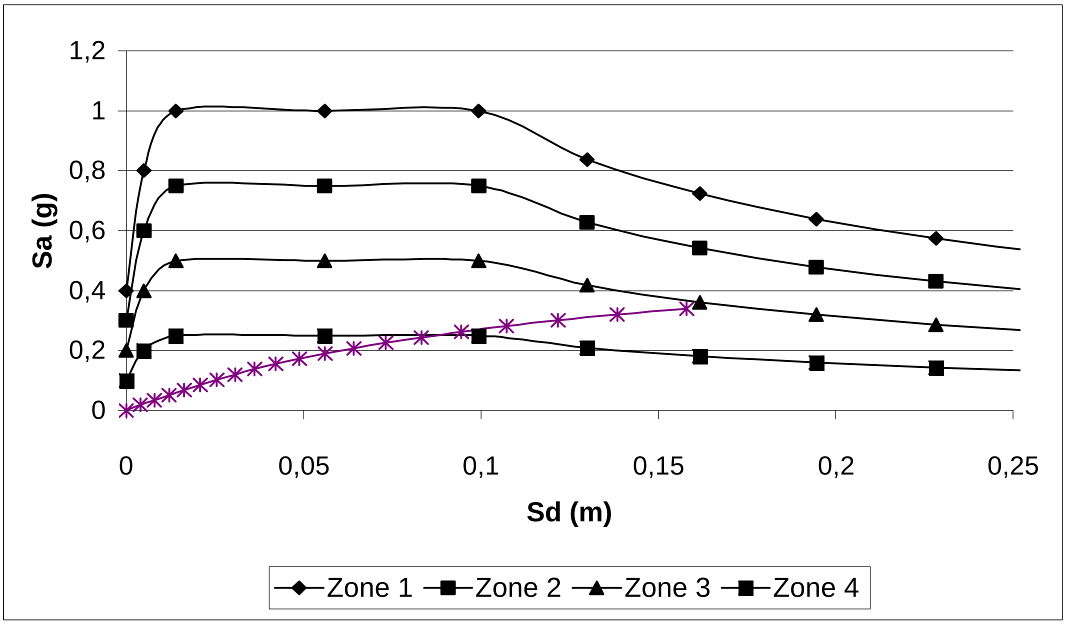


Figure 5-30: Capacity vs. Demand Spectra for Form 5, Soil Type:Z2

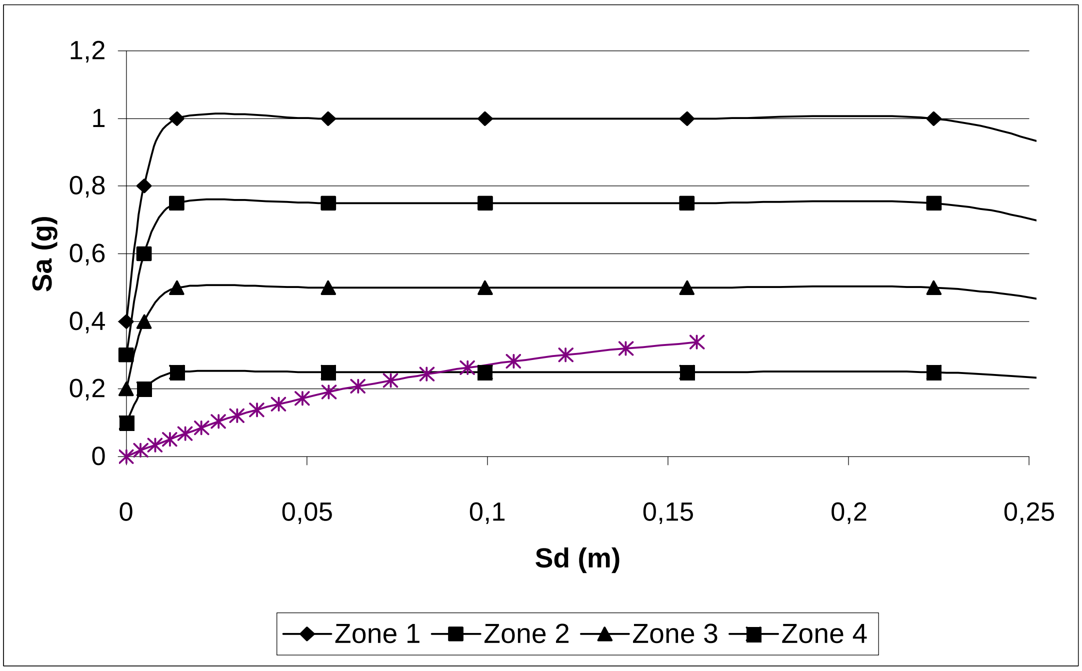


Figure 5-31: Capacity vs. Demand Spectra for Form 5, Soil Type:Z3

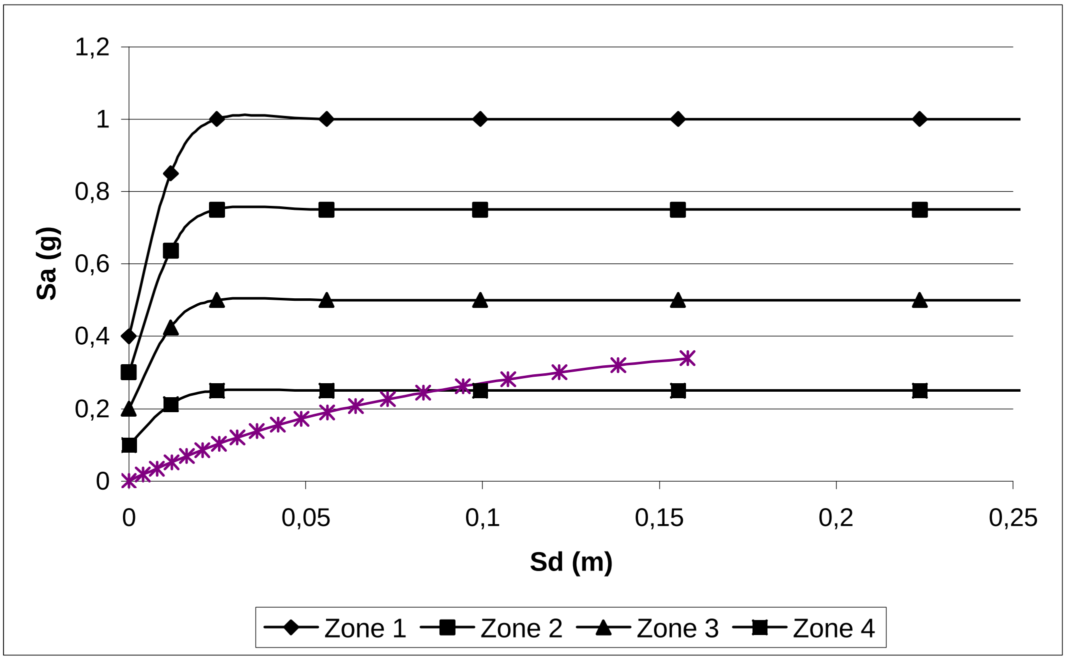


Figure 5-32: Capacity vs. Demand Spectra for Form 5, Soil Type:Z4

5.3.6 Case Study #1, Form 6

In Form 6, all connections were considered as double web angle connections. Types and location of these connections can be seen in Figure 5-33. As expected from the ultimate moment capacity of double web angle type connections, the frame is relatively soft and weak to resist lateral loads. According to Table 5-6, it can be said that drift limit is the governing parameter in construction of capacity spectrum. Location of hinges can be seen in Figure 5-35.

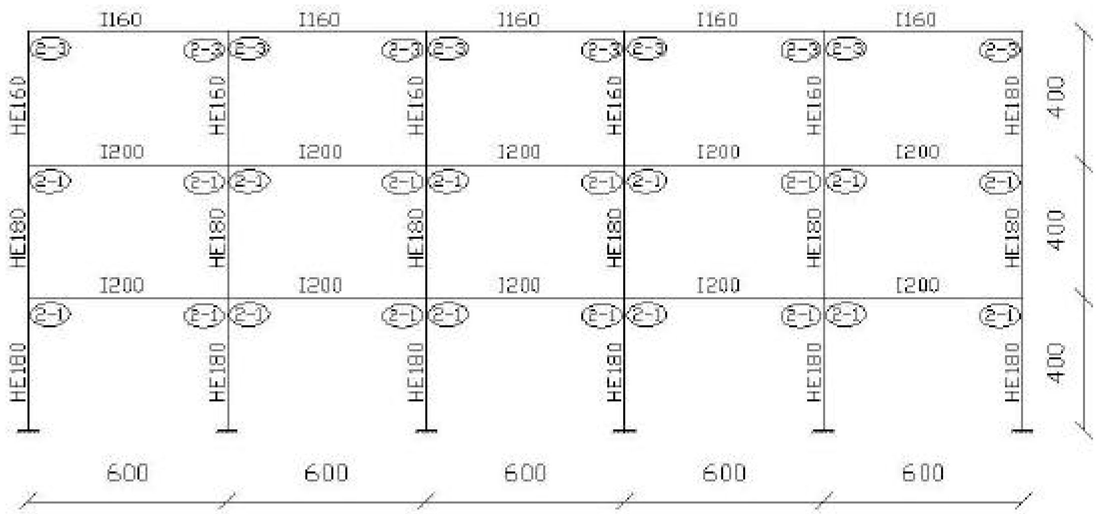


Figure 5-33: Semi Rigid Connection Types (Form 6)

Table 5-6: Data for Performance Analysis for Case Study 1-Form 6

Number of Floors : 3			**Connections :**	Semi-Rigid		**Load Increment :**	400 kg				
Number of Bays : 5			**P-Delta effect :**	Yes							
Top Disp. *(meter)*	***B. Shear*** *(ton)*	***W_1*** *(ton)*	***W_2*** *(ton)*	***W_3*** *(ton)*	f_1	f_2	f_3	***PF1***	α_1	***S_d*** *(meter)*	***S_a*** *(meter/s^2)*
0,0000	0,0000	10,478	10,478	5,6080				0,0000	0,0000	0,0000	0,0000
0,0084	0,4000	10,478	10,478	5,6080	0,2645	0,7568	1,0000	1,3214	0,8113	0,0064	0,0186
0,0171	0,8000	10,478	10,478	5,6080	0,3240	0,7631	1,0000	1,3270	0,8492	0,0129	0,0355
0,0262	1,2000	10,478	10,478	5,6080	0,3260	0,7650	1,0000	1,3257	0,8504	0,0197	0,0531
0,0362	1,6000	10,478	10,478	5,6080	0,3260	0,7655	1,0000	1,3253	0,8503	0,0273	0,0708
0,0478	2,0000	10,478	10,478	5,6080	0,3242	0,7647	1,0000	1,3257	0,8493	0,0361	0,0887
0,0620	2,4000	10,478	10,478	5,6080	0,3204	0,7621	1,0000	1,3275	0,8470	0,0467	0,1067
0,0803	2,8000	10,478	10,478	5,6080	0,3140	0,7570	1,0000	1,3309	0,8432	0,0603	0,1250
0,1104	3,2000	10,478	10,478	5,6080	0,3032	0,7478	1,0000	1,3370	0,8366	0,0826	0,1440
0,1754	3,6000	10,478	10,478	5,6080	0,2759	0,7179	1,0000	1,3570	0,8184	0,1292	0,1656
0,2981	4,0000	10,478	10,478	5,6080	0,2444	0,6746	1,0000	1,3849	0,7944	0,2153	0,1896

Different from the previous forms, in Form 6, yield mechanisms occurred in most of the connections. This yielding started with the application of 7th lateral load increment. The effect of yielding on the capacity curve can be seen in Figure 5-34. Also different from the previous forms, capacity of frame in Form 6 managed to catch the demand curve only for the EQ Zone-3. Despite of its' high lateral drift, the energy dissipation of the frame is considerably smaller than the previous ones.

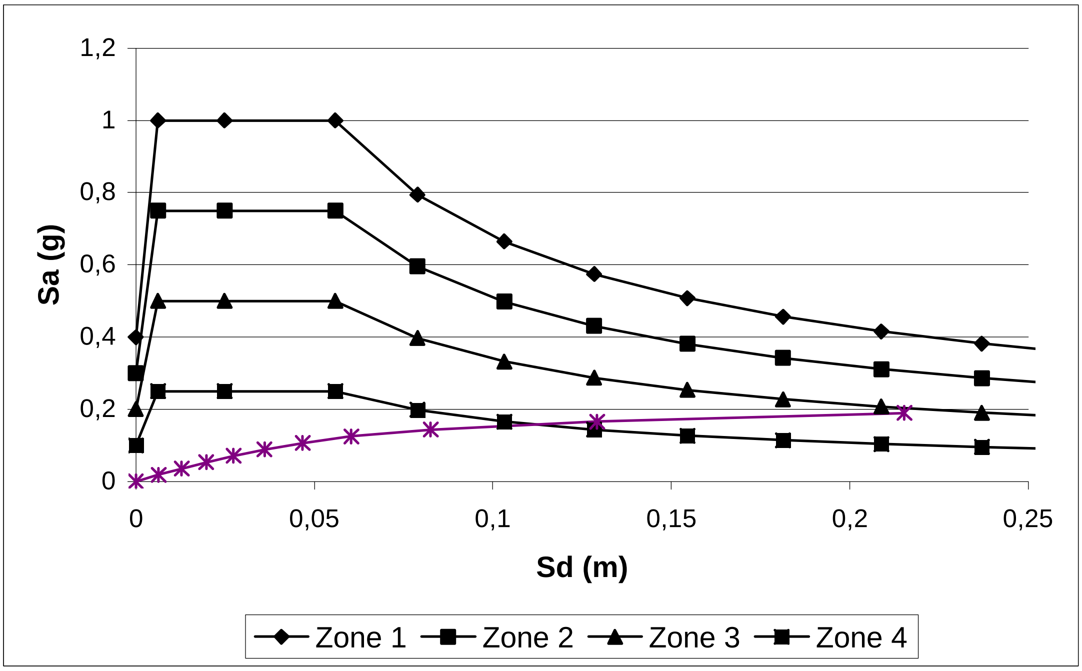


Figure 5-34: Capacity vs. Demand Spectra for Semi-Rigid Case (Form 6)

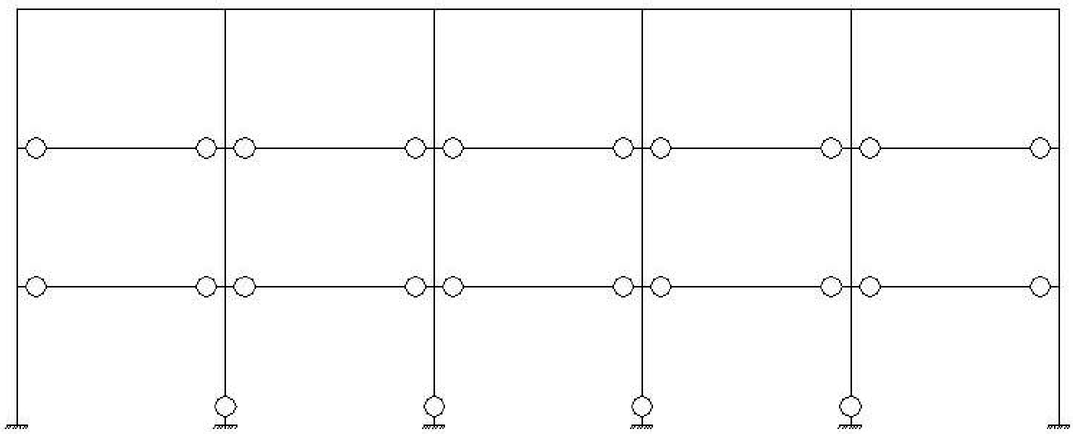


Figure 5-35: Hinge Formation for Form 6, Soil Type:1

(Base Shear: 4,0 t, Displacement: 29,8 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-36, Figure 5-37 and Figure 5-38 respectively.

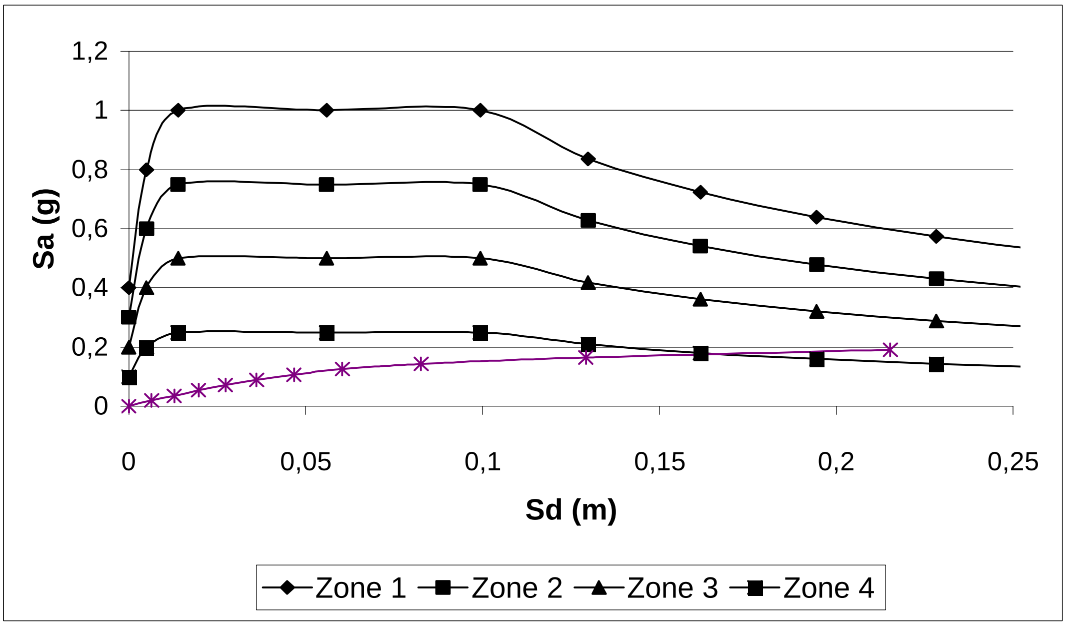


Figure 5-36: Capacity vs. Demand Spectra for Form 6, Soil Type:Z2

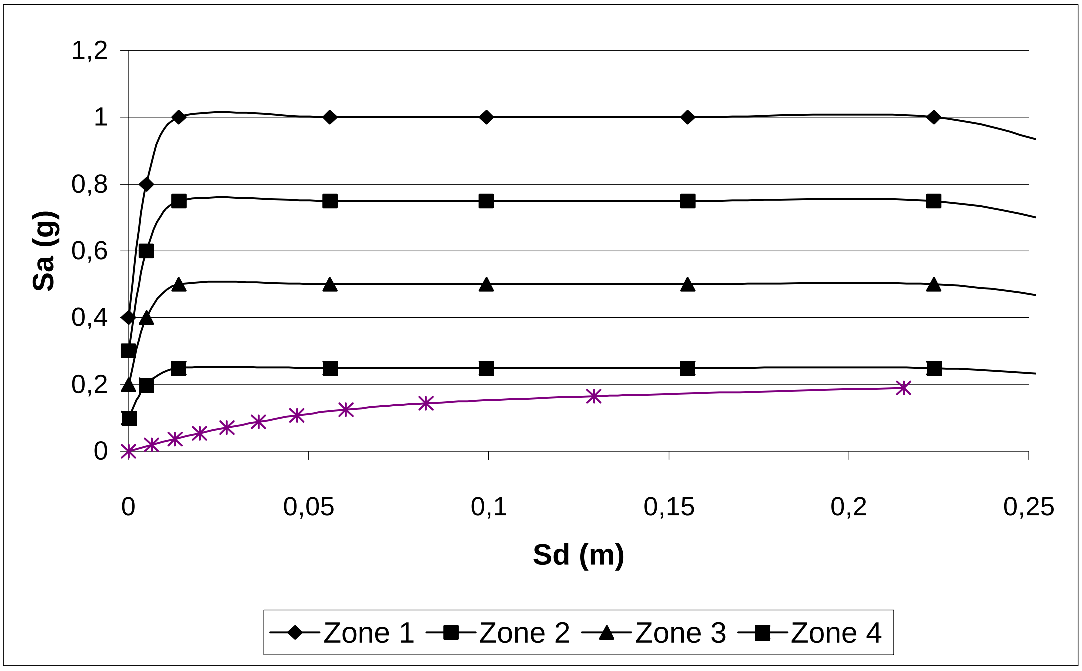


Figure 5-37: Capacity vs. Demand Spectra for Form 6, Soil Type:Z3

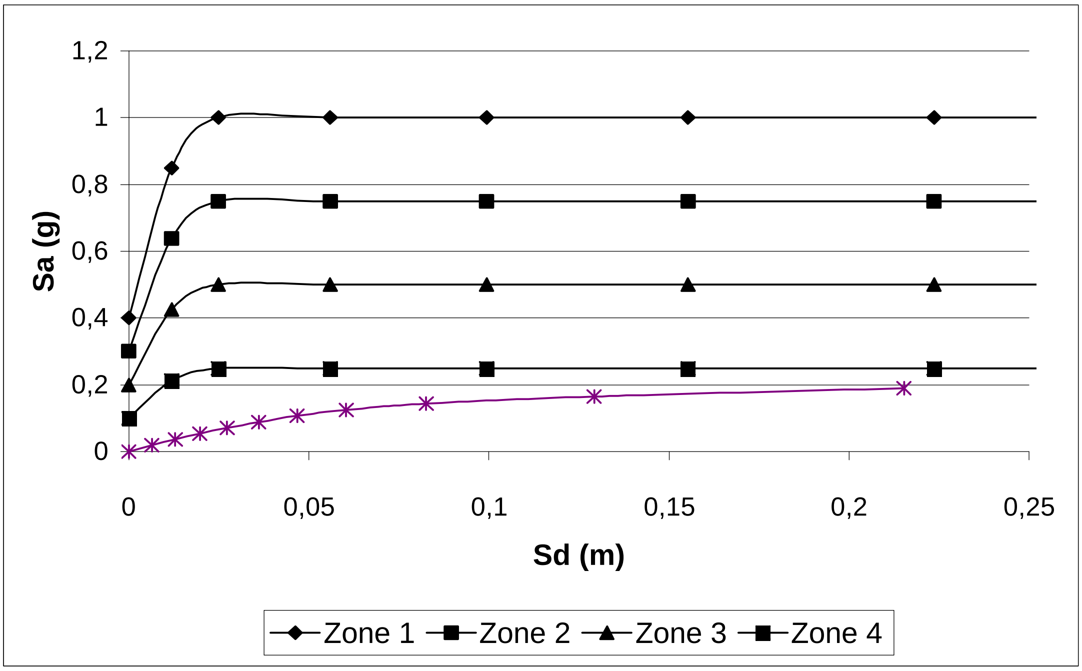


Figure 5-38: Capacity vs. Demand Spectra for Form 6, Soil Type:Z4

5.3.7 Case Study #1, Form 7

For the next form, Form 7, all connections of the first two floors were considered as top and seat angle connections. Types and location of these connections can be seen in Figure 5-39.

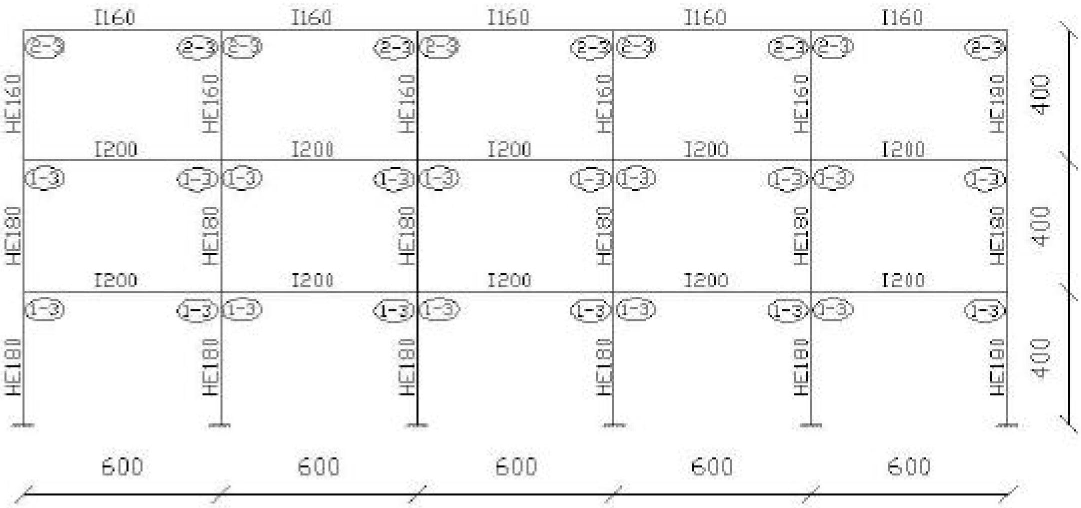


Figure 5-39 Semi Rigid Connection Types (Form 7)

Table 5-7: Data for Performance Analysis for Case Study 1-Form 7

Number of Floors : 3		**Connections :** Semi-Rigid			**Load Increment :** 400 kg						
Number of Bays : 5		**P-Delta effect :** Yes									
Top Disp. *(meter)*	***B. Shear*** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	***PF1***	α_1	S_d *(meter)*	S_a *(meter/s²)*
0,0000	0,0000	10,478	10,478	5,608				0,0000	0,0000	0,0000	0,0000
0,0047	0,4000	10,478	10,478	5,608	0,2645	0,7781	1,0000	1,3033	0,8112	0,0036	0,0186
0,0094	0,8000	10,478	10,478	5,608	0,3733	0,7911	1,0000	1,3070	0,8762	0,0072	0,0344
0,0141	1,2000	10,478	10,478	5,608	0,3783	0,7956	1,0000	1,3034	0,8786	0,0108	0,0514
0,0188	1,6000	10,478	10,478	5,608	0,3808	0,7978	1,0000	1,3016	0,8799	0,0144	0,0685
0,0235	2,0000	10,478	10,478	5,608	0,3823	0,7991	1,0000	1,3005	0,8806	0,0180	0,0855
0,0286	2,4000	10,478	10,478	5,608	0,3841	0,8018	1,0000	1,2983	0,8814	0,0220	0,1025
0,0337	2,8000	10,478	10,478	5,608	0,3853	0,8037	1,0000	1,2968	0,8820	0,0260	0,1195
0,0388	3,2000	10,478	10,478	5,608	0,3862	0,8050	1,0000	1,2957	0,8823	0,0300	0,1365
0,0440	3,6000	10,478	10,478	5,608	0,3868	0,8060	1,0000	1,2949	0,8826	0,0340	0,1535
0,0491	4,0000	10,478	10,478	5,608	0,3873	0,8068	1,0000	1,2943	0,8828	0,0380	0,1706
0,0546	4,4000	10,478	10,478	5,608	0,3858	0,8057	1,0000	1,2951	0,8821	0,0422	0,1878
0,0665	4,8000	10,478	10,478	5,608	0,3866	0,8126	1,0000	1,2896	0,8822	0,0516	0,2048
0,0801	5,2000	10,478	10,478	5,608	0,3875	0,8187	1,0000	1,2845	0,8824	0,0624	0,2218
0,0938	5,6000	10,478	10,478	5,608	0,3881	0,8229	1,0000	1,2811	0,8824	0,0733	0,2389
0,1352	6,0000	10,478	10,478	5,608	0,3481	0,7870	1,0000	1,3093	0,8626	0,1032	0,2619
0,2102	6,4000	10,478	10,478	5,608	0,3031	0,7401	1,0000	1,3435	0,8365	0,1564	0,2880

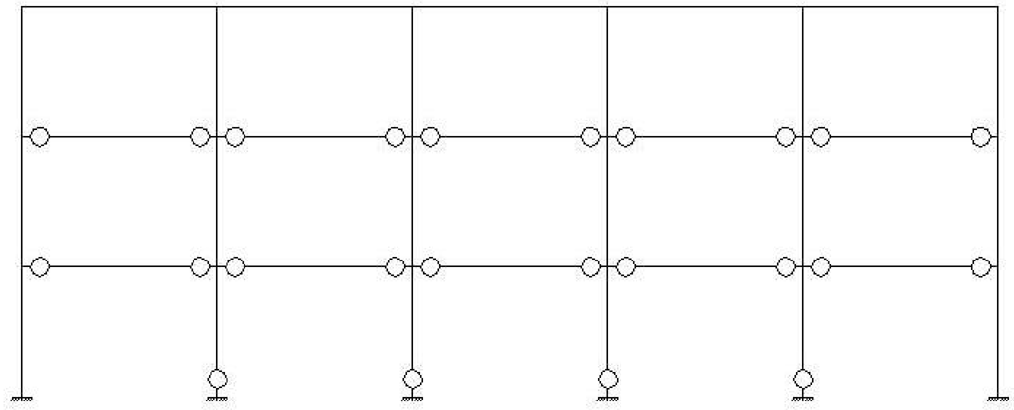


Figure 5-40: Hinge Formation for Form 7, Soil Type:Z1

(Base Shear: 4,0 t, Displacement: 29,8 cm)

Exceeding of the plastic moment capacities of base columns became the governing criteria in the construction of capacity spectrum. Although the sections of the frame are same for both Form 6 and Form 7, the frame with top and seat angle connections performs smaller drift values and also it can face with greater shear forces (Table 5-6). Basically this is due to the late yielding of connections. For this frame, connection yielding started at the 11th lateral load increment step as can be seen in Figure 5-41. Location of hinges for this form can be seen in Figure 5-40. Also the number of yielded connection is smaller than of Form 6.

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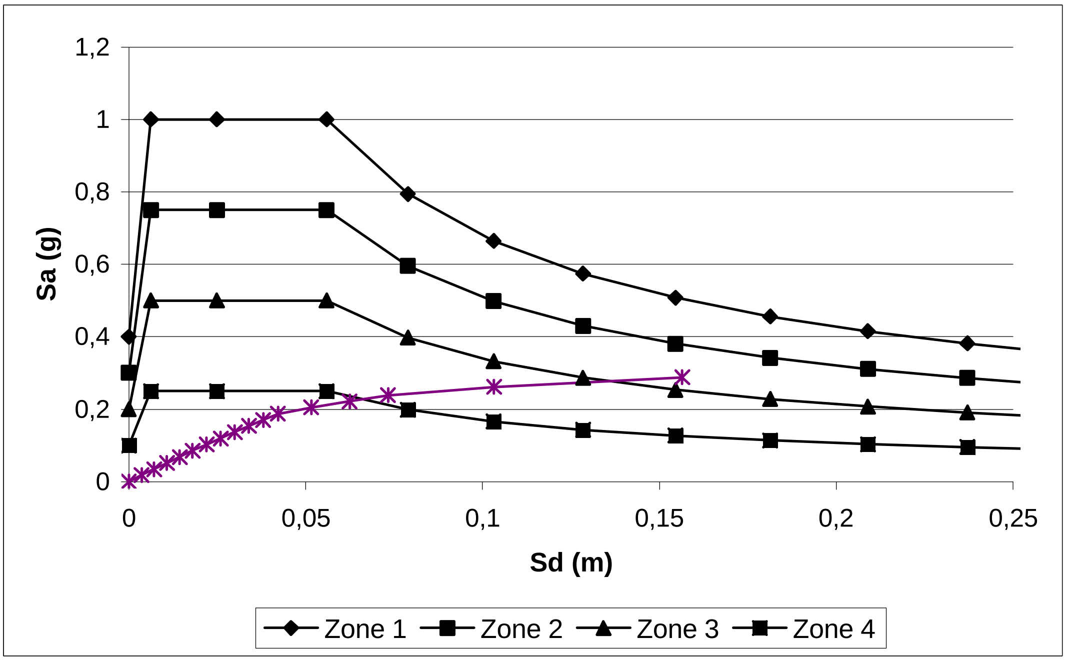


Figure 5-41: Capacity vs. Demand Spectra for Semi-Rigid Case (Form 7)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-42, Figure 5-43 and Figure 5-44 respectively.

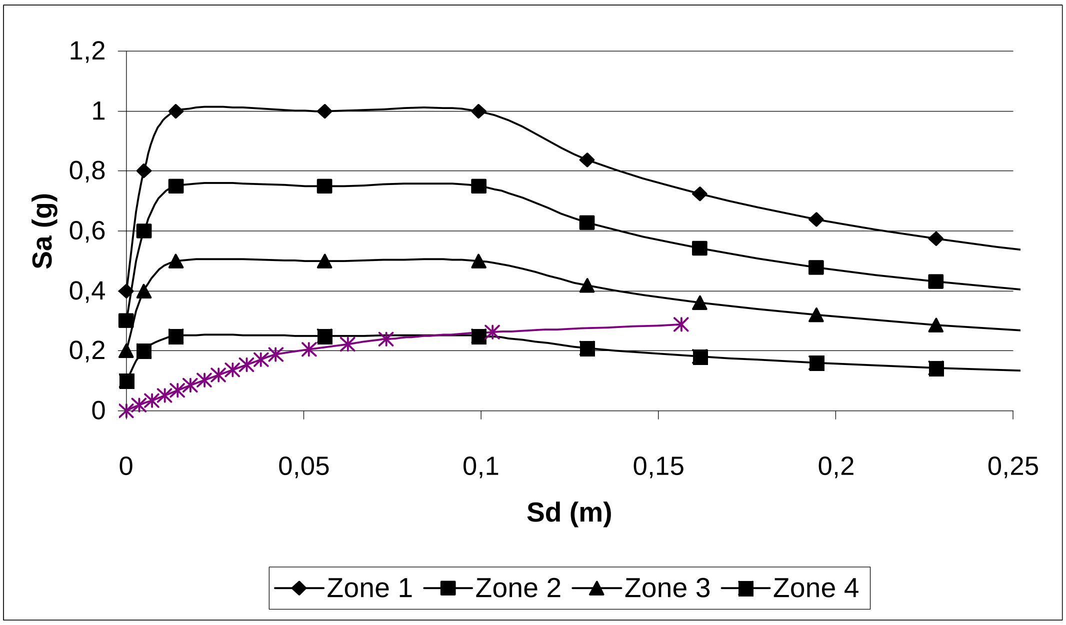


Figure 5-42: Capacity vs. Demand Spectra for Form 7, Soil Type:Z2

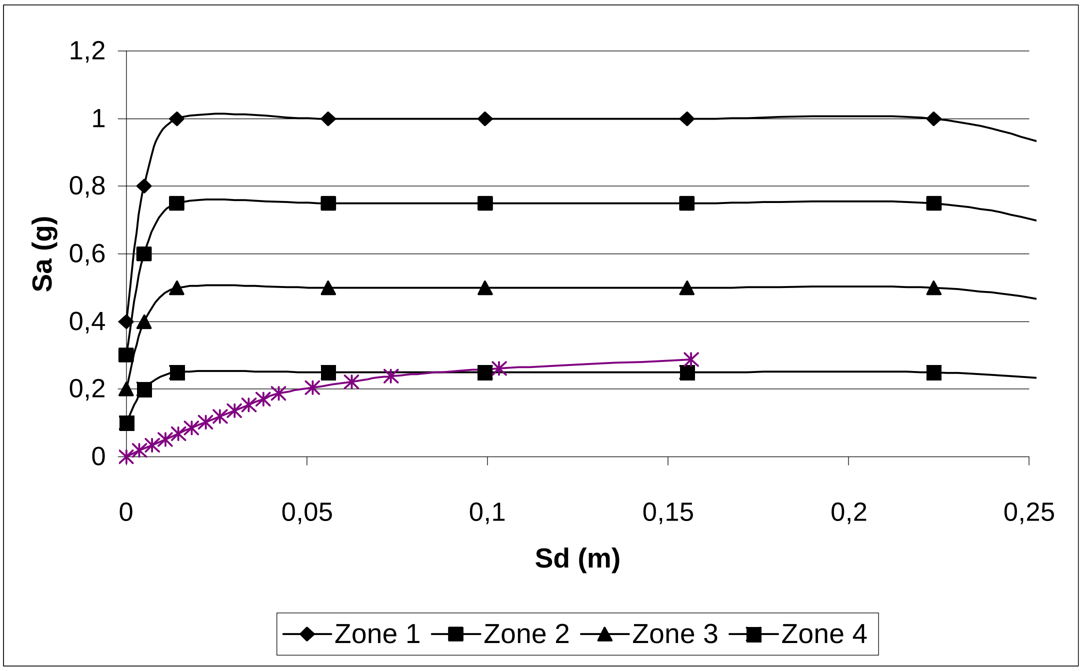


Figure 5-43: Capacity vs. Demand Spectra for Form 7, Soil Type:Z3

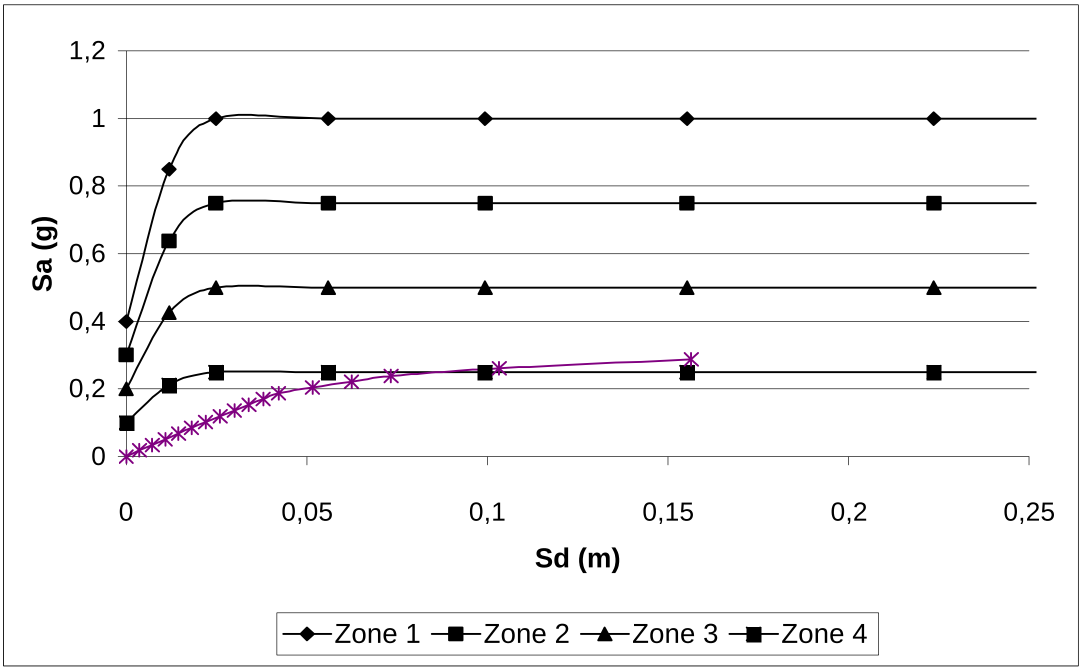


Figure 5-44: Capacity vs. Demand Spectra for Form 7, Soil Type:Z4

5.3.8 Case Study #1, Form 8

As last form of case study #1, combination of double web angle type connections and top and seat angle connections with double web angles was used in the connections of frame. Types and location of these connections can be seen in Figure 5-45. As expected, response of the frame lays between Form 7 and Form 4.

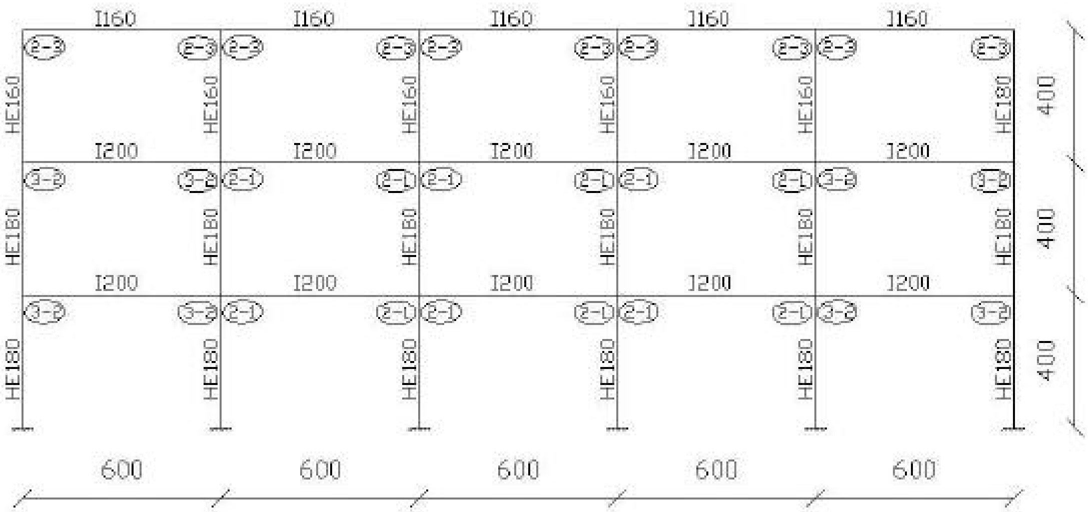


Figure 5-45 Semi Rigid Connection Types (Form 8)

As seen in Figure 5-47, case frame can resist to an earthquake that can occur in Zone-3. According to the analysis, some of the connections at the mid-joints yielded. But due to the strong connections located at the edge joints, this yielding cannot affect the capacity spectrum considerably.

Table 5-8: Data for Performance Analysis for Case Study 1-Form 8

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 400 kg

Number of Bays : 5 **P-Delta effect :** Yes

Top Disp. *(meter)*	***B. Shear*** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	***PF1***	α_1	S_d *(meter)*	S_a *(meter/s²)*
0,0000	0,0000	10,478	10,478	5,6080				0,0000	0,0000	0,0000	0,0000
0,0062	0,4000	10,478	10,478	5,6080	0,2645	0,7673	1,0000	1,3125	0,8113	0,0047	0,0186
0,0124	0,8000	10,478	10,478	5,6080	0,3491	0,7771	1,0000	1,3176	0,8635	0,0094	0,0349
0,0187	1,2000	10,478	10,478	5,6080	0,3526	0,7805	1,0000	1,3150	0,8653	0,0142	0,0522
0,0252	1,6000	10,478	10,478	5,6080	0,3541	0,7823	1,0000	1,3136	0,8661	0,0192	0,0695
0,0319	2,0000	10,478	10,478	5,6080	0,3547	0,7834	1,0000	1,3127	0,8664	0,0243	0,0869
0,0391	2,4000	10,478	10,478	5,6080	0,3547	0,7840	1,0000	1,3121	0,8664	0,0298	0,1043
0,0468	2,8000	10,478	10,478	5,6080	0,3541	0,7843	1,0000	1,3119	0,8661	0,0357	0,1217
0,0552	3,2000	10,478	10,478	5,6080	0,3530	0,7841	1,0000	1,3120	0,8654	0,0421	0,1392
0,0644	3,6000	10,478	10,478	5,6080	0,3512	0,7834	1,0000	1,3125	0,8645	0,0490	0,1568
0,0745	4,0000	10,478	10,478	5,6080	0,3488	0,7820	1,0000	1,3135	0,8632	0,0567	0,1744
0,0857	4,4000	10,478	10,478	5,6080	0,3458	0,7800	1,0000	1,3149	0,8615	0,0652	0,1923
0,0989	4,8000	10,478	10,478	5,6080	0,3422	0,7775	1,0000	1,3167	0,8595	0,0751	0,2102
0,1142	5,2000	10,478	10,478	5,6080	0,3377	0,7741	1,0000	1,3192	0,8570	0,0865	0,2284
0,1328	5,6000	10,478	10,478	5,6080	0,3306	0,7676	1,0000	1,3240	0,8530	0,1003	0,2471
0,1549	6,0000	10,478	10,478	5,6080	0,3221	0,7592	1,0000	1,3301	0,8481	0,1165	0,2663
0,1809	6,4000	10,478	10,478	5,6080	0,3129	0,7494	1,0000	1,3371	0,8426	0,1353	0,2859
0,2121	6,8000	10,478	10,478	5,6080	0,3028	0,7377	1,0000	1,3455	0,8363	0,1576	0,3061

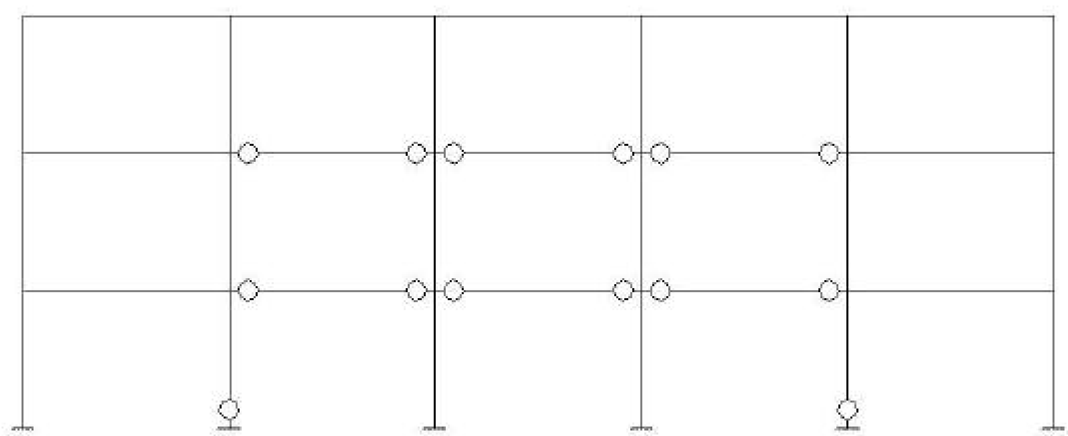


Figure 5-46: Hinge Formation for Form 8, Soil Type:Z1

(Base Shear: 6,8 t, Displacement: 21,2 cm)

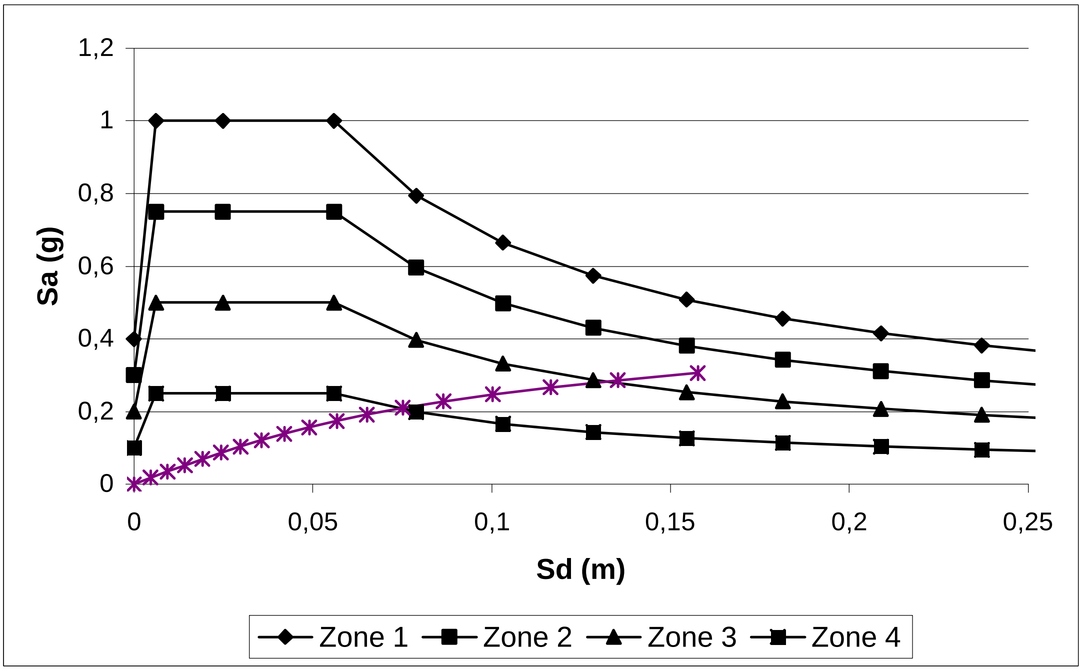


Figure 5-47: Capacity vs. Demand Spectra for Semi-Rigid Case (Form 8)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-48, Figure 5-49 and Figure 5-50 respectively.

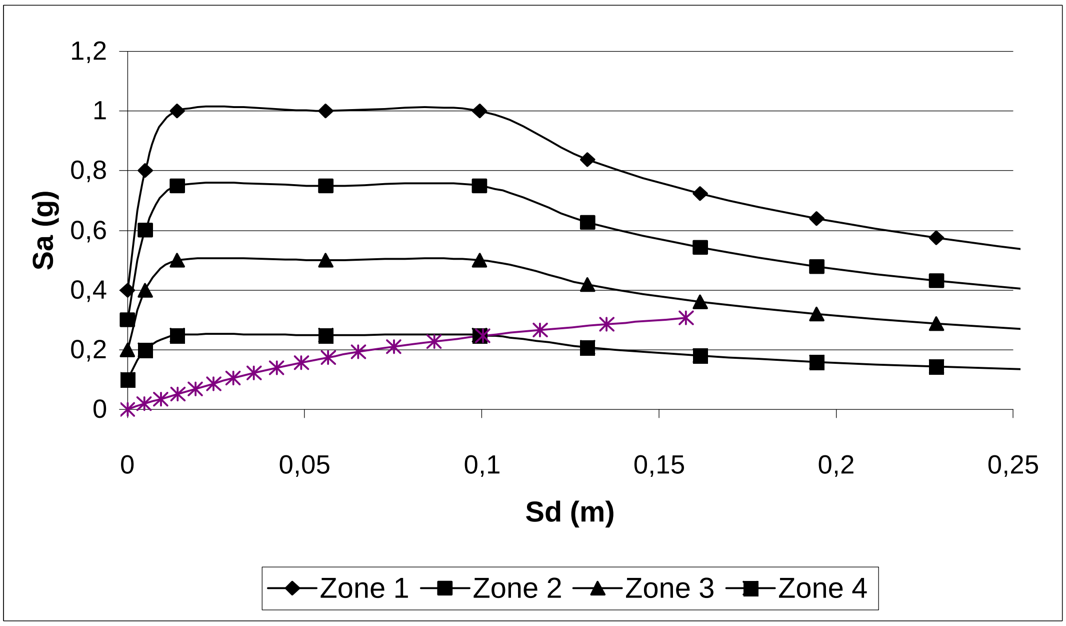


Figure 5-48: Capacity vs. Demand Spectra for Form 8, Soil Type:Z2

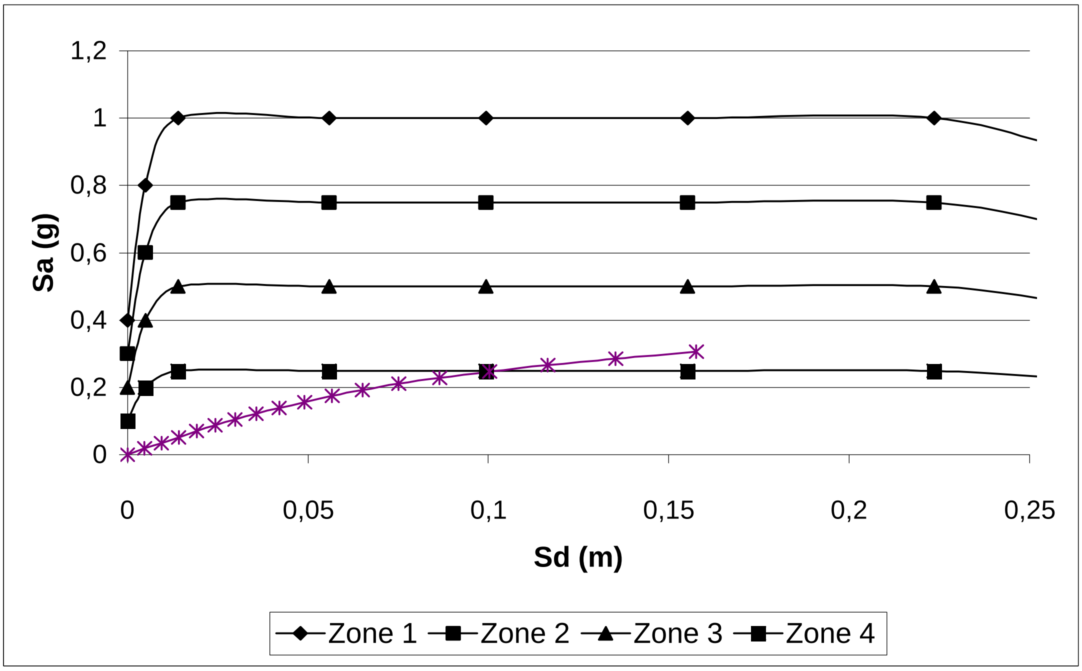


Figure 5-49: Capacity vs. Demand Spectra for Form 8, Soil Type:Z3

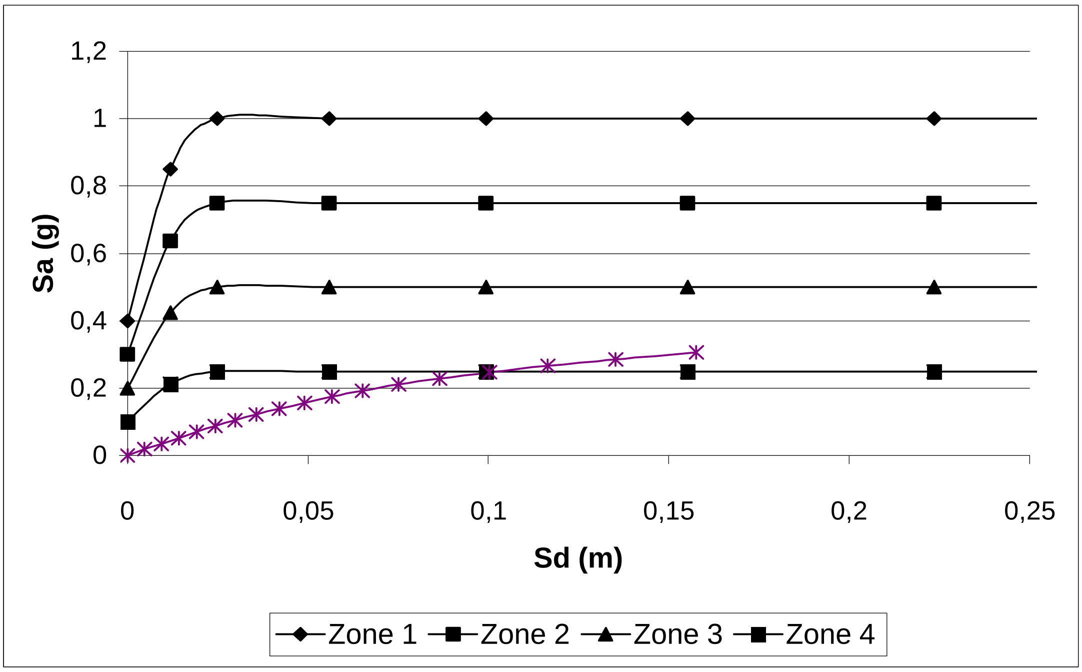


Figure 5-50: Capacity vs. Demand Spectra for Form 8, Soil Type:Z4

5.4 Performance Calculations for Case Study #2

5.4.1 Case Study #2, Form 1

Case Study #2 is the repetition of the forms in Case Study #1 for 3 bays. Only frame data, capacity calculation tables and capacity-demand graphs will be presented at this part. General evaluations of the performances of case Study #2 frames will be discussed within the final conclusion part of this study.

Table 5-9: Data for Performance Analysis for Case Study 2-Form 1

Number of Floors : 3 **Connections :** Fixed **Load Increment :** 400 kg

Number of Bays : 3 **P-Delta effect :** Yes

Top Disp. (meter)	B. Shear (ton)	W_1 (ton)	W_2 (ton)	W_3 (ton)	f_1	f_2	f_3	PF1	α_1	S_d (meter)	S_a (meter/s^2)
0,0000	0,0000	6,513	6,513	3,4810				0,0000	0,0000	0,0000	0,0000
0,0000	0,0000	6,513	6,513	3,4810	0,2645	0,1217	1,0000	1,4868	0,5402	0,0000	0,0000
0,0043	0,4000	6,513	6,513	3,4810	0,3718	0,7920	1,0000	1,3064	0,8754	0,0033	0,0277
0,0086	0,8000	6,513	6,513	3,4810	0,3814	0,8011	1,0000	1,2991	0,8800	0,0066	0,0551
0,0129	1,2000	6,513	6,513	3,4810	0,3846	0,8042	1,0000	1,2966	0,8816	0,0099	0,0825
0,0172	1,6000	6,513	6,513	3,4810	0,3862	0,8057	1,0000	1,2953	0,8823	0,0133	0,1099
0,0215	2,0000	6,513	6,513	3,4810	0,3872	0,8067	1,0000	1,2945	0,8828	0,0166	0,1372
0,0258	2,4000	6,513	6,513	3,4810	0,3879	0,8073	1,0000	1,2940	0,8831	0,0199	0,1646
0,0301	2,8000	6,513	6,513	3,4810	0,3884	0,8078	1,0000	1,2936	0,8833	0,0233	0,1920
0,0344	3,2000	6,513	6,513	3,4810	0,3887	0,8081	1,0000	1,2934	0,8835	0,0266	0,2194
0,0387	3,6000	6,513	6,513	3,4810	0,3890	0,8083	1,0000	1,2932	0,8836	0,0299	0,2468
0,0430	4,0000	6,513	6,513	3,4810	0,3892	0,8086	1,0000	1,2930	0,8837	0,0332	0,2742
0,0473	4,4000	6,513	6,513	3,4810	0,3894	0,8087	1,0000	1,2928	0,8838	0,0366	0,3016
0,0516	4,8000	6,513	6,513	3,4810	0,3895	0,8089	1,0000	1,2927	0,8839	0,0399	0,3290
0,0559	5,2000	6,513	6,513	3,4810	0,3897	0,8090	1,0000	1,2926	0,8839	0,0432	0,3564
0,0602	5,6000	6,513	6,513	3,4810	0,3898	0,8091	1,0000	1,2925	0,8840	0,0465	0,3838
0,0645	6,0000	6,513	6,513	3,4810	0,3899	0,8092	1,0000	1,2925	0,8840	0,0499	0,4112
0,0688	6,4000	6,513	6,513	3,4810	0,3899	0,8093	1,0000	1,2924	0,8841	0,0532	0,4386
0,0731	6,8000	6,513	6,513	3,4810	0,3900	0,8093	1,0000	1,2923	0,8841	0,0565	0,4660
0,0774	7,2000	6,513	6,513	3,4810	0,3901	0,8094	1,0000	1,2923	0,8841	0,0599	0,4934
0,0816	7,6000	6,513	6,513	3,4810	0,3901	0,8094	1,0000	1,2923	0,8841	0,0632	0,5207
0,0859	8,0000	6,513	6,513	3,4810	0,3902	0,8095	1,0000	1,2922	0,8842	0,0665	0,5481
0,0902	8,4000	6,513	6,513	3,4810	0,3902	0,8095	1,0000	1,2922	0,8842	0,0698	0,5755
0,0945	8,8000	6,513	6,513	3,4810	0,3903	0,8096	1,0000	1,2921	0,8842	0,0732	0,6029
0,0988	9,2000	6,513	6,513	3,4810	0,3903	0,8096	1,0000	1,2921	0,8842	0,0765	0,6303
0,1031	9,6000	6,513	6,513	3,4810	0,3903	0,8097	1,0000	1,2921	0,8842	0,0798	0,6577
0,1074	10,0000	6,513	6,513	3,4810	0,3904	0,8097	1,0000	1,2921	0,8843	0,0831	0,6851
0,1117	10,4000	6,513	6,513	3,4810	0,3904	0,8097	1,0000	1,2920	0,8843	0,0865	0,7125

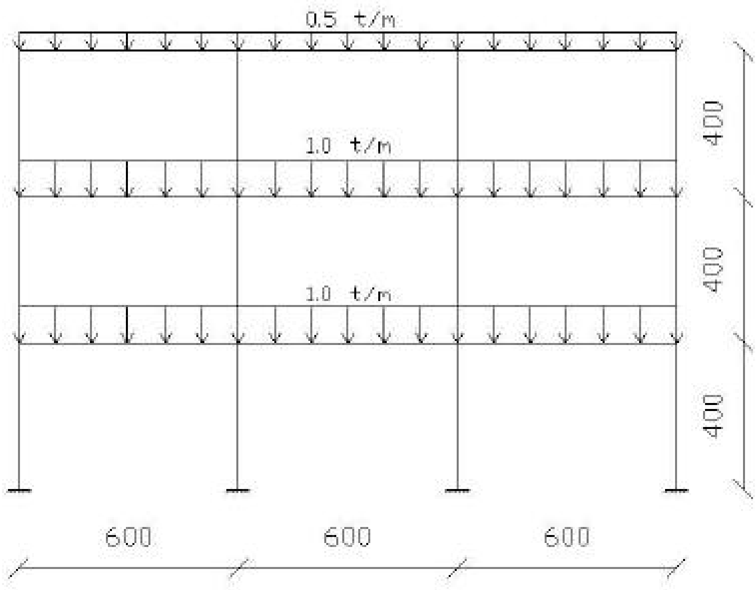


Figure 5-51: Loading and Geometry Data for Case Study #2

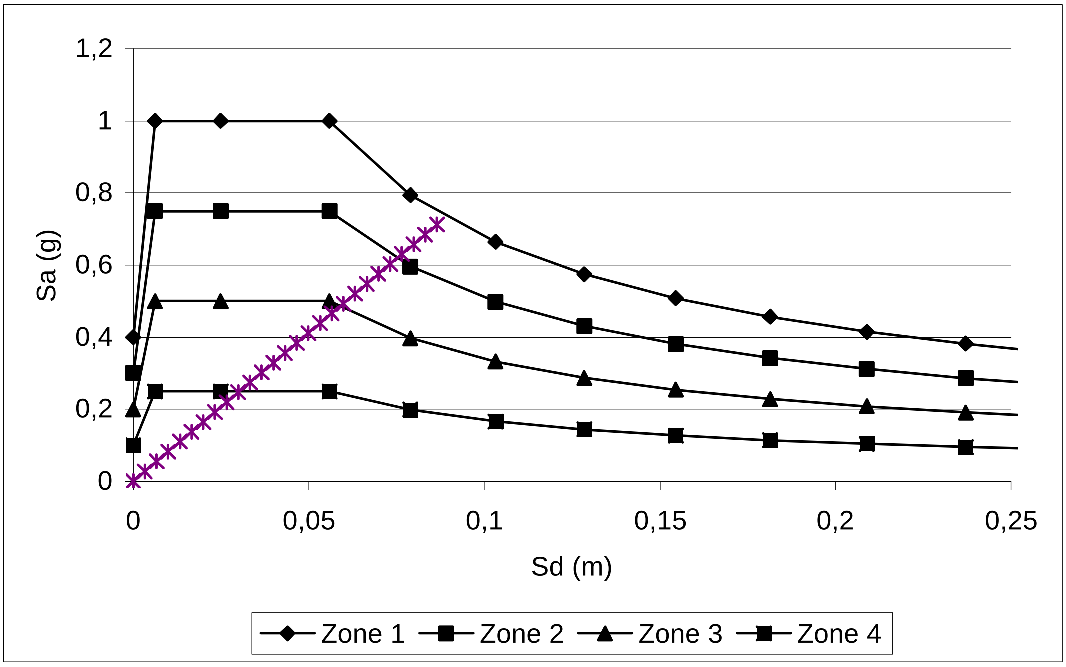


Figure 5-52: Capacity vs. Demand Spectra for Rigid Case 2 (Form 1)

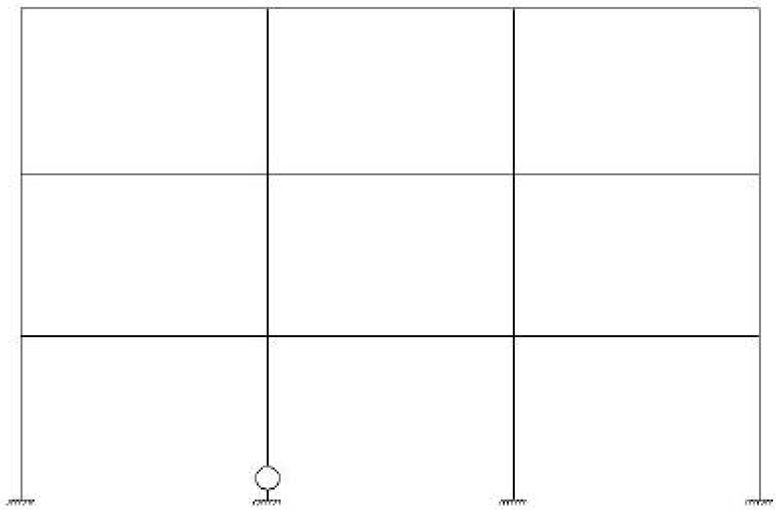


Figure 5-53: Hinge Formation for Form 1, Soil Type:Z1

(Base Shear: 10,4 t, Displacement: 11,2 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-54, Figure 5-55 and Figure 5-56 respectively.

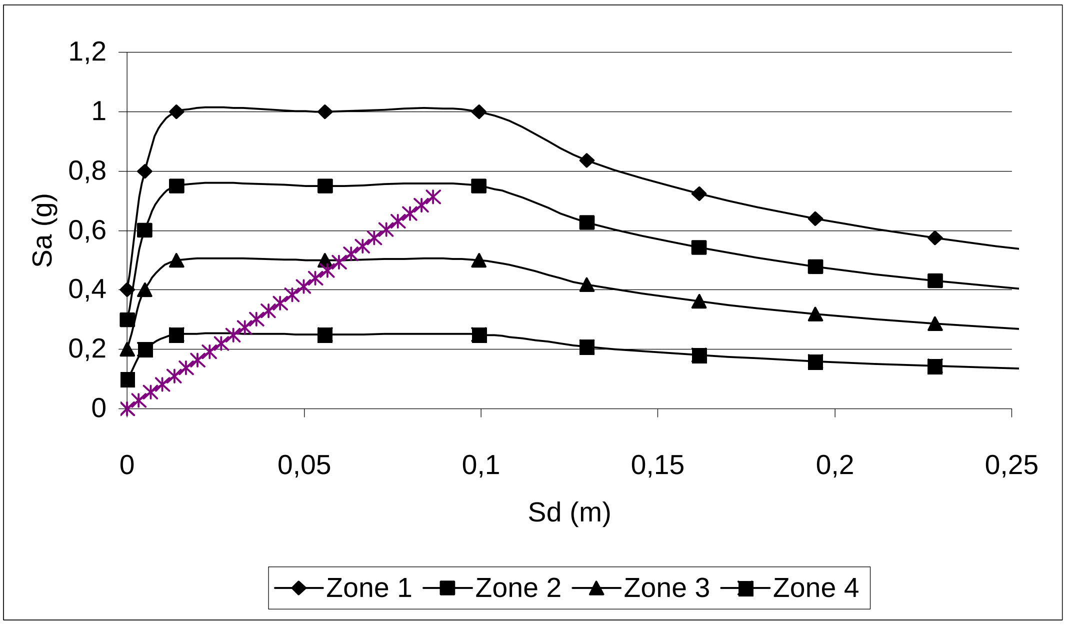


Figure 5-54: Capacity vs. Demand Spectra for Form 1, Soil Type:Z2

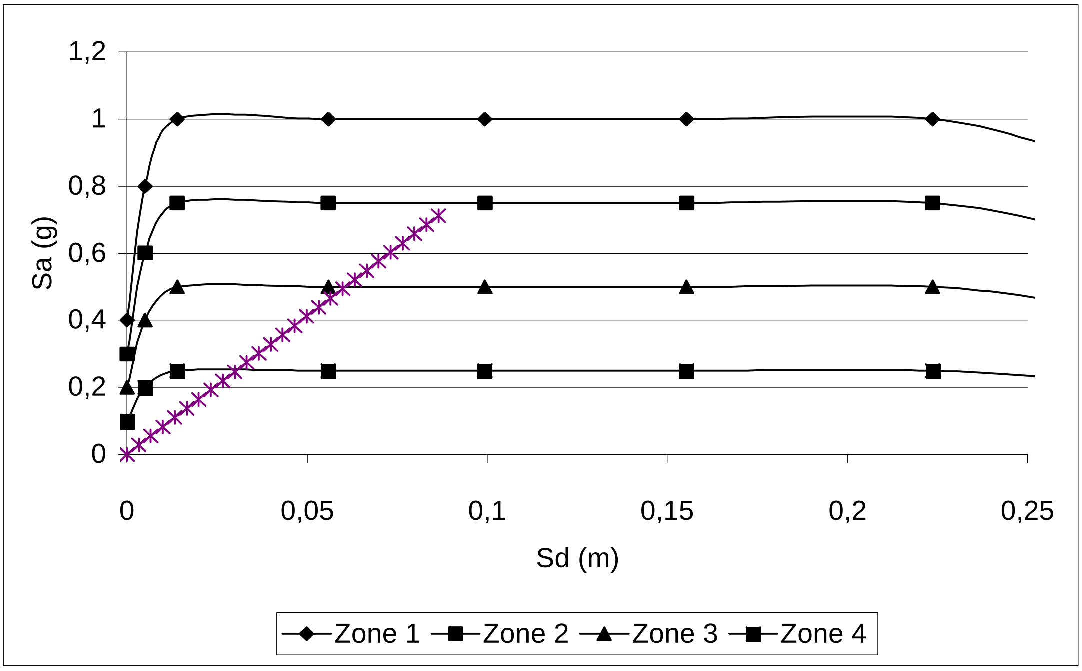


Figure 5-55: Capacity vs. Demand Spectra for Form 1, Soil Type:Z3

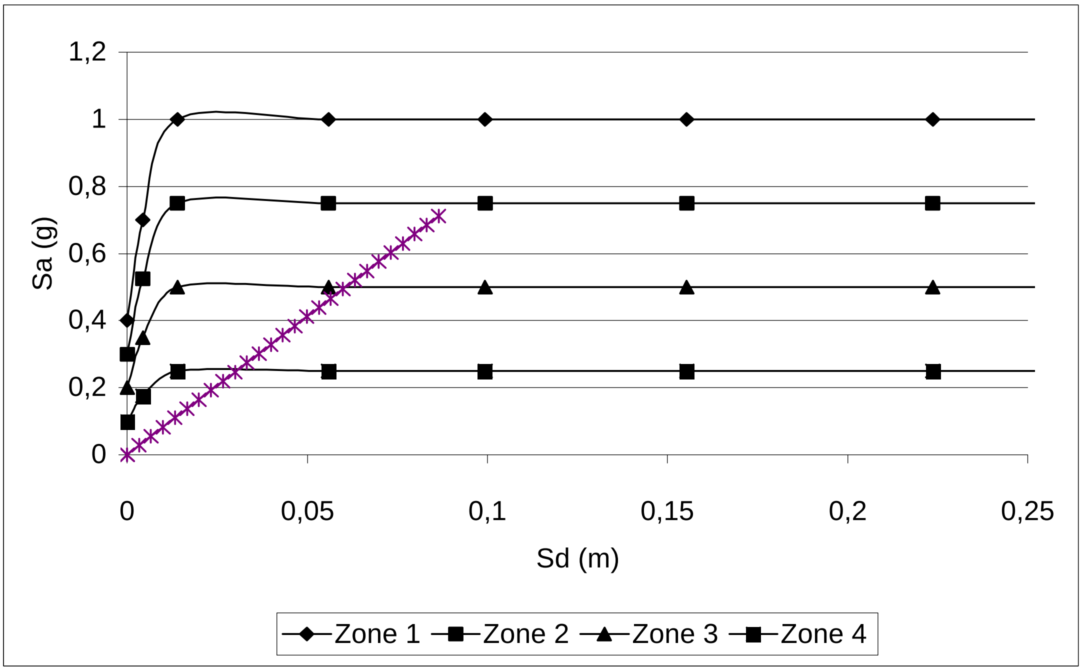


Figure 5-56: Capacity vs. Demand Spectra for Form 1, Soil Type:Z4

5.4.2 Case Study #2, Form 2

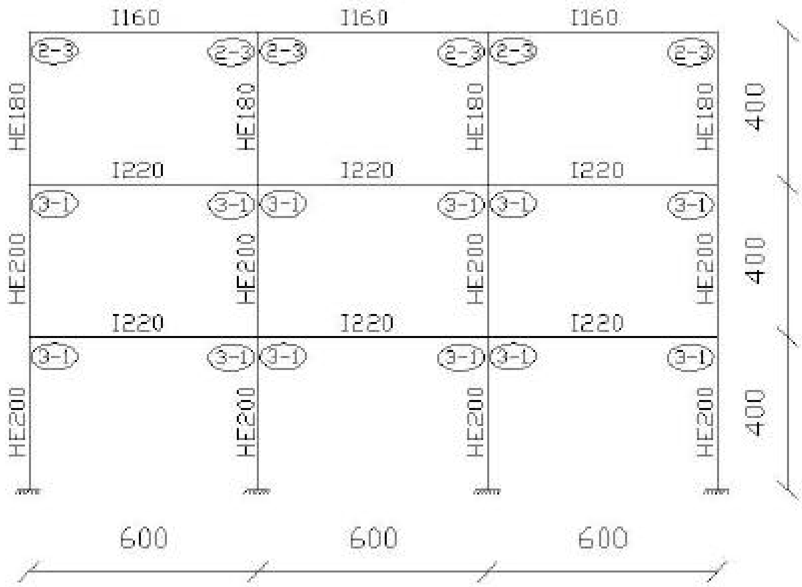


Figure 5-57 Semi Rigid Connection Types (Form 2)

Table 5-10: Data for Performance Analysis for Case Study 2-Form 2

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 400 kg

Number of Bays : 3 **P-Delta effect :** Yes

Top Disp. *(meter)*	***B. Shear*** *(ton)*	***W_1*** *(ton)*	***W_2*** *(ton)*	***W_3*** *(ton)*	f_1	f_2	f_3	***PF1***	α_1	***S_d*** *(meter)*	***S_a*** *(meter/s^2)*
0,00001	0,00000	6,513	6,513	3,481				0,00000	0,00000	0,00000	0,00000
0,00523	0,40000	6,513	6,513	3,481	0,34259	0,75316	1,00000	1,33724	0,86014	0,00391	0,02817
0,01049	0,80000	6,513	6,513	3,481	0,34999	0,76087	1,00000	1,33130	0,86425	0,00788	0,05608
0,01583	1,20000	6,513	6,513	3,481	0,35227	0,76348	1,00000	1,32925	0,86549	0,01191	0,08399
0,02129	1,60000	6,513	6,513	3,481	0,35316	0,76478	1,00000	1,32822	0,86596	0,01603	0,11193
0,02692	2,00000	6,513	6,513	3,481	0,35339	0,76551	1,00000	1,32762	0,86608	0,02028	0,13990
0,03275	2,40000	6,513	6,513	3,481	0,35320	0,76591	1,00000	1,32728	0,86597	0,02468	0,16790
0,03885	2,80000	6,513	6,513	3,481	0,35269	0,76607	1,00000	1,32713	0,86568	0,02927	0,19594
0,04526	3,20000	6,513	6,513	3,481	0,35187	0,76598	1,00000	1,32716	0,86523	0,03411	0,22405
0,05205	3,60000	6,513	6,513	3,481	0,35075	0,76562	1,00000	1,32739	0,86460	0,03921	0,25224
0,05928	4,00000	6,513	6,513	3,481	0,34930	0,76497	1,00000	1,32786	0,86380	0,04464	0,28053
0,06702	4,40000	6,513	6,513	3,481	0,34750	0,76394	1,00000	1,32860	0,86280	0,05044	0,30894
0,07533	4,80000	6,513	6,513	3,481	0,34531	0,76249	1,00000	1,32966	0,86159	0,05665	0,33750
0,08430	5,20000	6,513	6,513	3,481	0,34270	0,76054	1,00000	1,33111	0,86013	0,06333	0,36624
0,09403	5,60000	6,513	6,513	3,481	0,3396	0,75802	1,00000	1,33298	0,85841	0,07054	0,39521
0,10462	6,00000	6,513	6,513	3,481	0,3361	0,75487	1,00000	1,33533	0,85639	0,07834	0,42443
0,11618	6,40000	6,513	6,513	3,481	0,3321	0,75101	1,00000	1,33820	0,85406	0,08682	0,45397
0,12887	6,80000	6,513	6,513	3,481	0,3275	0,74640	1,00000	1,34161	0,85139	0,09605	0,48385
0,14284	7,20000	6,513	6,513	3,481	0,3224	0,74102	1,00000	1,34557	0,84836	0,10616	0,51414
0,15829	7,60000	6,513	6,513	3,481	0,3169	0,73489	1,00000	1,35006	0,84496	0,11725	0,54489
0,17541	8,00000	6,513	6,513	3,481	0,31091	0,72807	1,00000	1,35500	0,84121	0,12946	0,57613

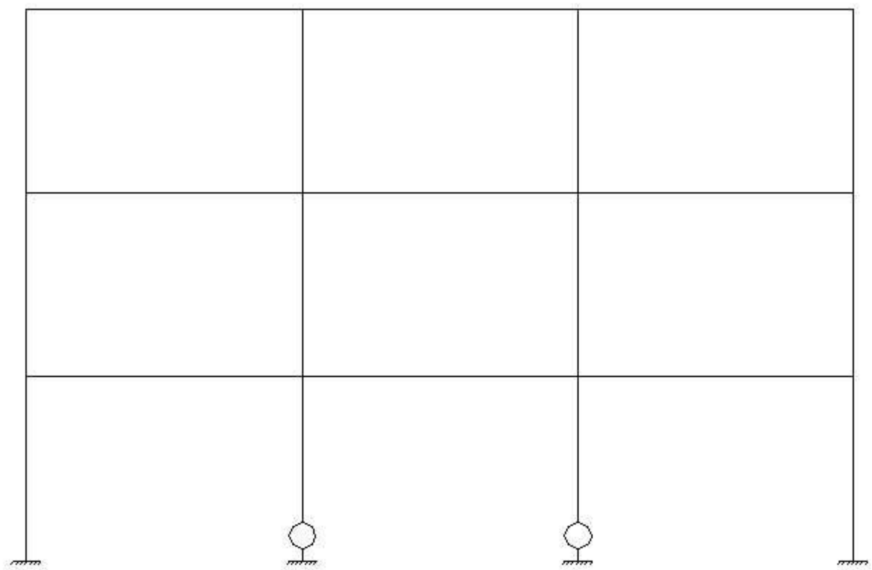


Figure 5-58: Hinge Formation for Form 2, Soil Type:Z1

(Base Shear: 8,0 t, Displacement: 17,5 cm)

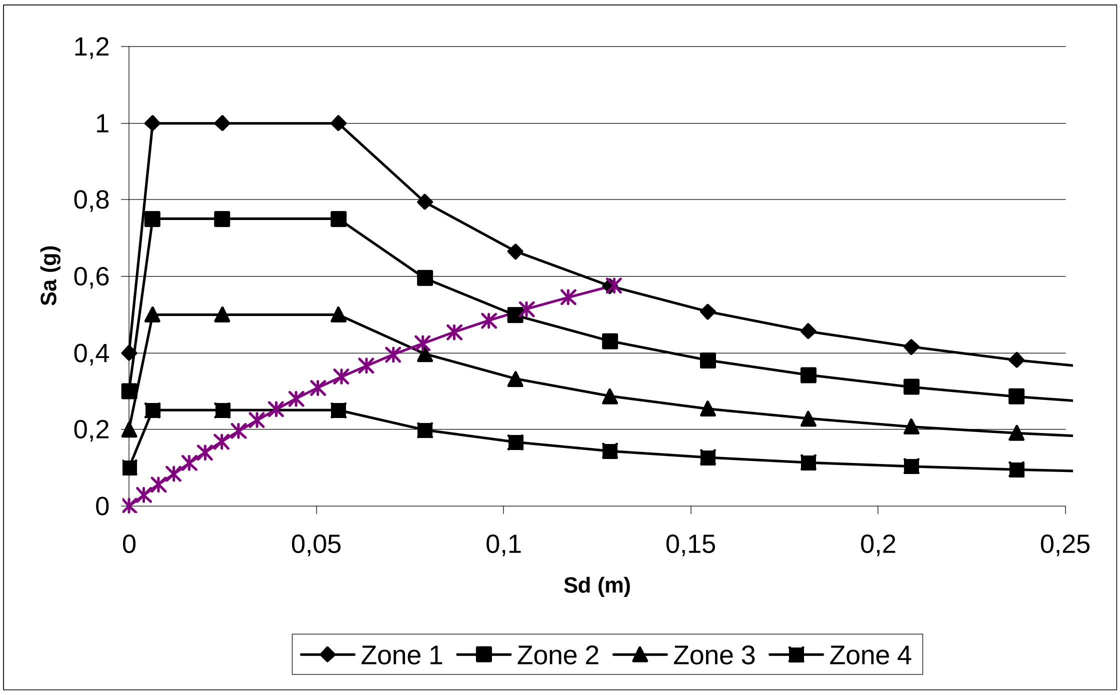


Figure 5-59: Capacity vs. Demand Spectra for Case 2 (Form 2)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-60, Figure 5-61 and Figure 5-62 respectively.

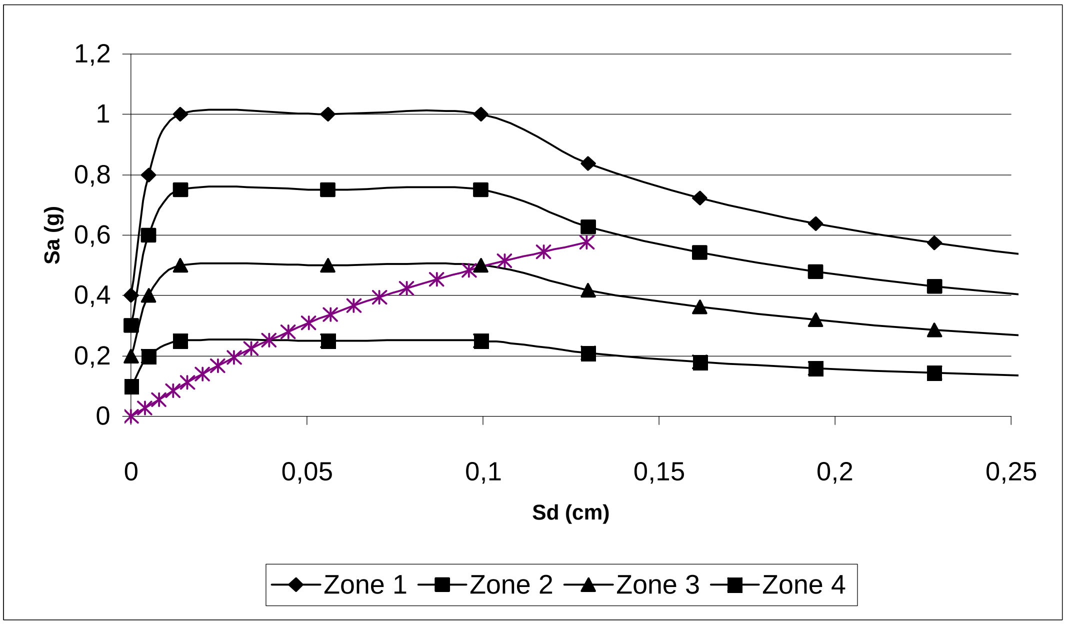


Figure 5-60: Capacity vs. Demand Spectra for Form 2, Soil Type:Z2

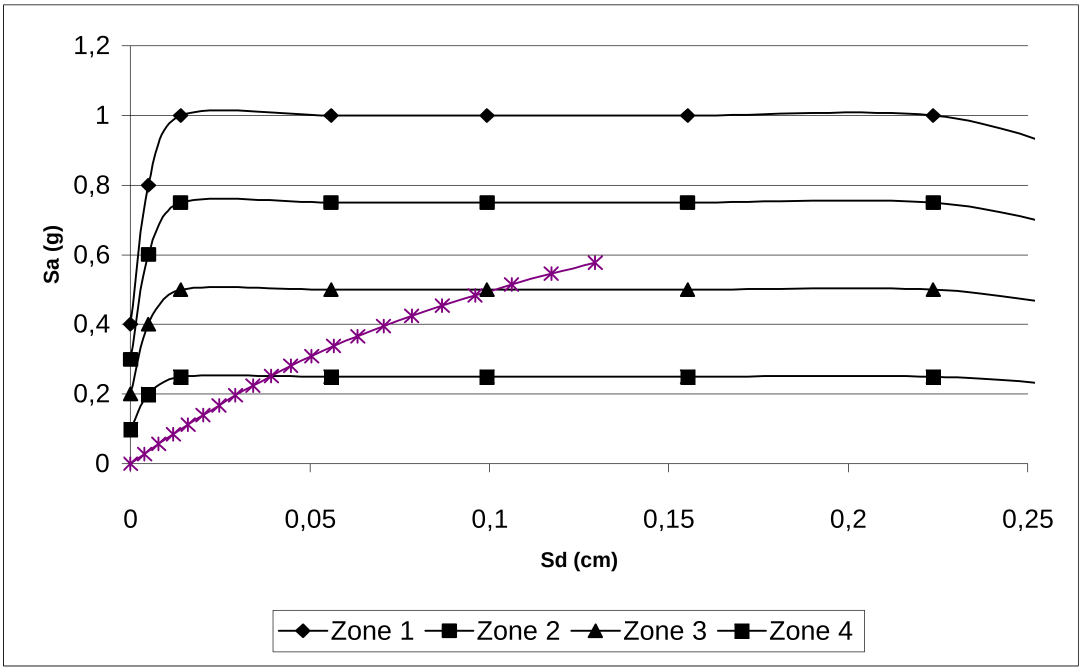


Figure 5-61: Capacity vs. Demand Spectra for Form 2, Soil Type:Z3

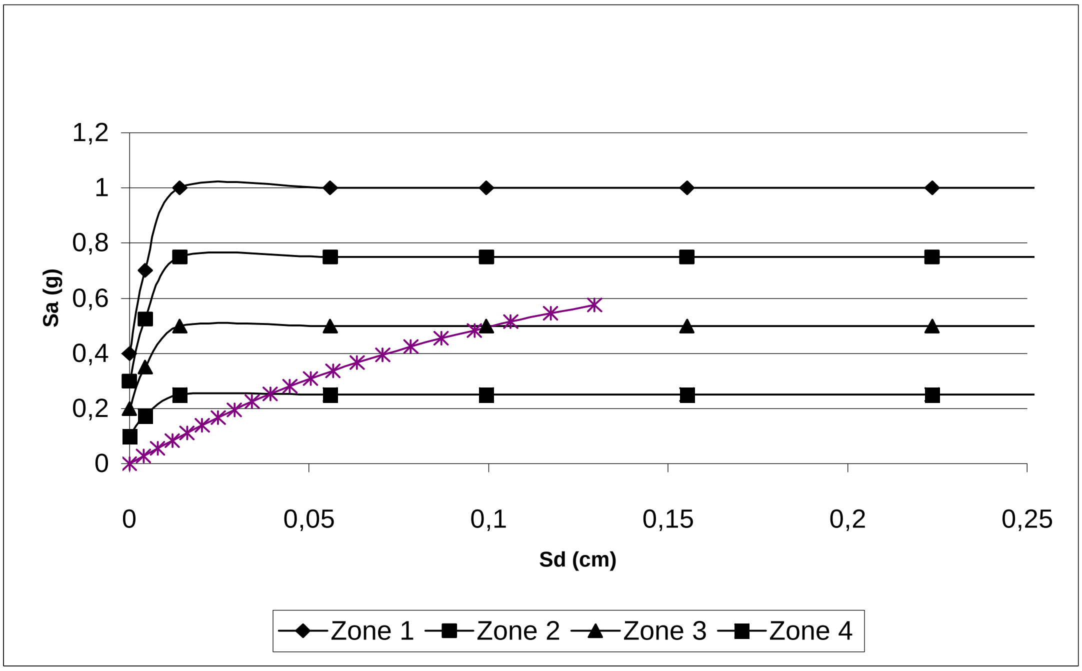


Figure 5-62: Capacity vs. Demand Spectra for Form 2, Soil Type:Z4

5.4.3 Case Study #2, Form 3

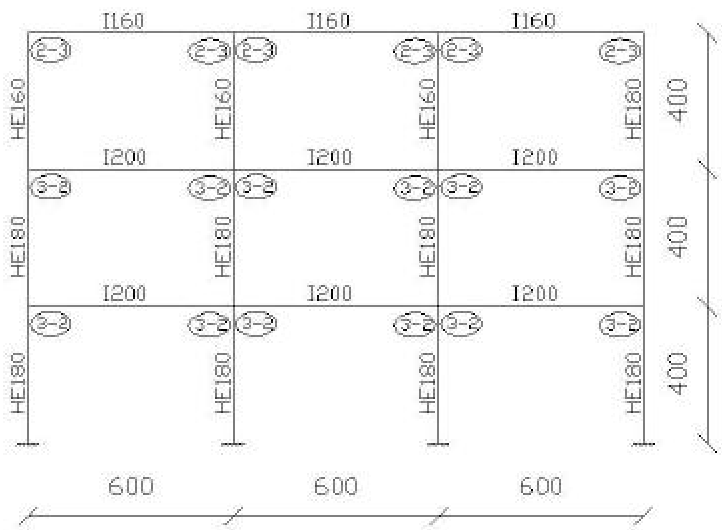


Figure 5-63 Semi Rigid Connection Types (Form 3)

Table 5-11: Data for Performance Analysis for Case Study 2-Form 3

Number of Floors : 3 **Connections :** Fixed **Load Increment :** 400 kg

Number of Bays : 3 **P-Delta effect :** Yes

Top Disp. *(meter)*	***B. Shear*** *(ton)*	***W_1*** *(ton)*	***W_2*** *(ton)*	***W_3*** *(ton)*	*f_1*	*f_2*	*f_3*	***PF1***	α_1	***S_d*** *(meter)*	***S_a*** *($meter/s^2$)*
0,0000	0,0000	6,332	6,332	3,4030				0,0000	0,0000	0,0000	0,0000
0,0000	0,0000	6,332	6,332	3,4030	0,2645	0,2008	1,0000	1,5481	0,6118	0,0000	0,0000
0,0064	0,4000	6,332	6,332	3,4030	0,3906	0,8185	1,0000	1,2842	0,8840	0,0050	0,0282
0,0129	0,8000	6,332	6,332	3,4030	0,3976	0,8238	1,0000	1,2799	0,8872	0,0101	0,0561
0,0193	1,2000	6,332	6,332	3,4030	0,3999	0,8256	1,0000	1,2785	0,8882	0,0151	0,0841
0,0258	1,6000	6,332	6,332	3,4030	0,4011	0,8265	1,0000	1,2777	0,8888	0,0202	0,1120
0,0322	2,0000	6,332	6,332	3,4030	0,4018	0,8270	1,0000	1,2773	0,8891	0,0252	0,1400
0,0387	2,4000	6,332	6,332	3,4030	0,4023	0,8274	1,0000	1,2770	0,8893	0,0303	0,1680
0,0451	2,8000	6,332	6,332	3,4030	0,4026	0,8276	1,0000	1,2768	0,8894	0,0353	0,1959
0,0516	3,2000	6,332	6,332	3,4030	0,4029	0,8278	1,0000	1,2766	0,8896	0,0404	0,2239
0,0580	3,6000	6,332	6,332	3,4030	0,4031	0,8280	1,0000	1,2765	0,8896	0,0454	0,2519
0,0644	4,0000	6,332	6,332	3,4030	0,4032	0,8281	1,0000	1,2764	0,8897	0,0505	0,2798
0,0709	4,4000	6,332	6,332	3,4030	0,4033	0,8282	1,0000	1,2763	0,8898	0,0555	0,3078
0,0773	4,8000	6,332	6,332	3,4030	0,4035	0,8282	1,0000	1,2762	0,8898	0,0606	0,3357
0,0838	5,2000	6,332	6,332	3,4030	0,4035	0,8283	1,0000	1,2762	0,8899	0,0657	0,3637
0,0902	5,6000	6,332	6,332	3,4030	0,4036	0,8284	1,0000	1,2761	0,8899	0,0707	0,3917
0,0967	6,0000	6,332	6,332	3,4030	0,4037	0,8284	1,0000	1,2761	0,8899	0,0758	0,4196
0,1031	6,4000	6,332	6,332	3,4030	0,4037	0,8285	1,0000	1,2761	0,8899	0,0808	0,4476
0,1096	6,8000	6,332	6,332	3,4030	0,4038	0,8285	1,0000	1,2760	0,8900	0,0859	0,4756
0,1160	7,2000	6,332	6,332	3,4030	0,4038	0,8285	1,0000	1,2760	0,8900	0,0909	0,5035

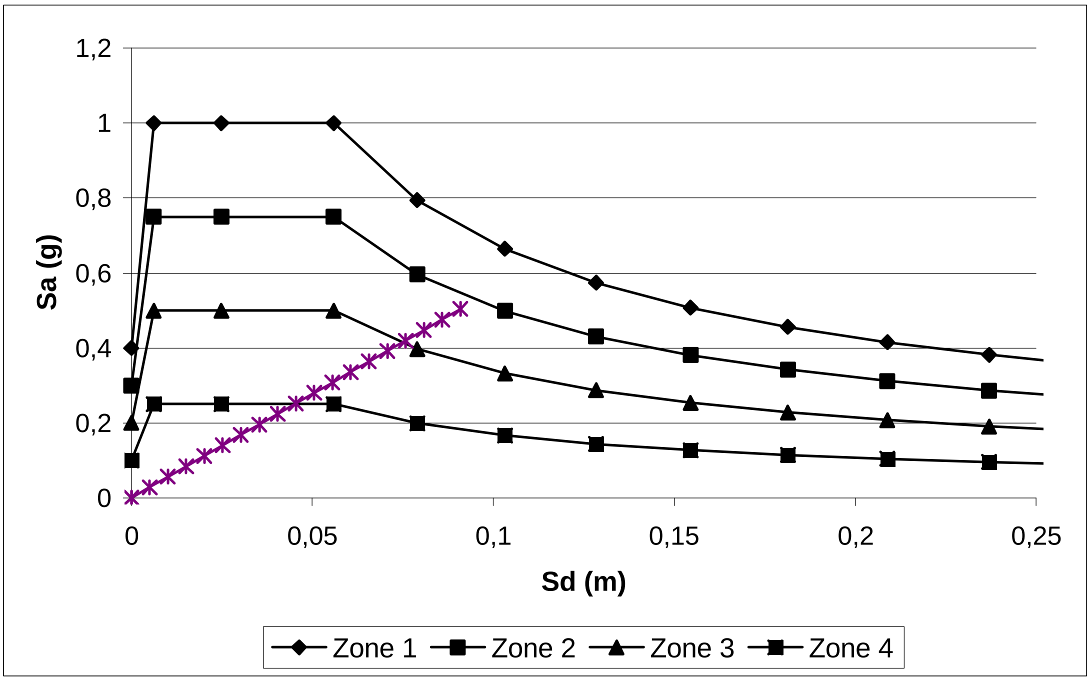


Figure 5-64: Capacity vs. Demand Spectra for Semi-Rigid Case 2 (Form 3)

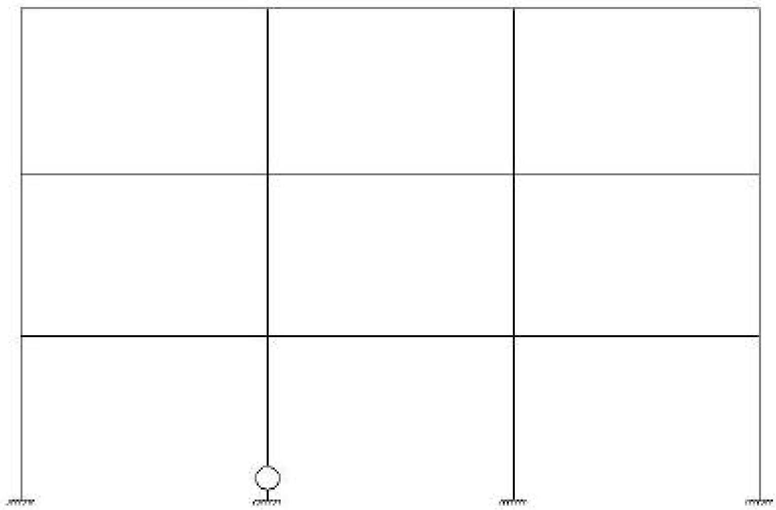


Figure 5-65: Hinge Formation for Form 3, Soil Type:Z1

(Base Shear: 7,2 t, Displacement: 11,6 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-66, Figure 5-67 and Figure 5-68 respectively.

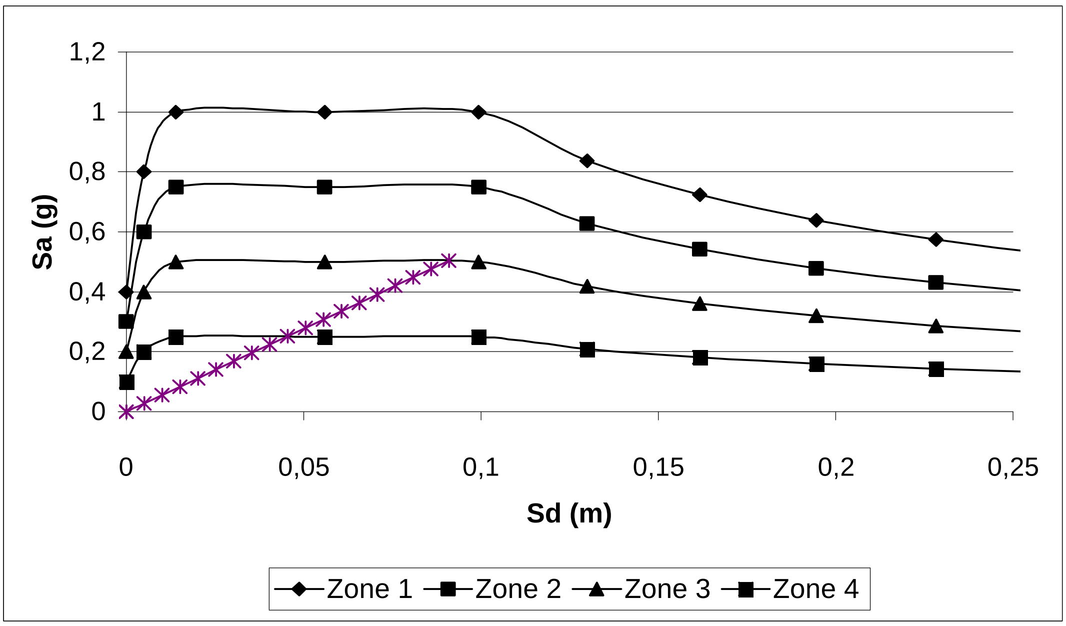


Figure 5-66: Capacity vs. Demand Spectra for Form 3, Soil Type:Z2

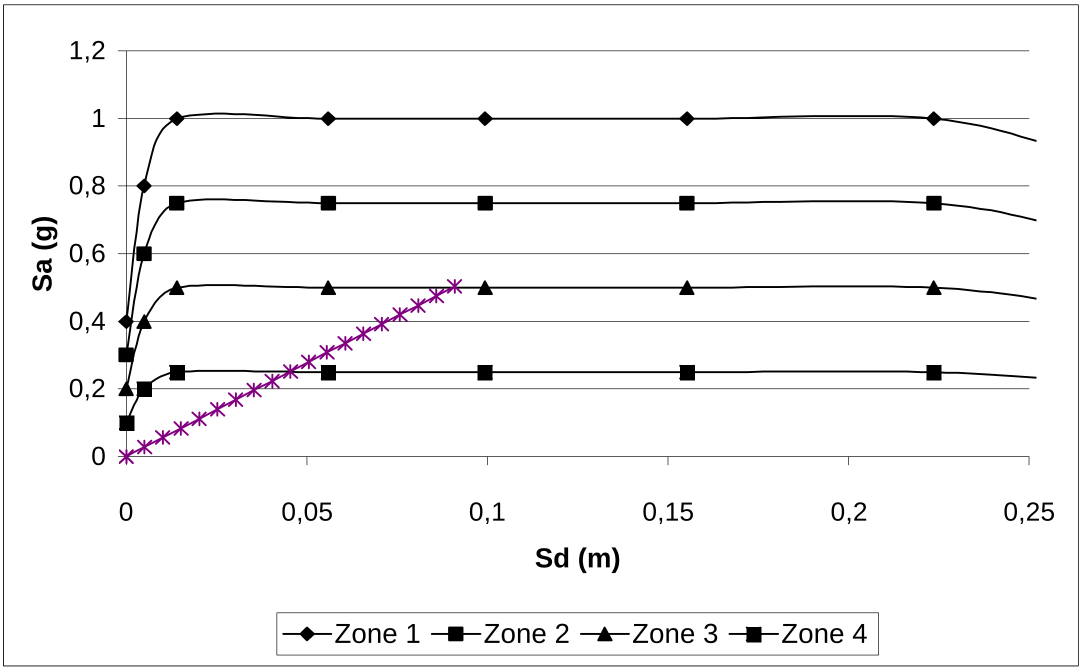


Figure 5-67: Capacity vs. Demand Spectra for Form 3, Soil Type:Z3

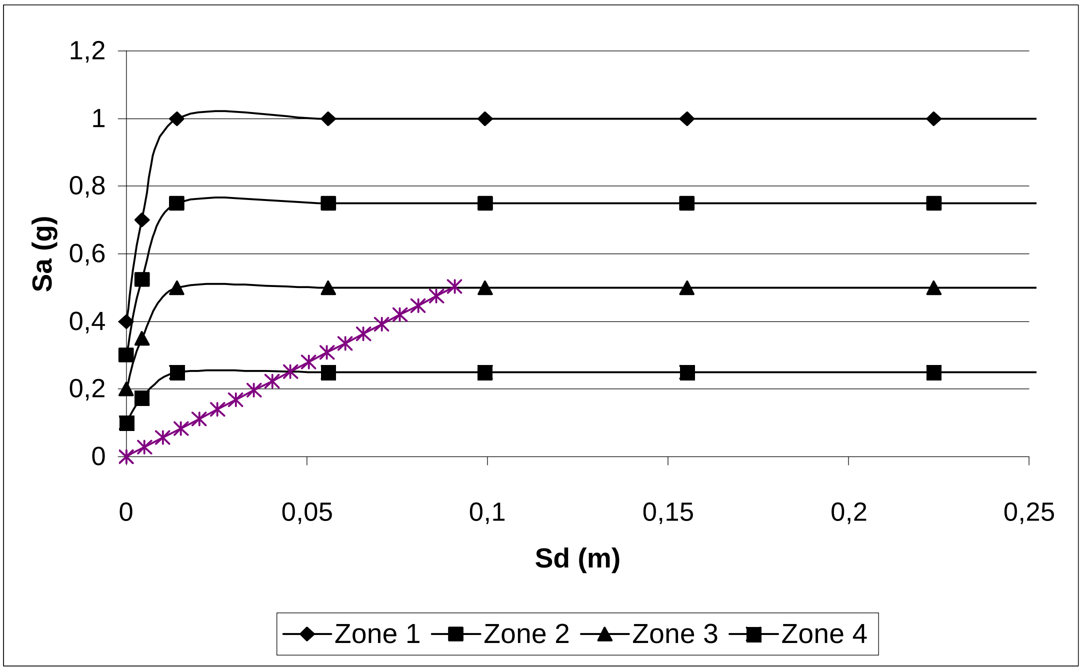


Figure 5-68: Capacity vs. Demand Spectra for Form 3, Soil Type:Z4

5.4.4 Case Study #2, Form 4

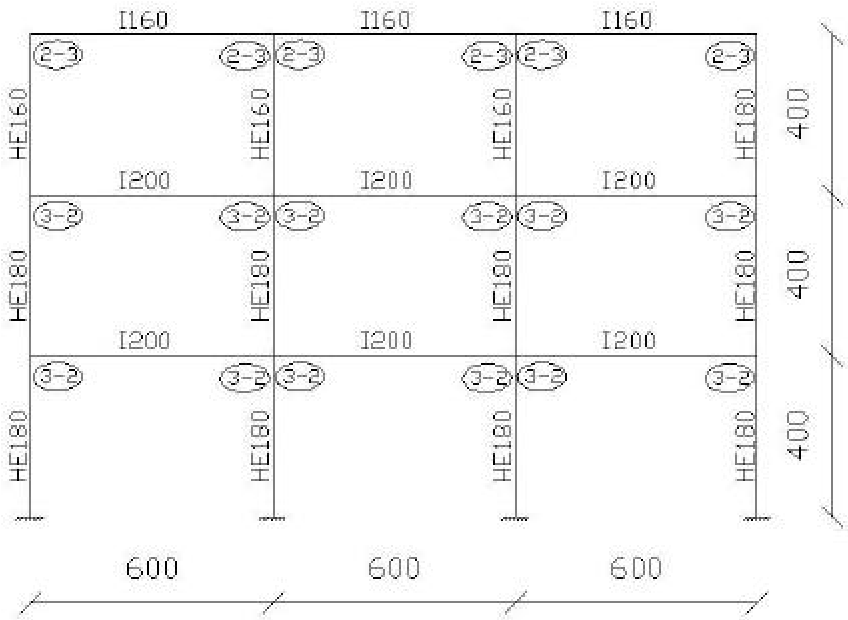


Figure 5-69 Semi Rigid Connection Types (Form 4)

Table 5-12: Data for Performance Analysis for Case Study 2-Form 4

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 400 kg

Number of Bays : 3 **P-Delta effect :** Yes

Top Disp. *(meter)*	***B. Shear*** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	***PF1***	α_1	S_d *(meter)*	S_a *(meter/s²)*
0,0000	0,0000	6,332	6,332	3,403				0,0000	0,0000	0,0000	0,0000
0,0076	0,4000	6,332	6,332	3,403	0,2645	0,7817	1,0000	1,2998	0,8112	0,0059	0,0307
0,0153	0,8000	6,332	6,332	3,403	0,3687	0,7866	1,0000	1,3100	0,8739	0,0117	0,0570
0,0232	1,2000	6,332	6,332	3,403	0,3703	0,7883	1,0000	1,3087	0,8747	0,0177	0,0854
0,0312	1,6000	6,332	6,332	3,403	0,3708	0,7891	1,0000	1,3080	0,8750	0,0239	0,1138
0,0396	2,0000	6,332	6,332	3,403	0,3707	0,7896	1,0000	1,3076	0,8749	0,0303	0,1423
0,0484	2,4000	6,332	6,332	3,403	0,3703	0,7898	1,0000	1,3074	0,8747	0,0370	0,1708
0,0576	2,8000	6,332	6,332	3,403	0,3695	0,7899	1,0000	1,3074	0,8742	0,0441	0,1993
0,0675	3,2000	6,332	6,332	3,403	0,3683	0,7896	1,0000	1,3076	0,8736	0,0516	0,2280
0,0782	3,6000	6,332	6,332	3,403	0,3668	0,7891	1,0000	1,3080	0,8728	0,0598	0,2567
0,0897	4,0000	6,332	6,332	3,403	0,3648	0,7881	1,0000	1,3087	0,8718	0,0685	0,2856
0,1023	4,4000	6,332	6,332	3,403	0,3623	0,7866	1,0000	1,3099	0,8705	0,0781	0,3146
0,1161	4,8000	6,332	6,332	3,403	0,3592	0,7844	1,0000	1,3115	0,8689	0,0885	0,3438
0,1314	5,2000	6,332	6,332	3,403	0,3555	0,7814	1,0000	1,3138	0,8669	0,1000	0,3733
0,1485	5,6000	6,332	6,332	3,403	0,3510	0,7775	1,0000	1,3168	0,8645	0,1128	0,4032
0,1677	6,0000	6,332	6,332	3,403	0,3457	0,7724	1,0000	1,3207	0,8617	0,1270	0,4334

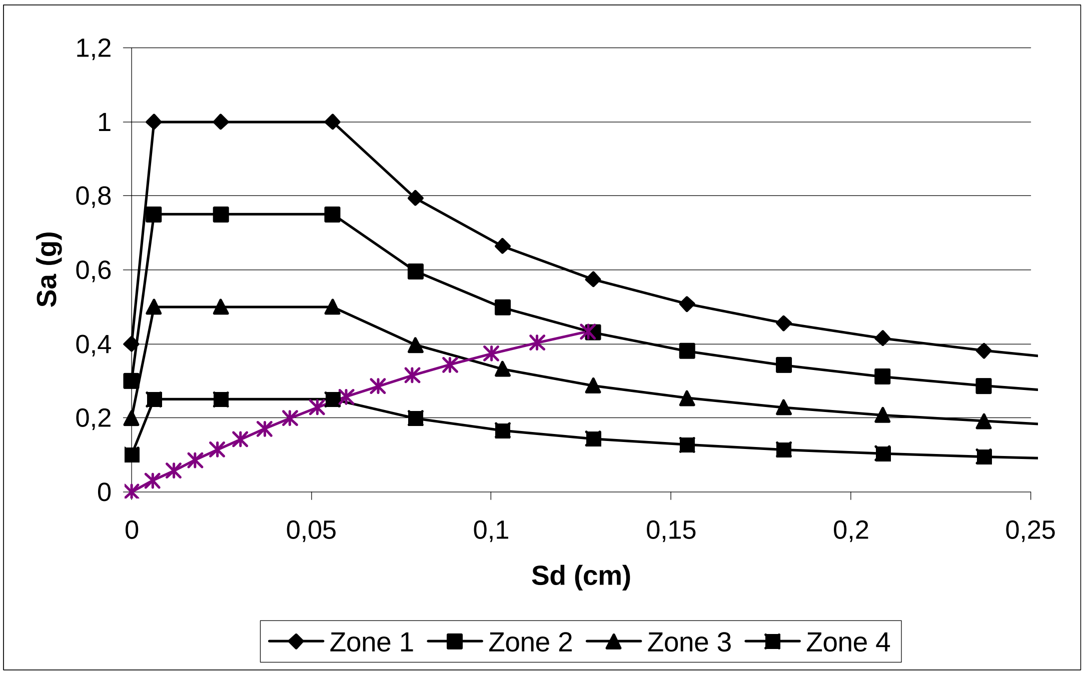


Figure 5-70: Capacity vs. Demand Spectra for Semi-Rigid Case 2 (Form 4)

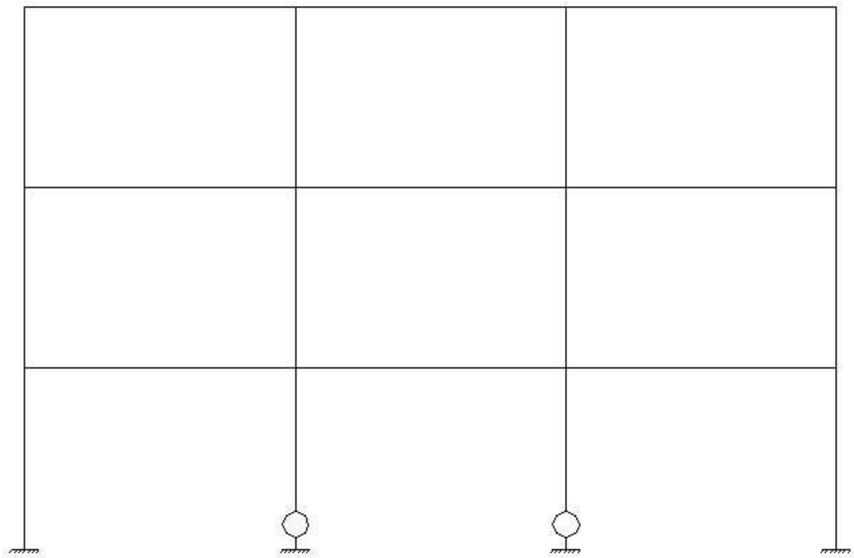


Figure 5-71: Hinge Formation for Form 4, Soil Type:Z1

(Base Shear: 6,0 t, Displacement: 16,8 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-72, Figure 5-73 and Figure 5-74 respectively.

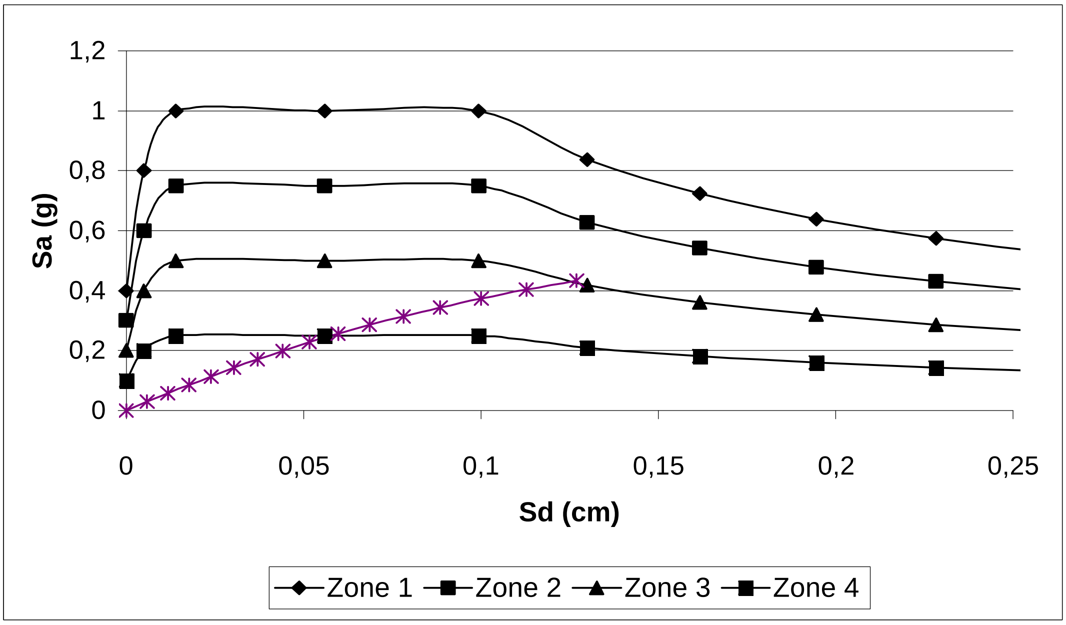


Figure 5-72: Capacity vs. Demand Spectra for Form 4, Soil Type:Z2

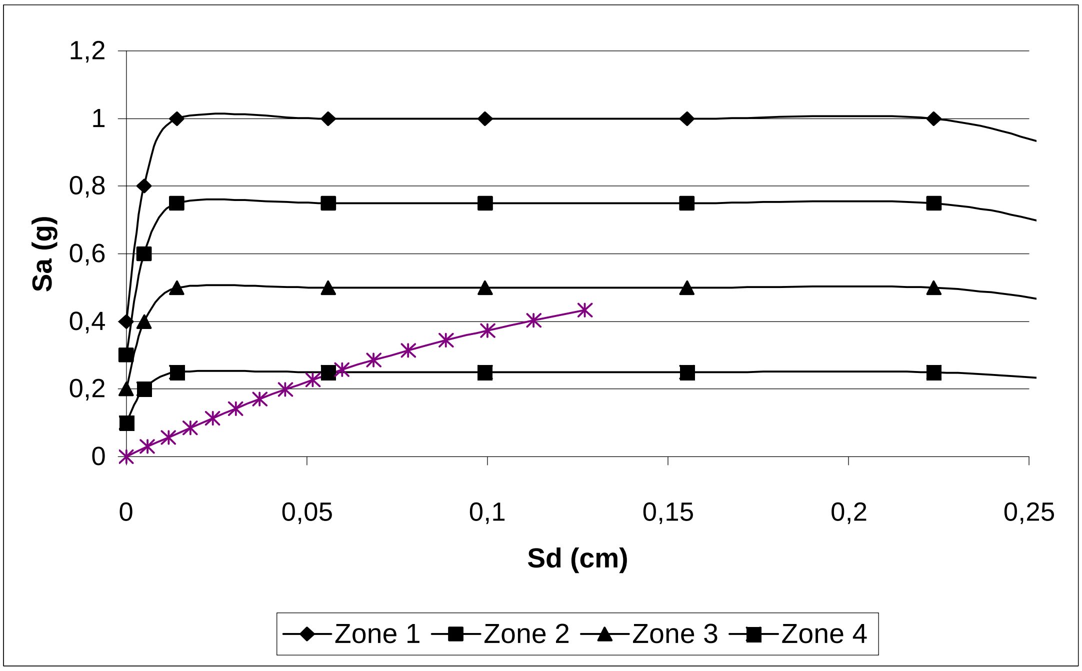


Figure 5-73: Capacity vs. Demand Spectra for Form 4, Soil Type:Z3

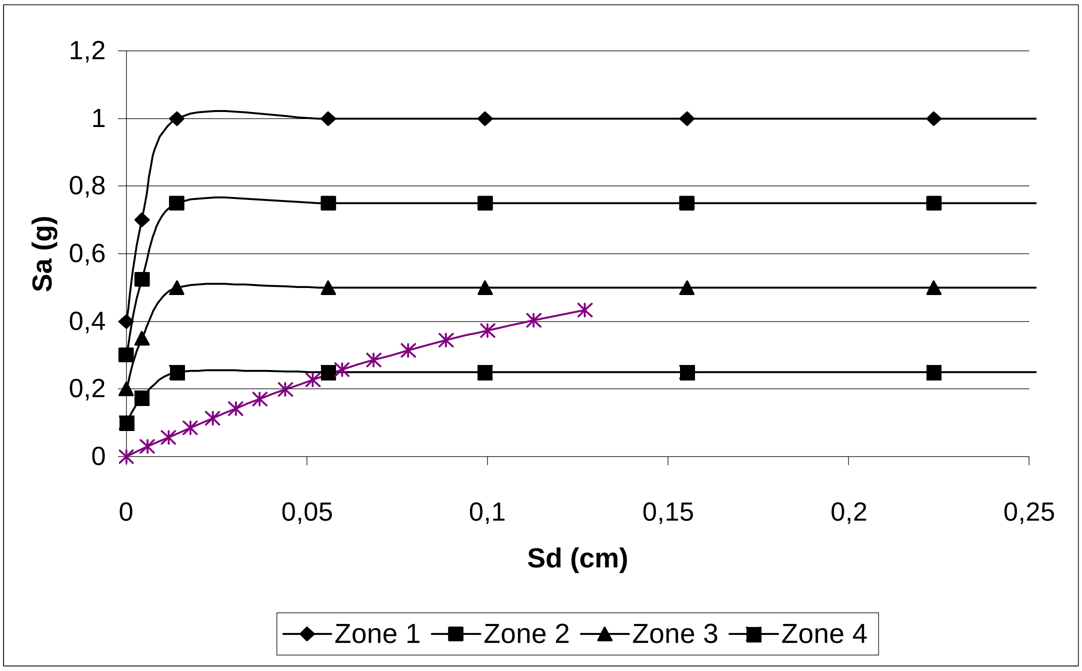


Figure 5-74: Capacity vs. Demand Spectra for Form 4, Soil Type:Z4

5.4.5 Case Study #2, Form 5

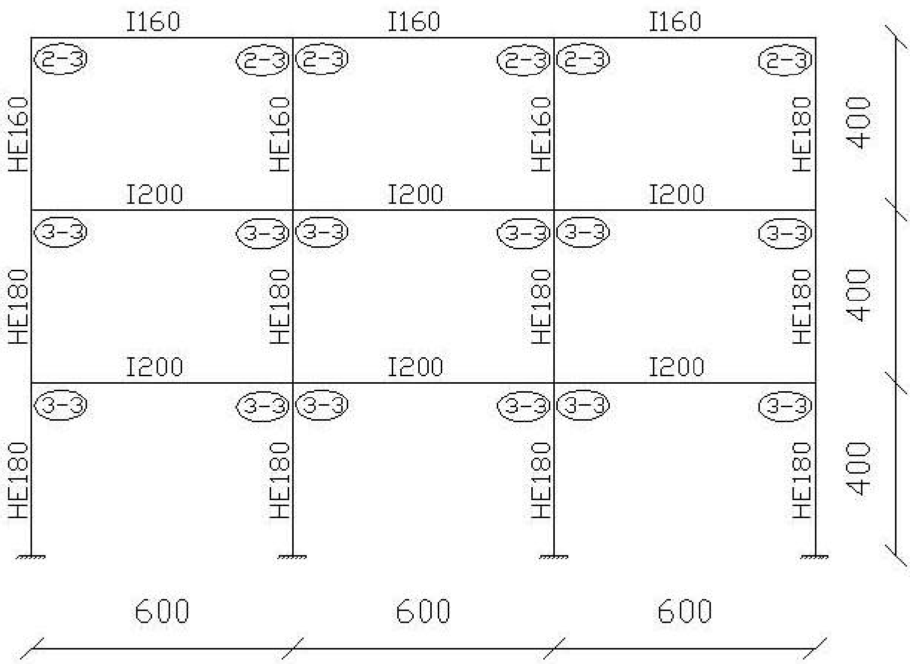


Figure 5-75 Semi Rigid Connection Types (Form 5)

Table 5-13: Data for Performance Analysis for Case Study 2-Form 5

Number of Floors : 3			Connections :		Semi-Rigid		Load Increment :		400 kg		
Number of Bays : 3			P-Delta effect :		Yes						
Top Disp. *(meter)*	**B. Shear** *(ton)*	W_1 (ton)	W_2 (ton)	W_3 (ton)	f_1	f_2	f_3	**PF1**	α_1	S_d (meter)	S_a (meter/s^2)
0,0000	0,0000	6,332	6,332	3,403				0,0000	0,0000	0,0000	0,0000
0,0085	0,4000	6,332	6,332	3,403	0,2645	0,7775	1,0000	1,3033	0,8113	0,0065	0,0307
0,0172	0,8000	6,332	6,332	3,403	0,3590	0,7818	1,0000	1,3136	0,8688	0,0131	0,0573
0,0264	1,2000	6,332	6,332	3,403	0,3597	0,7833	1,0000	1,3125	0,8692	0,0201	0,0859
0,0364	1,6000	6,332	6,332	3,403	0,3588	0,7837	1,0000	1,3121	0,8687	0,0278	0,1146
0,0477	2,0000	6,332	6,332	3,403	0,3568	0,7833	1,0000	1,3123	0,8676	0,0364	0,1435
0,0608	2,4000	6,332	6,332	3,403	0,3534	0,7818	1,0000	1,3134	0,8658	0,0463	0,1725
0,0761	2,8000	6,332	6,332	3,403	0,3486	0,7787	1,0000	1,3156	0,8631	0,0578	0,2019
0,0942	3,2000	6,332	6,332	3,403	0,3420	0,7738	1,0000	1,3192	0,8596	0,0714	0,2317
0,1161	3,6000	6,332	6,332	3,403	0,3339	0,7669	1,0000	1,3243	0,8550	0,0877	0,2621
0,1428	4,0000	6,332	6,332	3,403	0,3241	0,7575	1,0000	1,3311	0,8493	0,1073	0,2931
0,1761	4,4000	6,332	6,332	3,403	0,3124	0,7451	1,0000	1,3401	0,8423	0,1314	0,3251
0,2192	4,8000	6,332	6,332	3,403	0,2984	0,7286	1,0000	1,3517	0,8334	0,1622	0,3585

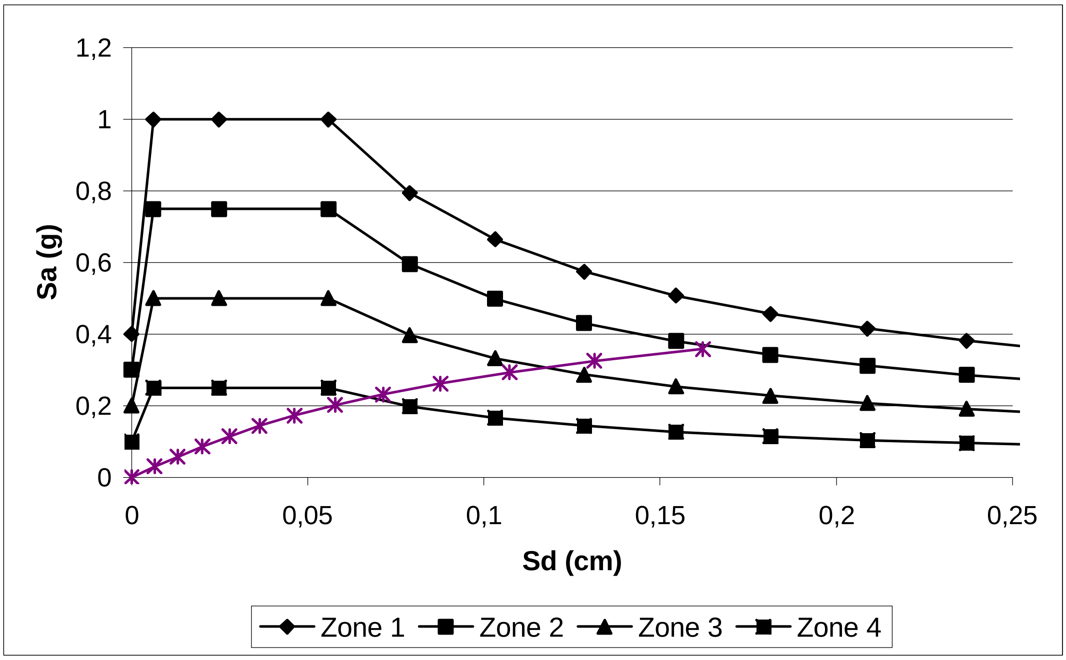


Figure 5-76: Capacity vs. Demand Spectra for Semi-Rigid Case 2 (Form 5)

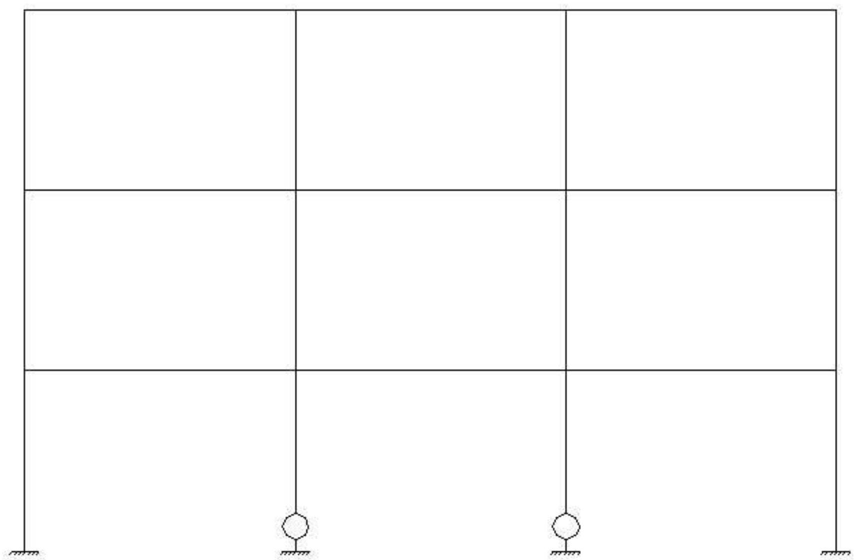


Figure 5-77: Hinge Formation for Form 5, Soil Type:Z1

(Base Shear: 4,8 t, Displacement: 21,9 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-78, Figure 5-79 and Figure 5-80 respectively.

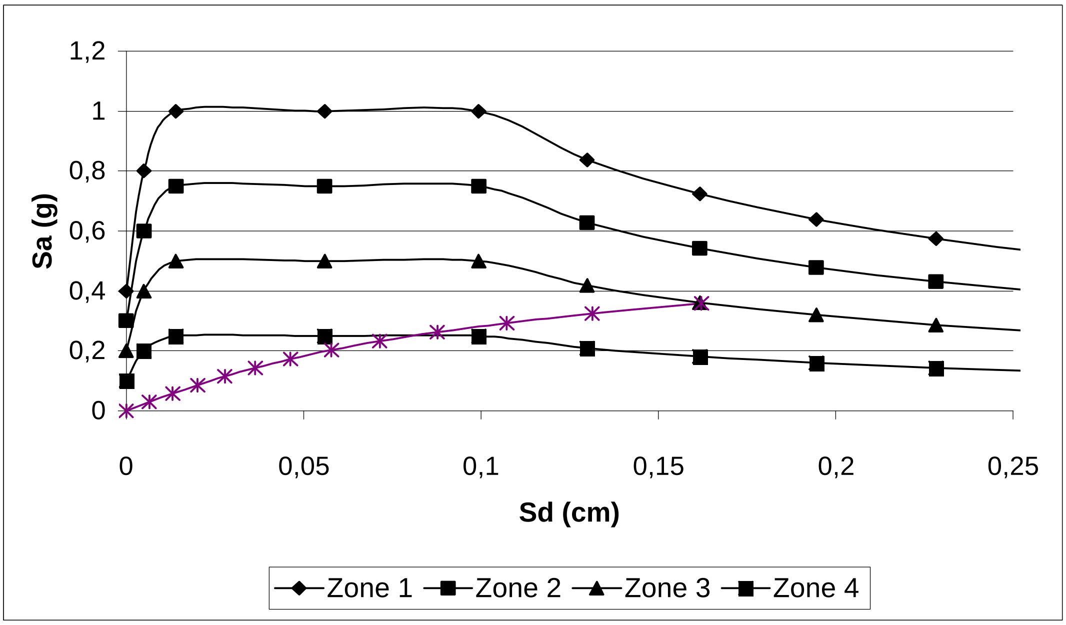


Figure 5-78: Capacity vs. Demand Spectra for Form 5, Soil Type:Z2

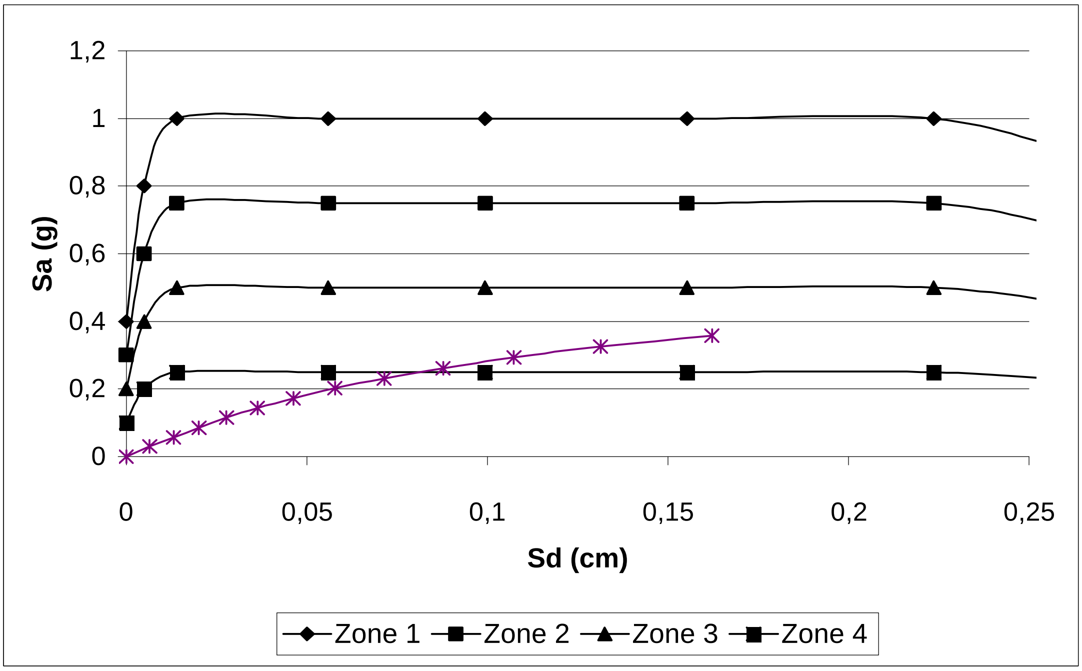


Figure 5-79: Capacity vs. Demand Spectra for Form 5, Soil Type:Z3

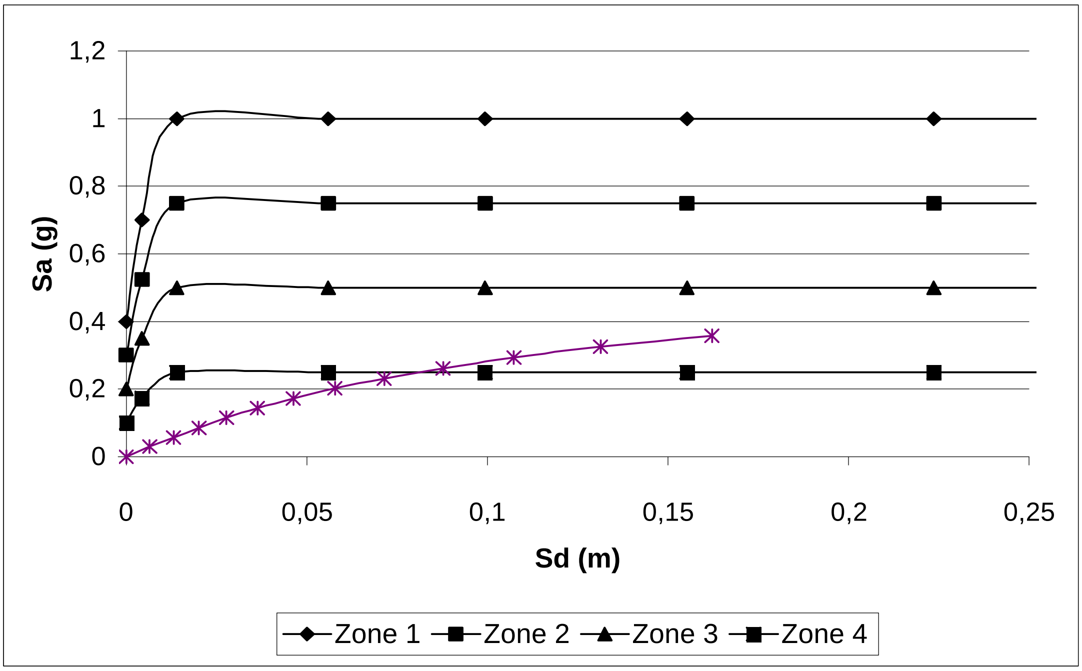


Figure 5-80: Capacity vs. Demand Spectra for Form 5, Soil Type:Z4

5.4.6 Case Study #2, Form 6

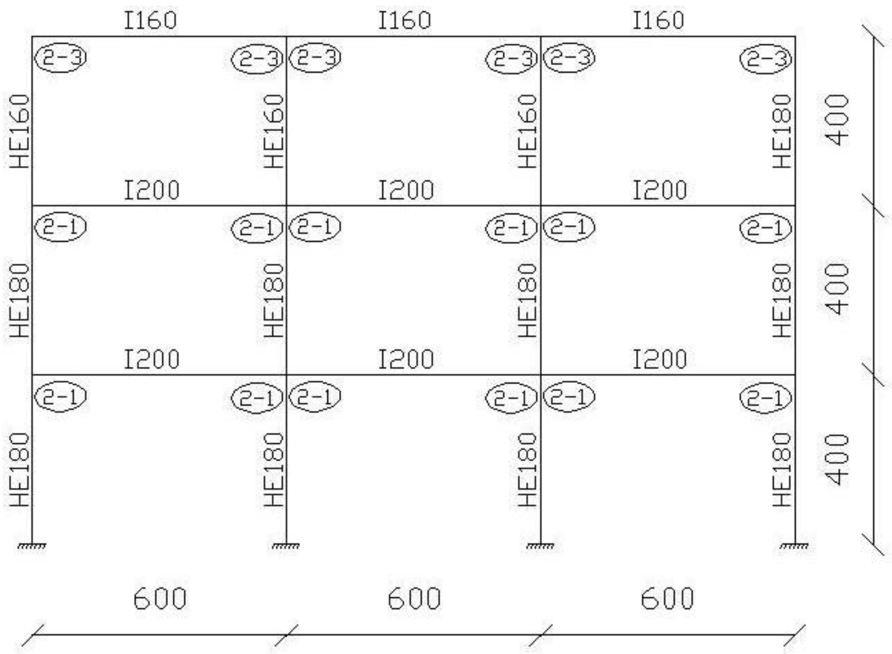


Figure 5-81 Semi Rigid Connection Types (Form 6)

Table 5-14: Data for Performance Analysis for Case Study 2-Form 6

Number of Floors : 3	**Connections :**	Semi-Rigid	**Load Increment :**	400 kg
Number of Bays : 3	**P-Delta effect :**	Yes		

Top Disp. *(meter)*	***B. Shear*** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	***PF1***	α_1	S_d *(meter)*	S_a *(meter/s²)*
0,0000	0,0000	6,332	6,332	3,403				0,0000	0,0000	0,0000	0,0000
0,0135	0,4000	6,332	6,332	3,403	0,2645	0,7615	1,0000	1,3168	0,8114	0,0103	0,0307
0,0280	0,8000	6,332	6,332	3,403	0,3216	0,7632	1,0000	1,3261	0,8478	0,0211	0,0587
0,0449	1,2000	6,332	6,332	3,403	0,3195	0,7623	1,0000	1,3266	0,8466	0,0338	0,0882
0,0668	1,6000	6,332	6,332	3,403	0,3138	0,7583	1,0000	1,3292	0,8431	0,0503	0,1181
0,0984	2,0000	6,332	6,332	3,403	0,3034	0,7498	1,0000	1,3348	0,8367	0,0737	0,1488
0,1508	2,4000	6,332	6,332	3,403	0,2882	0,7357	1,0000	1,3440	0,8269	0,1122	0,1806
0,2408	2,8000	6,332	6,332	3,403	0,2651	0,7086	1,0000	1,3616	0,8109	0,1769	0,2149
0,3800	3,2000	6,332	6,332	3,403	0,2418	0,6747	1,0000	1,3832	0,7926	0,2747	0,2513

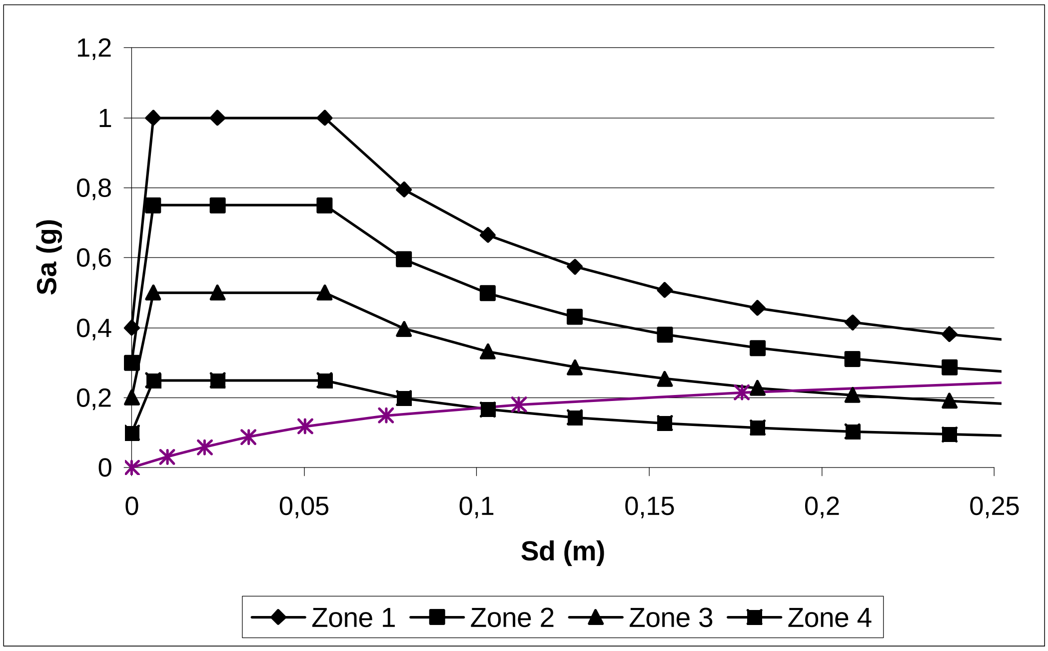


Figure 5-82: Capacity vs. Demand Spectra for Semi-Rigid Case 2 (Form 6)

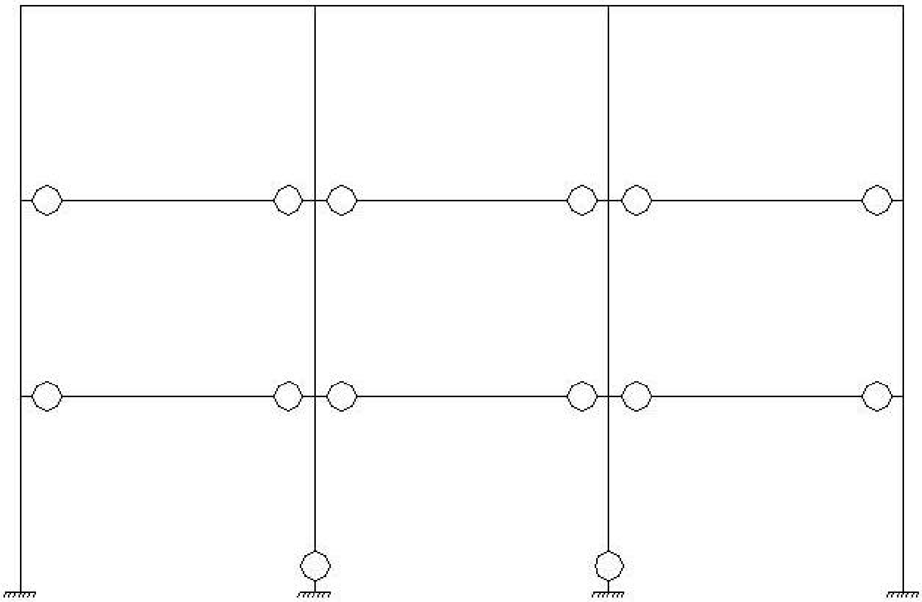


Figure 5-83: Hinge Formation for Form 5, Soil Type:Z1

(Base Shear: 4,8 t, Displacement: 21,9 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-84, Figure 5-85 and Figure 5-86 respectively.

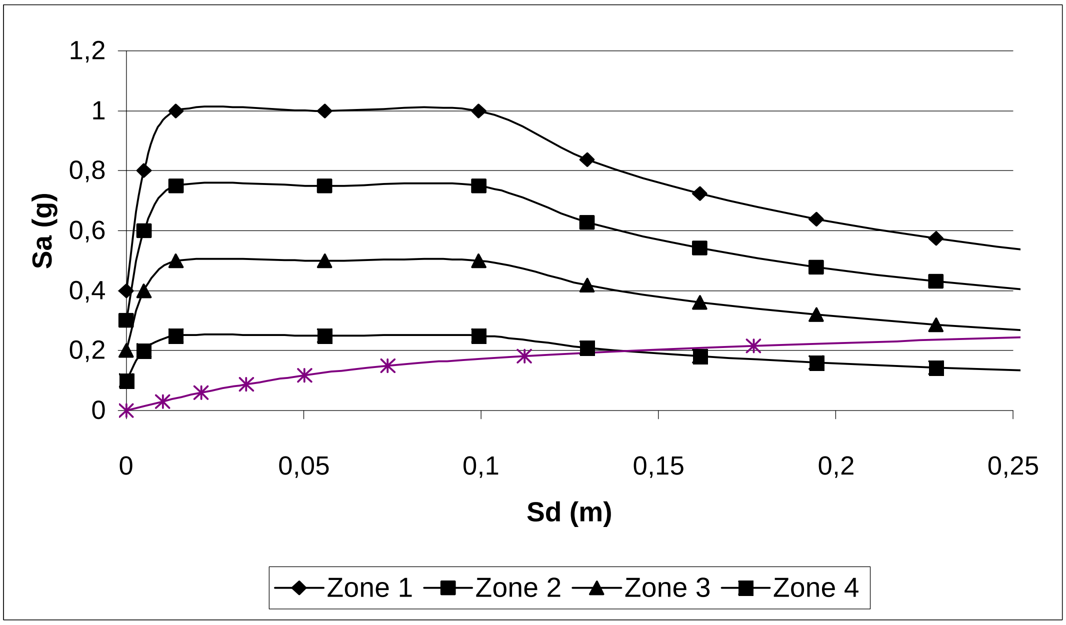


Figure 5-84: Capacity vs. Demand Spectra for Form 6, Soil Type:Z2

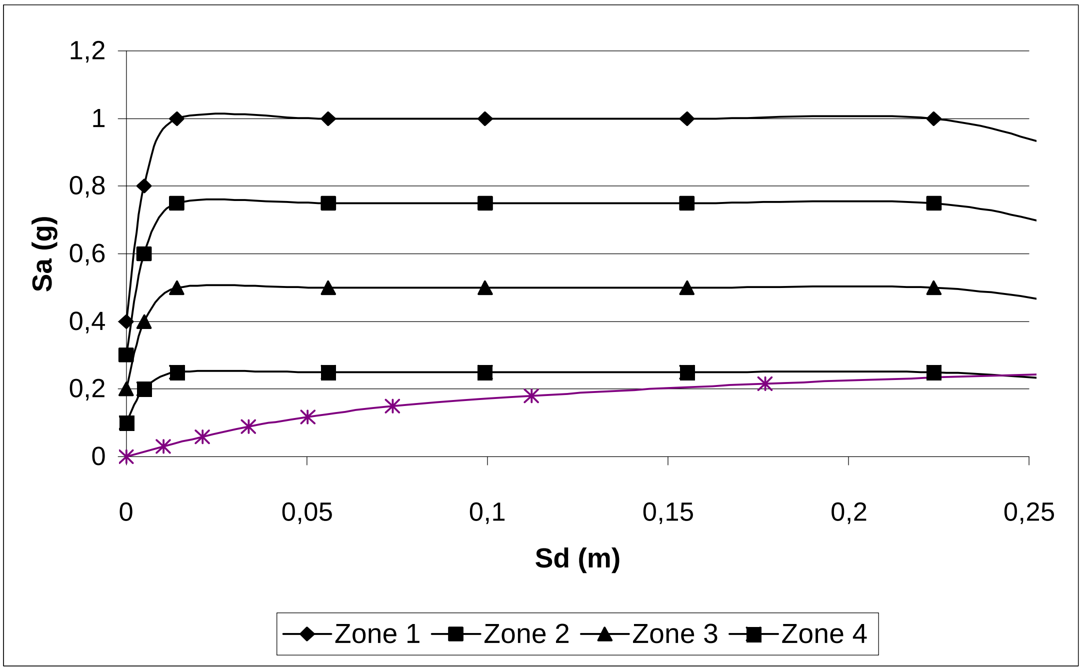


Figure 5-85: Capacity vs. Demand Spectra for Form 6, Soil Type:Z3

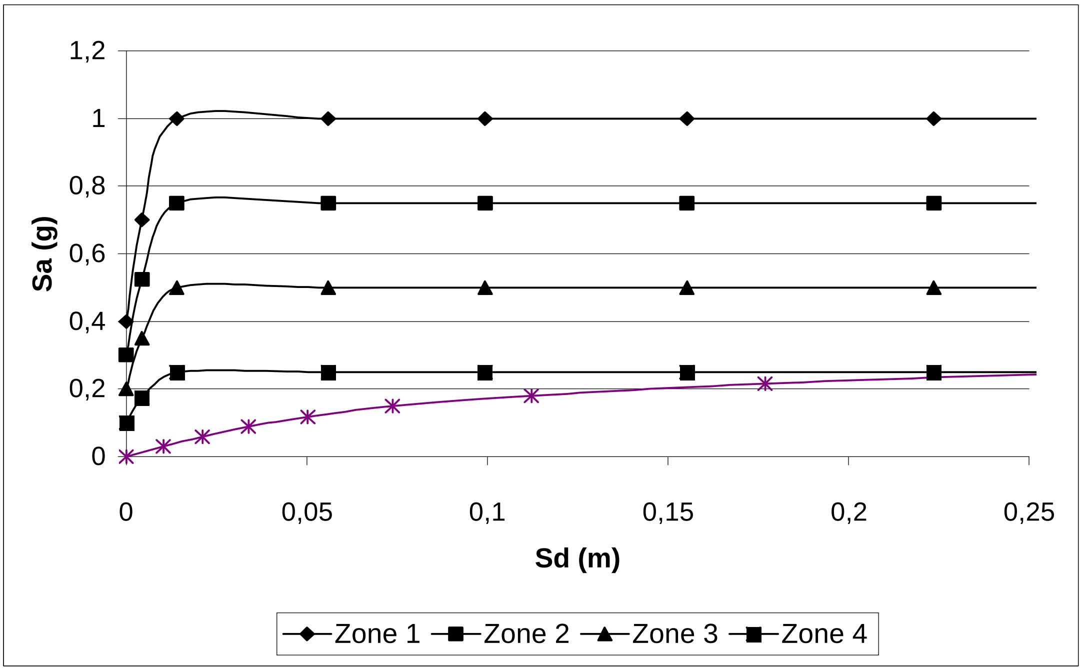


Figure 5-86: Capacity vs. Demand Spectra for Form 6, Soil Type:Z4

5.4.7 Case Study #2, Form 7

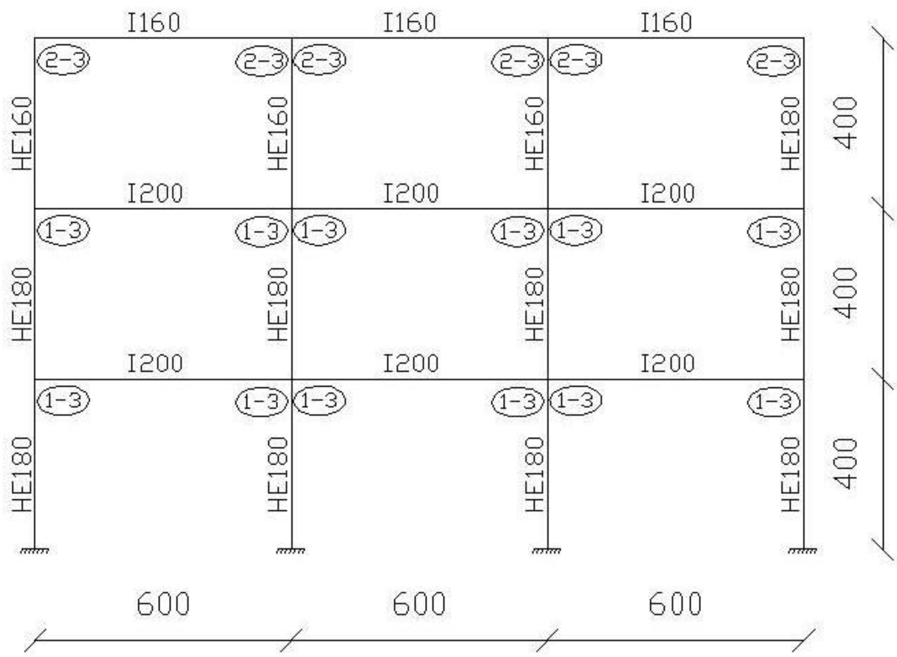


Figure 5-87 Semi Rigid Connection Types (Form 7)

Table 5-15: Data for Performance Analysis for Case Study 2-Form 7

Number of Floors : 3			**Connections :**		Semi-Rigid		**Load Increment :**		400 kg		
Number of Bays : 3			**P-Delta effect :**		Yes						
Top Disp. *(meter)*	***B. Shear*** *(ton)*	W_1 *(ton)*	W_2 *(ton)*	W_3 *(ton)*	f_1	f_2	f_3	***PF1***	α_1	S_d *(meter)*	S_a *(meter/s^2)*
0,0000	0,0000	6,332	6,332	3,403				0,0000	0,0000	0,0000	0,0000
0,0074	0,4000	6,332	6,332	3,403	0,2645	0,7910	1,0000	1,2919	0,8110	0,0057	0,0307
0,0148	0,8000	6,332	6,332	3,403	0,3750	0,7959	1,0000	1,3026	0,8769	0,0113	0,0568
0,0222	1,2000	6,332	6,332	3,403	0,3769	0,7975	1,0000	1,3013	0,8779	0,0170	0,0851
0,0296	1,6000	6,332	6,332	3,403	0,3778	0,7983	1,0000	1,3007	0,8783	0,0228	0,1134
0,0381	2,0000	6,332	6,332	3,403	0,3798	0,8019	1,0000	1,2977	0,8792	0,0294	0,1416
0,0467	2,4000	6,332	6,332	3,403	0,3810	0,8042	1,0000	1,2958	0,8797	0,0360	0,1698
0,0566	2,8000	6,332	6,332	3,403	0,3771	0,8020	1,0000	1,2976	0,8778	0,0436	0,1985
0,0741	3,2000	6,332	6,332	3,403	0,3757	0,8072	1,0000	1,2933	0,8769	0,0573	0,2271
0,0956	3,6000	6,332	6,332	3,403	0,3754	0,8126	1,0000	1,2889	0,8764	0,0742	0,2557
0,1437	4,0000	6,332	6,332	3,403	0,3402	0,7824	1,0000	1,3120	0,8583	0,1095	0,2901
0,2272	4,4000	6,332	6,332	3,403	0,3005	0,7425	1,0000	1,3404	0,8349	0,1695	0,3280

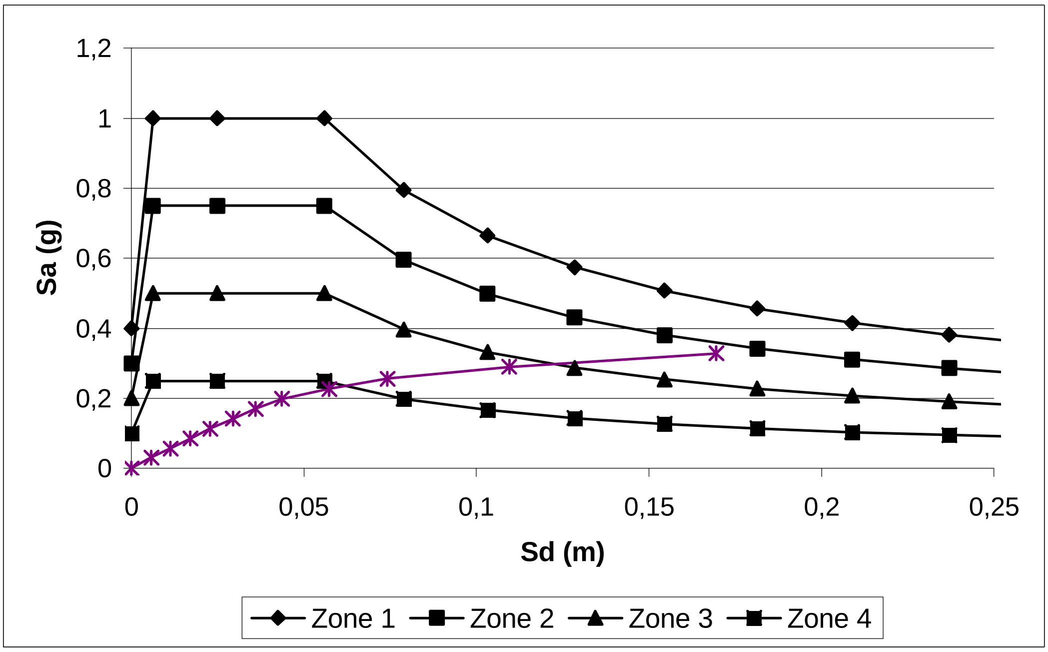


Figure 5-88: Capacity vs. Demand Spectra for Semi-Rigid Case 2 (Form 7)

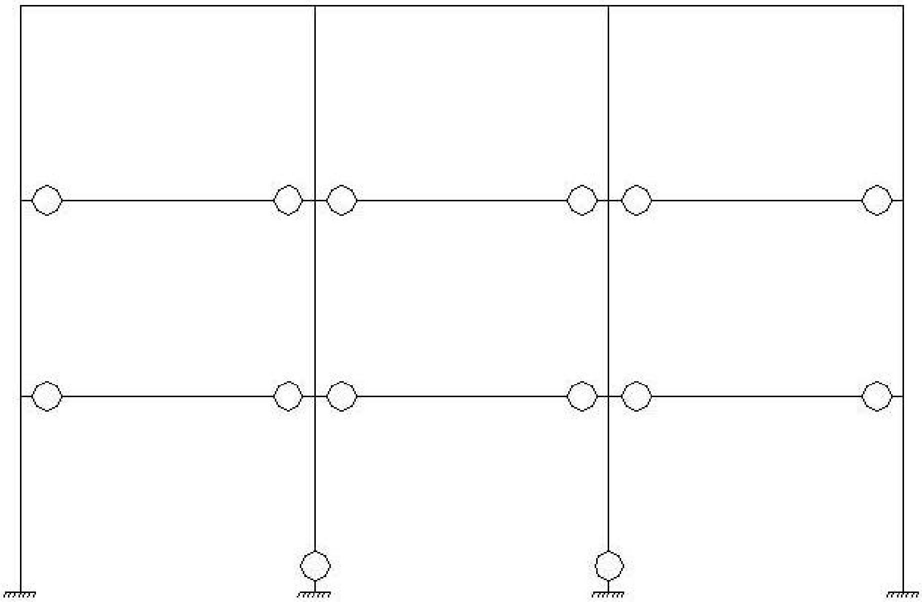


Figure 5-89: Hinge Formation for Form 7, Soil Type:Z1

(Base Shear: 4,4 t, Displacement: 22,7 cm)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-90, Figure 5-91 and Figure 5-92 respectively.

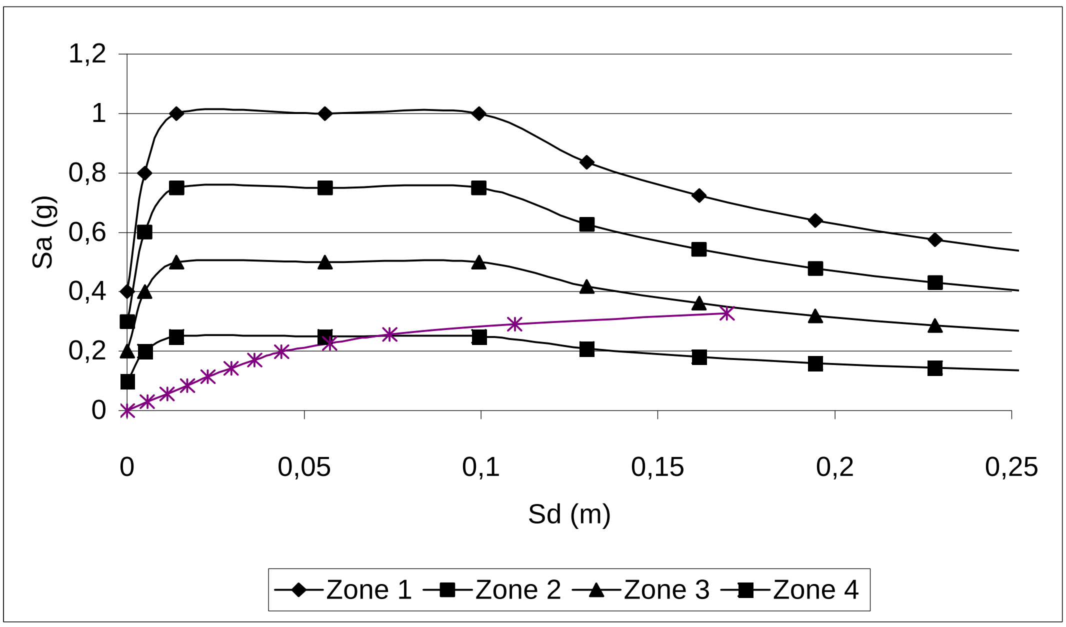


Figure 5-90: Capacity vs. Demand Spectra for Form 7, Soil Type:Z2

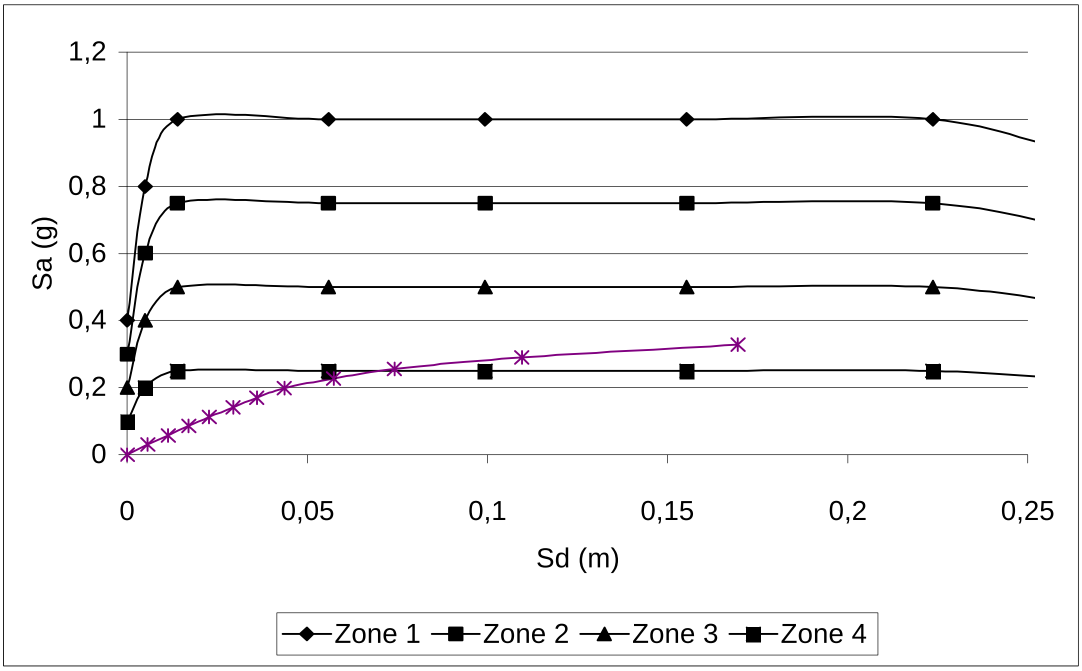


Figure 5-91: Capacity vs. Demand Spectra for Form 7, Soil Type:Z3

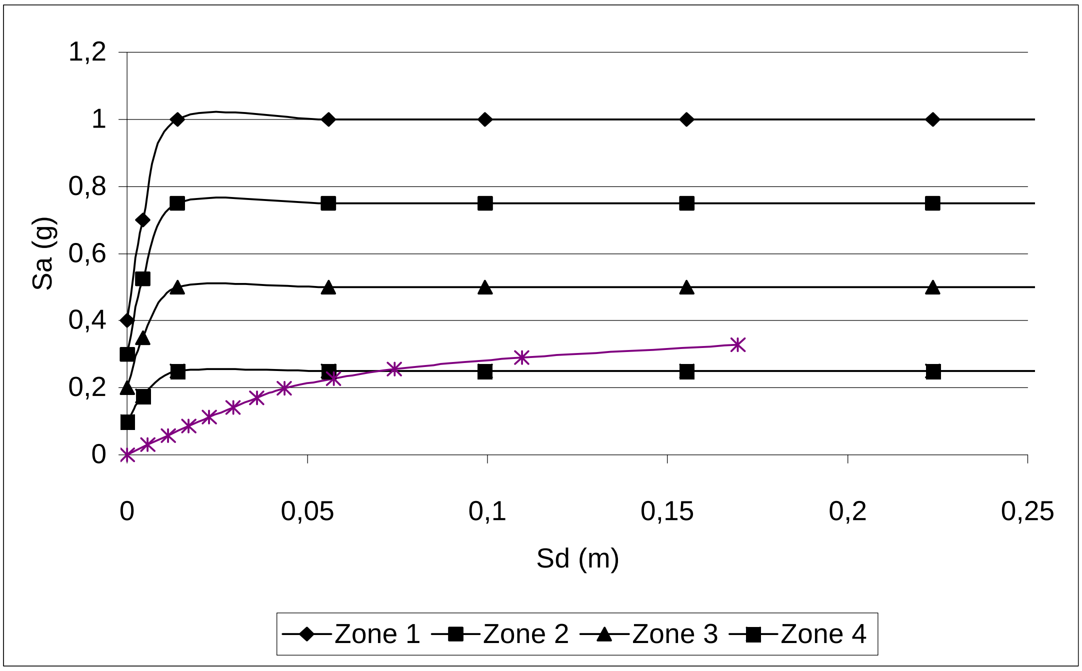


Figure 5-92: Capacity vs. Demand Spectra for Form 7, Soil Type:Z4

5.5 Performance Calculations for Case Study #3

5.5.1 Case Study #3, Form 1

Case Study #3 is a 3-floor, 1-bay steel frame with same geometric and loading properties. Again only frame data, capacity calculation tables and capacity-demand graphs will be presented at this part. General evaluations of the performances of case Study #3 frames will be discussed within the final conclusion part of this study.

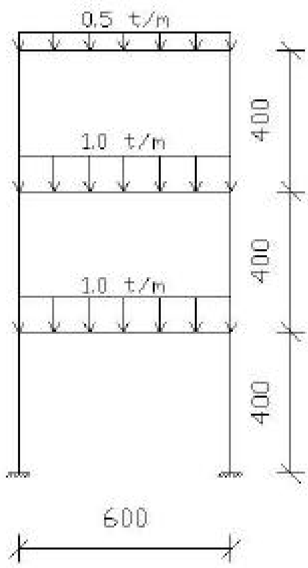


Figure 5-93: Loading and Geometry Data for Case Study #3

Table 5-16: Data for Performance Analysis for Case Study 3-Form 1

Number of Floors : 3 | Connections : Fixed | Load Increment : 400 kg

Number of Bays : 3 | P-Delta effect : Yes

Top Disp. (meter)	B. Shear (ton)	W_1 (ton)	W_2 (ton)	W_3 (ton)	f_1	f_2	f_3	PF1	α_1	S_d (meter)	S_a (meter/s^2)
0,0000	0,0000	2,187	2,187	1,1980				0,0000	0,0000	0,0000	0,0000
0,0000	0,0000	2,187	2,187	1,1980	0,2645	0,1888	1,0000	1,5321	0,6020	0,0000	0,0000
0,0154	0,4000	2,187	2,187	1,1980	0,3659	0,7916	1,0000	1,3034	0,8724	0,0118	0,0823
0,0309	0,8000	2,187	2,187	1,1980	0,3670	0,7925	1,0000	1,3028	0,8730	0,0237	0,1645
0,0464	1,2000	2,187	2,187	1,1980	0,3674	0,7928	1,0000	1,3025	0,8732	0,0356	0,2466
0,0619	1,6000	2,187	2,187	1,1980	0,3676	0,7929	1,0000	1,3024	0,8733	0,0475	0,3288
0,0774	2,0000	2,187	2,187	1,1980	0,3677	0,7930	1,0000	1,3024	0,8733	0,0594	0,4110
0,0928	2,4000	2,187	2,187	1,1980	0,3677	0,7930	1,0000	1,3023	0,8734	0,0713	0,4932
0,1083	2,8000	2,187	2,187	1,1980	0,3678	0,7931	1,0000	1,3023	0,8734	0,0832	0,5754
0,1238	3,2000	2,187	2,187	1,1980	0,3678	0,7931	1,0000	1,3023	0,8734	0,0951	0,6575
0,1393	3,6000	2,187	2,187	1,1980	0,3679	0,7931	1,0000	1,3023	0,8734	0,1069	0,7397

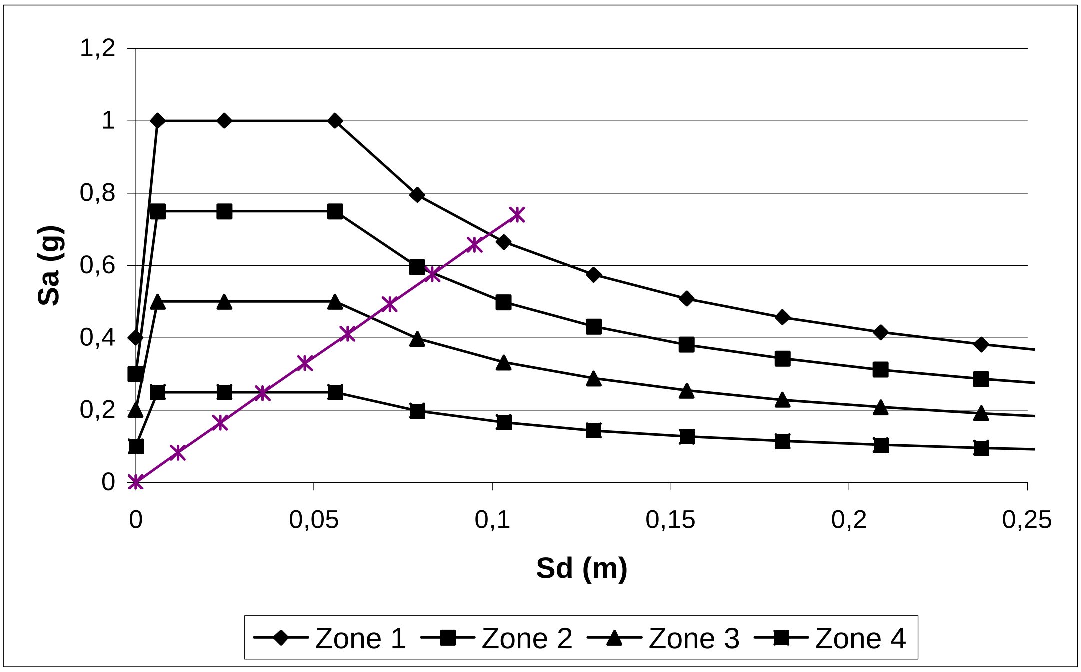


Figure 5-94: Capacity vs. Demand Spectra for Rigid Case 3 (Form 1)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-95, Figure 5-96 and Figure 5-97 respectively.

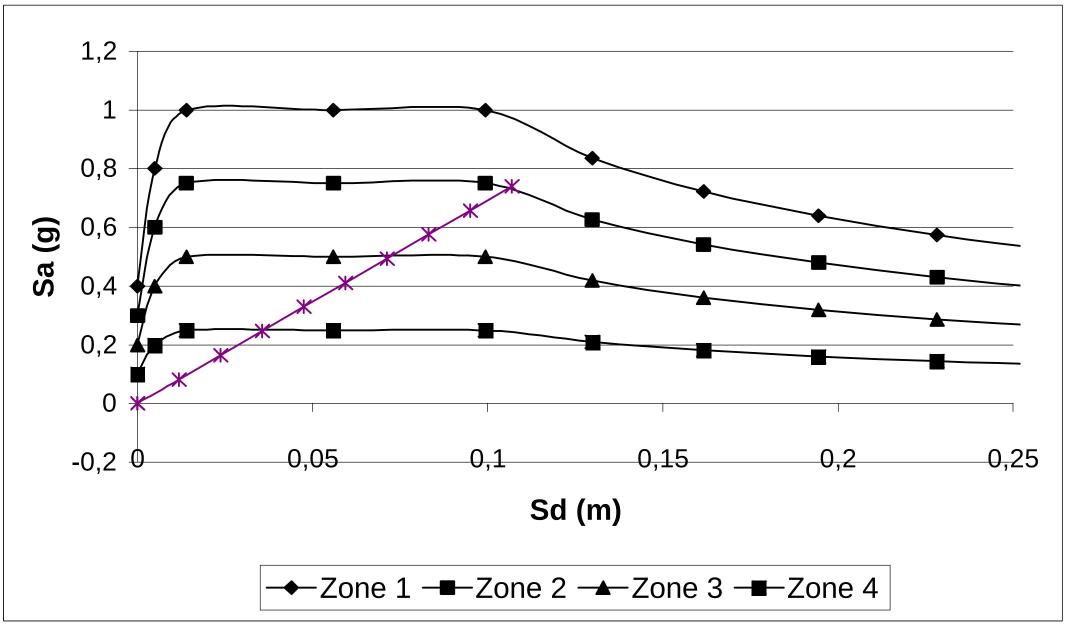


Figure 5-95: Capacity vs. Demand Spectra for Form 1, Soil Type:Z2

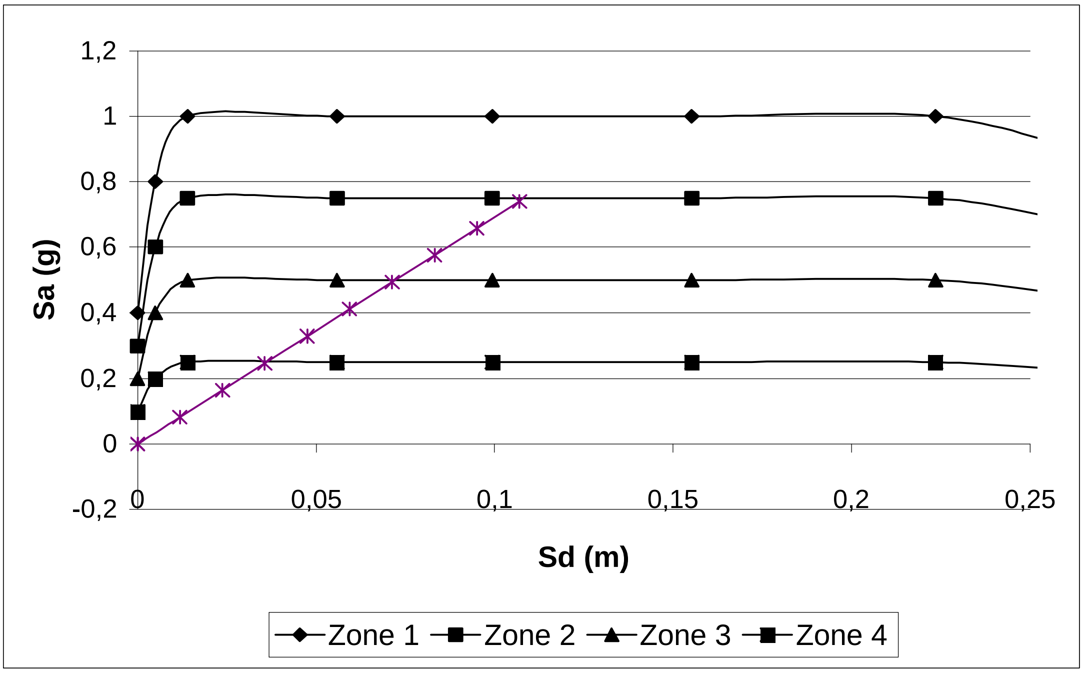


Figure 5-96: Capacity vs. Demand Spectra for Form 1, Soil Type:Z3

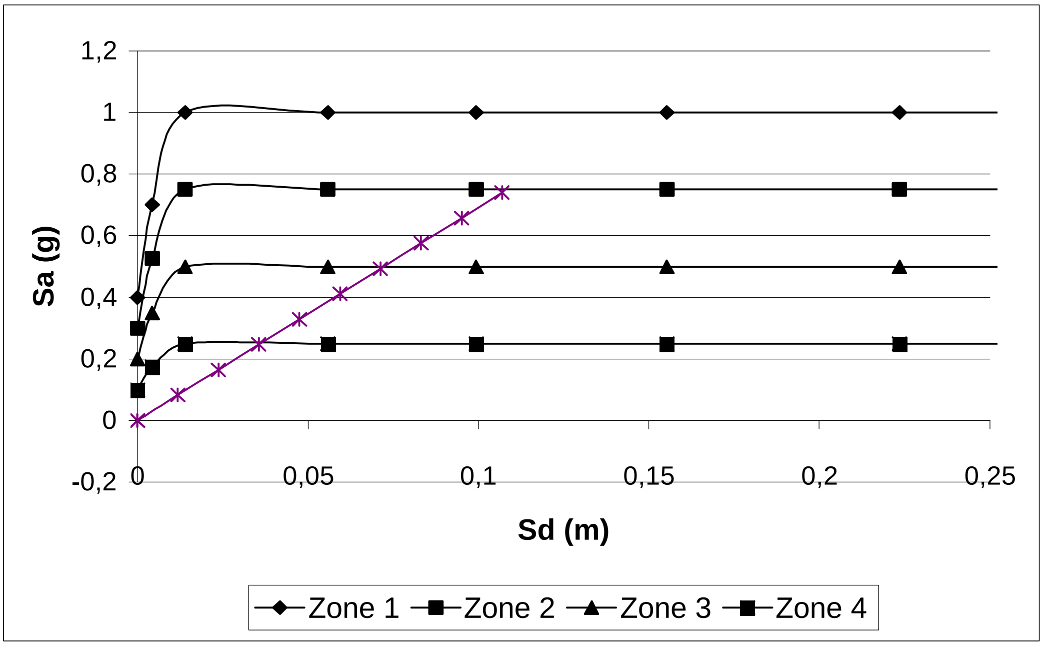


Figure 5-97: Capacity vs. Demand Spectra for Form 1, Soil Type:Z4

5.5.2 Case Study #3, Form 2

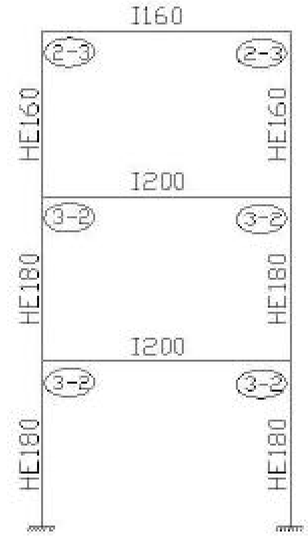


Figure 5-98 Semi Rigid Connection Types (Form 2)

Table 5-17: Data for Performance Analysis for Case Study 3-Form 2

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 400 kg

Number of Bays : 1 **P-Delta effect :** Yes

Top Disp. (meter)	B. Shear (ton)	W_1 (ton)	W_2 (ton)	W_3 (ton)	f_1	f_2	f_3	PF1	α_1	S_d (meter)	S_a (meter/s^2)
0,0000	0,0000	2,187	2,187	1,1980				0,0000	0,0000	0,0000	0,0000
0,0191	0,4000	2,187	2,187	1,1980	0,2645	0,7468	1,0000	1,3264	0,8117	0,0144	0,0884
0,0398	0,8000	2,187	2,187	1,1980	0,3313	0,7459	1,0000	1,3386	0,8538	0,0297	0,1682
0,0640	1,2000	2,187	2,187	1,1980	0,3268	0,7426	1,0000	1,3409	0,8511	0,0477	0,2530
0,0940	1,6000	2,187	2,187	1,1980	0,3191	0,7360	1,0000	1,3455	0,8465	0,0698	0,3392
0,1330	2,0000	2,187	2,187	1,1980	0,3074	0,7243	1,0000	1,3535	0,8392	0,0983	0,4277
0,1864	2,4000	2,187	2,187	1,1980	0,2911	0,7059	2,0000	0,7543	0,6195	0,2472	0,6952
0,2622	2,8000	2,187	2,187	1,1980	0,2714	0,6816	3,0000	0,4748	0,4839	0,5523	1,0386

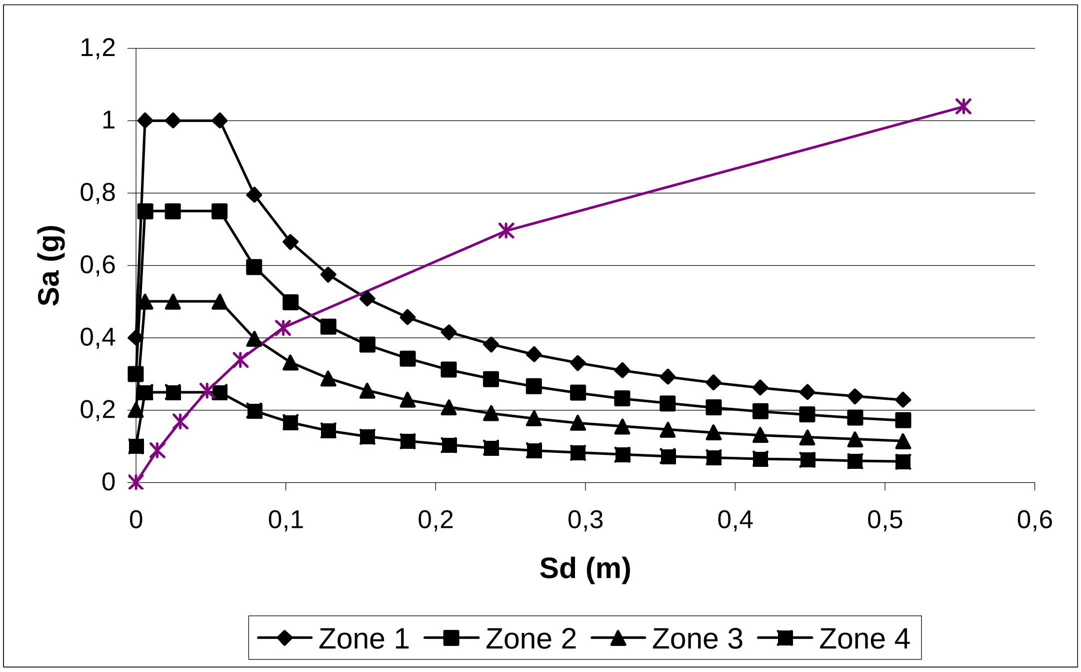


Figure 5-99: Capacity vs. Demand Spectra for Semi-Rigid Case 3 (Form 2)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-100, Figure 5-101 and Figure 5-102 respectively.

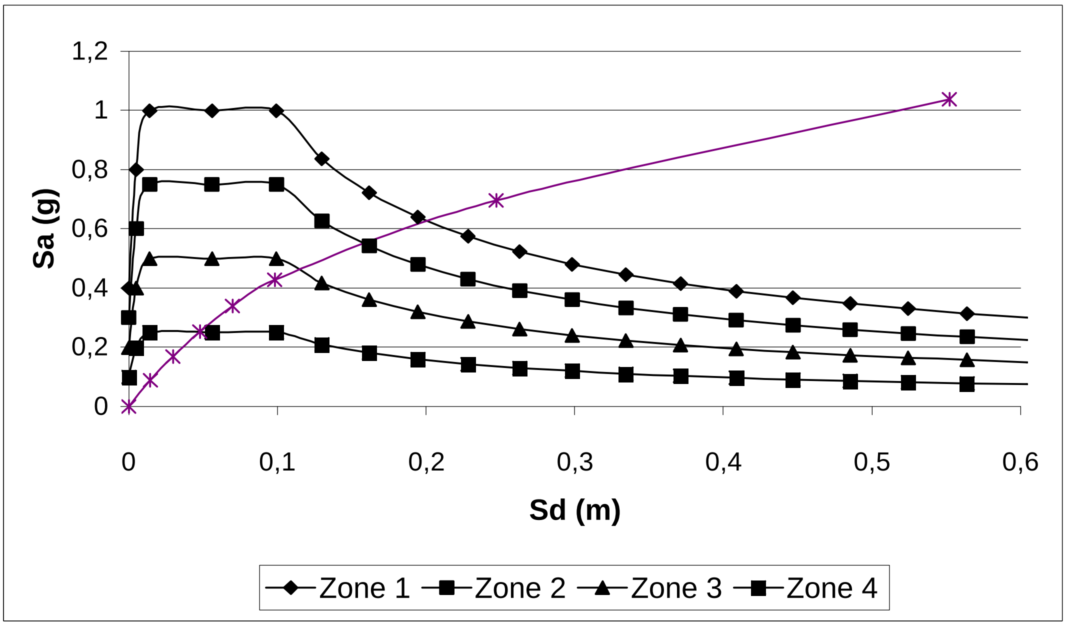


Figure 5-100: Capacity vs. Demand Spectra for Form 2, Soil Type:Z2

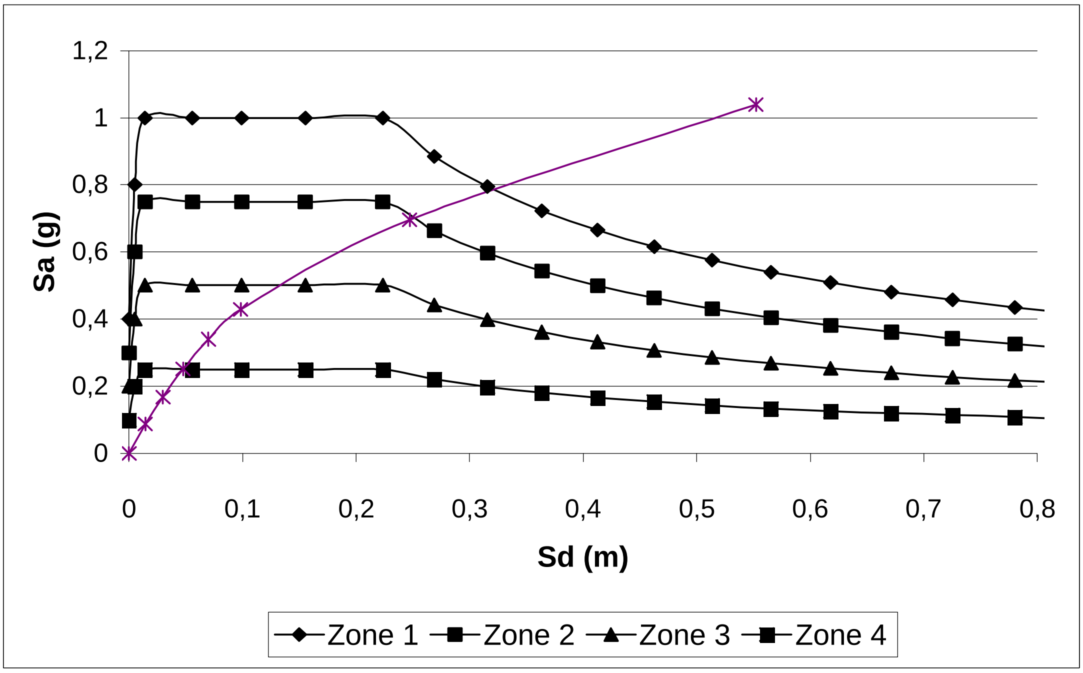


Figure 5-101: Capacity vs. Demand Spectra for Form 2, Soil Type:Z3

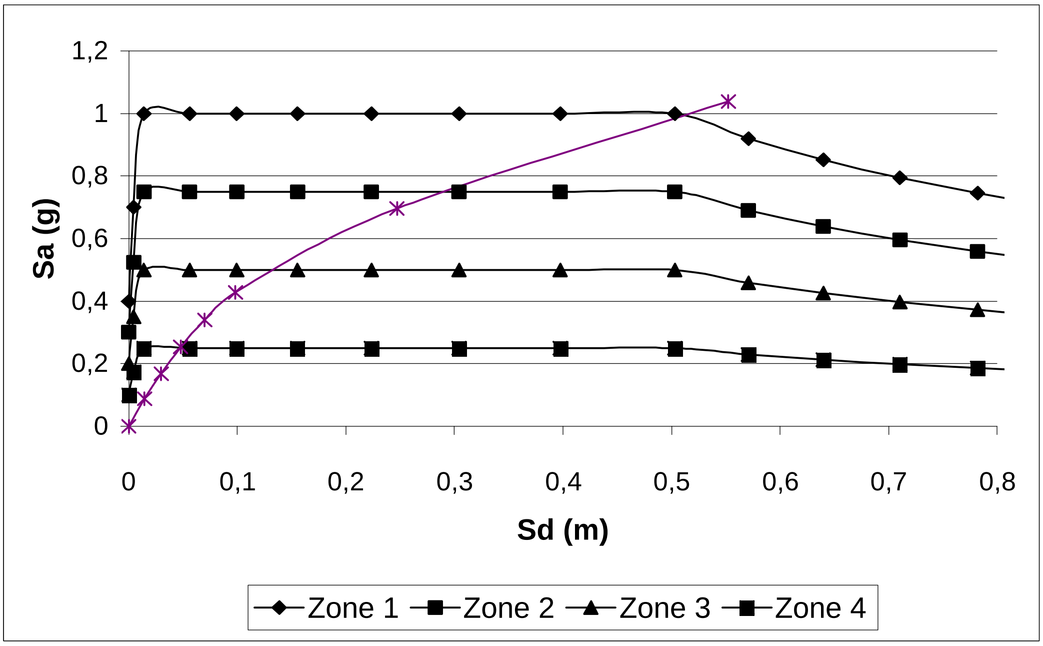


Figure 5-102: Capacity vs. Demand Spectra for Form 2, Soil Type:Z4

5.5.3 Case Study #3, Form 3

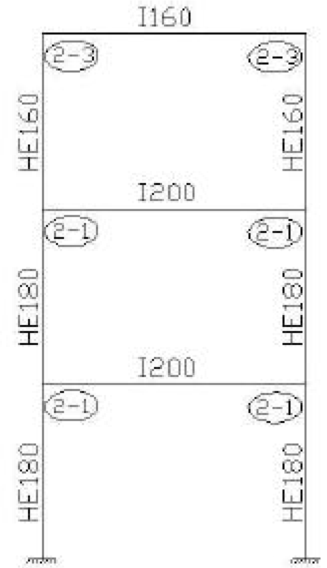


Figure 5-103 Semi Rigid Connection Types (Form 3)

Table 5-18: Data for Performance Analysis for Case Study 3-Form 3

Number of Floors : 3 **Connections :** Semi-Rigid **Load Increment :** 200 kg

Number of Bays : 1 **P-Delta effect :** Yes

Top Disp.	***Base Shear***	***W_1***	***W_2***	***W_3***	ϕ_1	ϕ_2	ϕ_3	***PF1***	α_1	***S_d***	***S_a***
0,0000	0,0000	2,187	2,187	1,1980				0,0000	0,0000	0,0000	0,0000
0,0174	0,2000	2,187	2,187	1,1980	0,2645	0,7260	1,0000	1,3437	0,8113	0,0129	0,0442
0,0372	0,4000	2,187	2,187	1,1980	0,2878	0,7236	1,0000	1,3509	0,8267	0,0275	0,0868
0,0615	0,6000	2,187	2,187	1,1980	0,2819	0,7184	1,0000	1,3540	0,8227	0,0455	0,1309
0,0971	0,8000	2,187	2,187	1,1980	0,2724	0,7095	1,0000	1,3593	0,8161	0,0714	0,1759
0,1494	1,0000	2,187	2,187	1,1980	0,2575	0,6924	1,0000	1,3696	0,8051	0,1091	0,2229
0,2318	1,2000	2,187	2,187	1,1980	0,2391	0,6674	2,0000	0,7432	0,5840	0,3119	0,3688
0,3389	1,4000	2,187	2,187	1,1980	0,2235	0,6433	3,0000	0,4654	0,4585	0,7283	0,5480

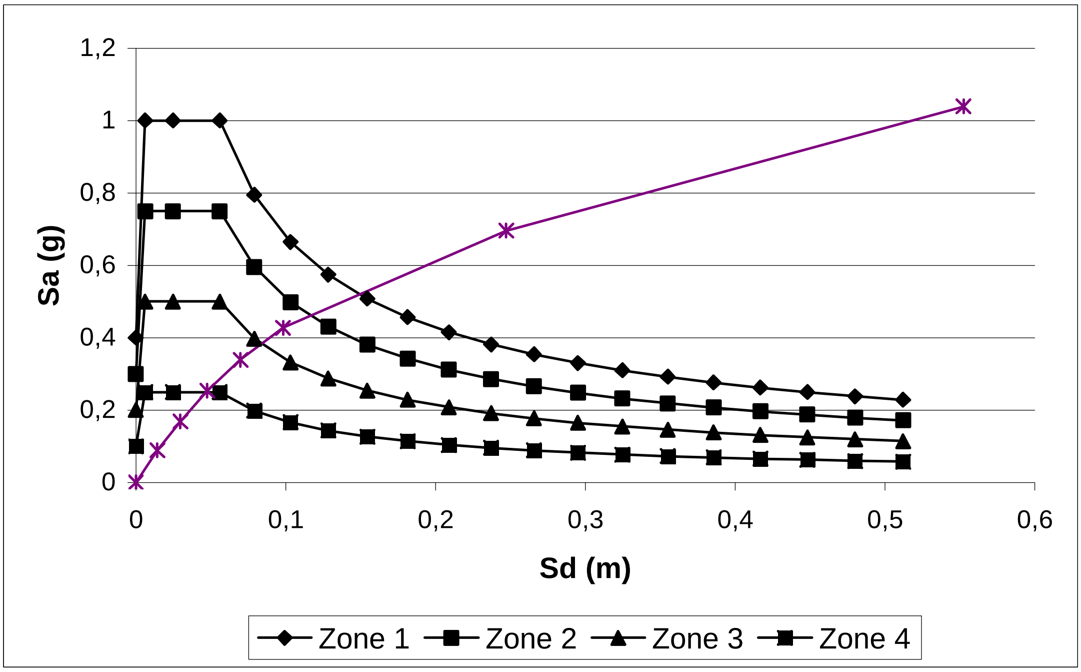


Figure 5-104: Capacity vs. Demand Spectra for Semi-Rigid Case 3 (Form 1)

Performance of the frame for Soil Type 2, 3, 4 can be found in Figure 5-105, Figure 5-106 and Figure 5-107 respectively.

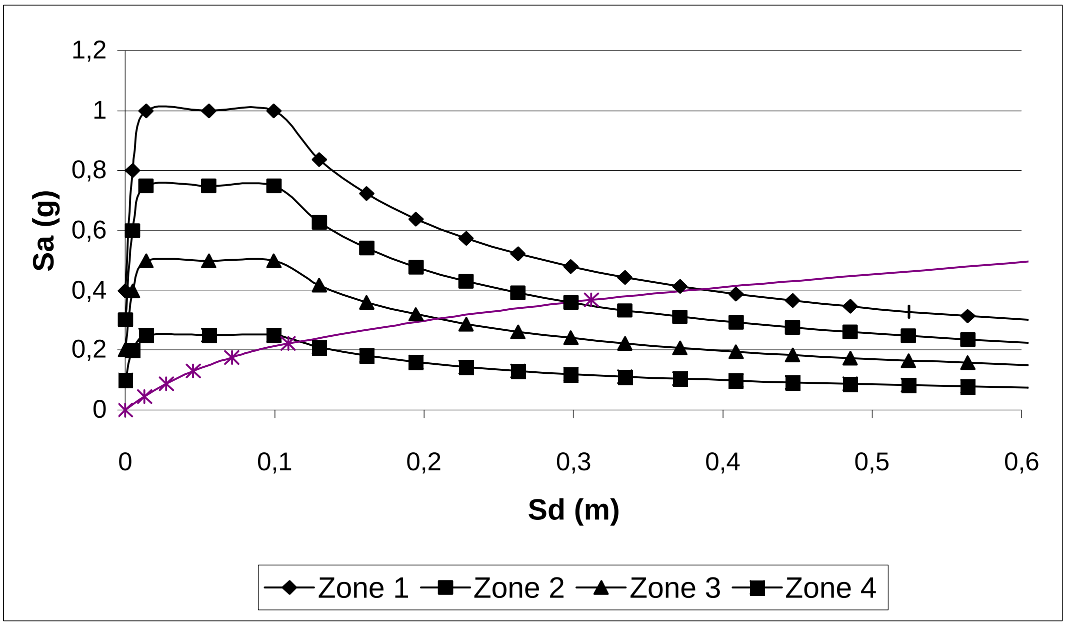


Figure 5-105: Capacity vs. Demand Spectra for Form 3, Soil Type:Z2

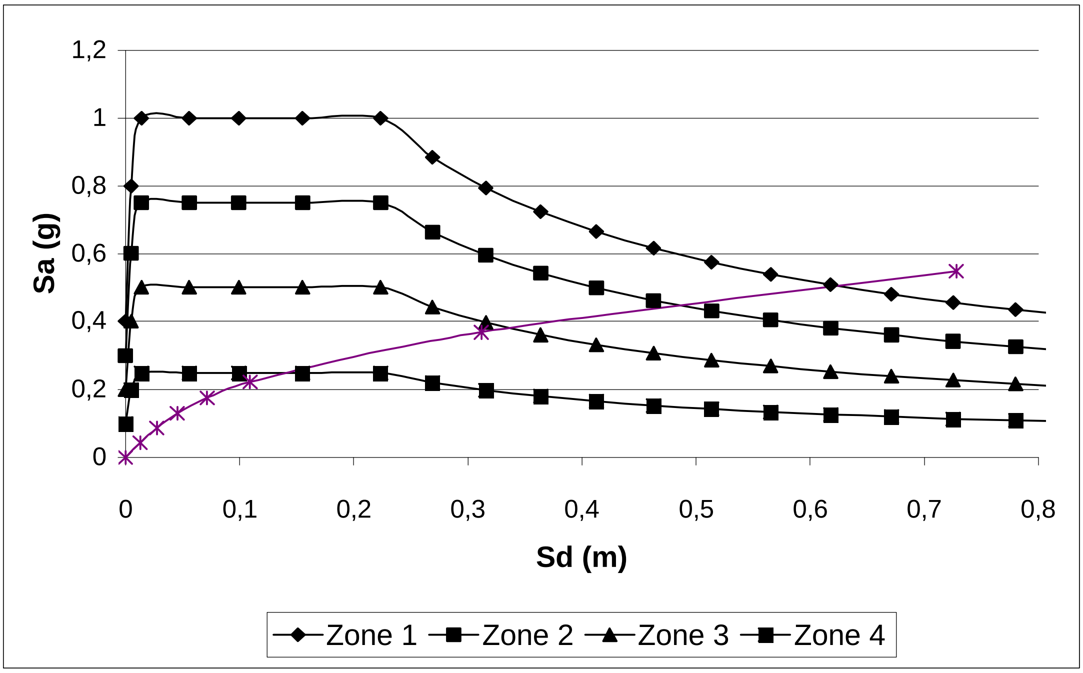


Figure 5-106: Capacity vs. Demand Spectra for Form 3, Soil Type:Z3

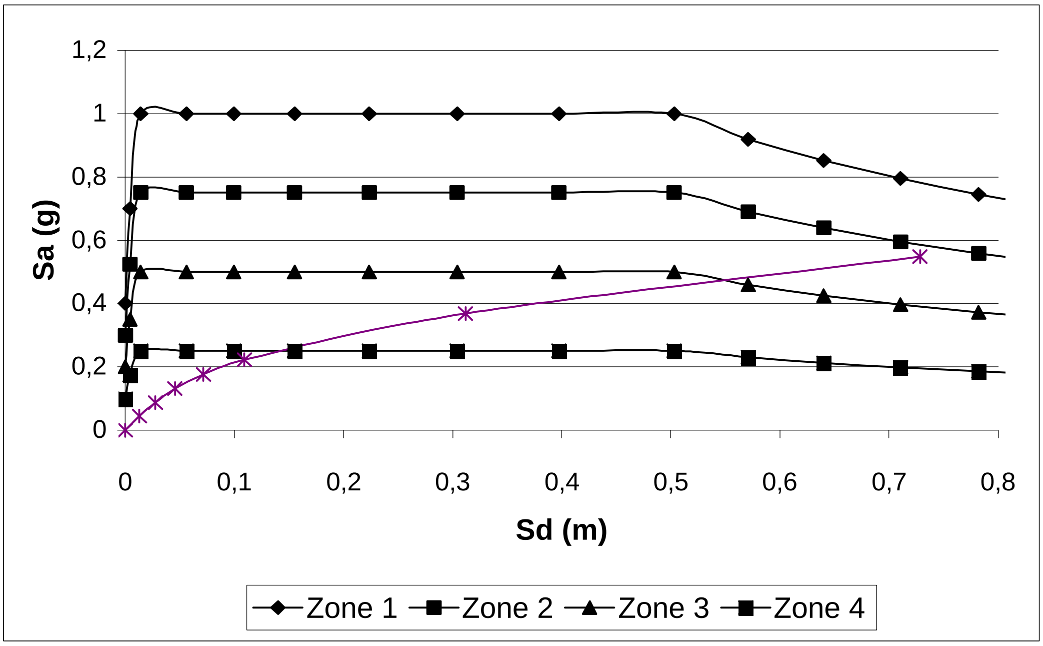
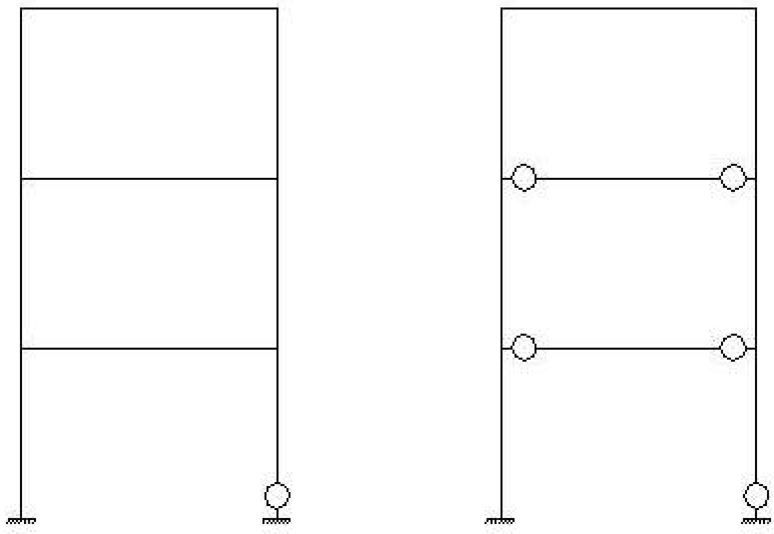


Figure 5-107: Capacity vs. Demand Spectra for Form 3, Soil Type:Z4



(a) **(b)**

Figure 5-108: Hinge Formation for Case Study #3, Soil Type:Z1, (a) Form 1 and Form 2, (b) Form 3

CHAPTER 6

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusion

In this study, seismic performances of 2-D steel frames were evaluated according to Capacity Spectrum Method proposed in ATC-40, 1996. This study was performed for frames with semi-rigid connections and for rigidly connected frames in order to compare the assumed and real behaviours.

It is observed that, the differences in the behaviours of steel frames with different bay numbers due to variations in the connections are close to each other in case studies. The following conclusions can be valid for the case studies studied for the stated loading, geometry and constant damping value.

- Generally, the performances of frames with semi-rigid connections are better than the ones that are rigidly connected. In other words, since all connections used in practice are semi-rigid in some amount, it can be said that making nonlinear analysis with appreciate assumptions will guide to the designer on the way of getting true and relatively strong response of the frame. In the comparison of rigid and semi-rigid frames, although both frame can resist an earthquake that occur in EQ Zone 1, corresponding accelerations and displacements are totally different. By rigid assumption, the designer can get the acceleration value 25% more than the actual one. As a consequent of this assumption, the member sections are selected greater than those needed. Therefore optimum designs can be obtained by working on semi-rigid connections.

- In case of analysis of frames with semi-rigid connections, the designer should control the serviceability limits carefully, since lateral deformations of semi-rigidly connected frames increase considerably relative to ones for rigid frames. According to the comparison of ultimate displacements of frames with rigid and semi-rigid connections, top displacement for semi-rigid case can be up to 60% greater than the displacement of the rigid frame. Therefore, the designer should take the increase in the lateral displacement in addition to the decrease in the ground acceleration.

- Considering the performance-displacement criteria, it is seen that displacement values are quite high. However, these displacements are not considered in the scope of serviceability, since the presumed performance level was selected as Life Safety Level.

- One of the designative criteria for the design of structures is the energy absorption capacity. In the comparison of energy dissipation of frames by considering the area under capacity curve, energy absorption of semi-rigid frames is greater than the rigid frames up to 40%. This is mainly due to the ductile behaviour of the structure.

- According to the performances of frames, it is seen that, performances of semi-rigid frames can be affected oppositely beyond such a low rigidity. So the idea “The lower rigidity, the higher performance” cannot be valid for all cases.

- According to the results of case studies, the performances of steel frames , in which the members are fastened by top and seat angle connections with double web angle, seems to be the most convenient one. But this thesis cannot be generalized, because, in addition to the connection type, the size of profiles, bolts and some other parameters affect the performance of the frame.

- Since the difference in soil zone affects the demand curve entirely, similar to the rigid frames, performances of frames with semi-rigid connections are affected negatively with worse soil conditions.

Consequently, in order to make the most suitable design for a seismic and soil region where the structure to be constructed, necessary studies on the connection details should be performed in order to achieve desired performance, serviceability and optimum member criterias.

6.2 Future Recommendations

Based on this study, the following recommendations can be made;

- A study on the performances of frames with other types of semi-rigid connections can be performed.

- The response of frames against cyclic loading can be investigated.

- The results of this study can be investigated and verified by experimental studies on steel frames with semi-rigid connections.

- All these performance evaluations can be studied for 3-D steel frames.

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APPENDIX A

INPUT DATA FOR THE SAMPLE FRAME DISCUSSED IN CHAPTER 3

Frame Geometry Data

1. Number of Floors = 4
2. Number of Bays = 2
3. Value of Modulus of Elasticity = 20389020 t/m^2
4. Floor Heights = 3,6576 m
5. Bay Widths = 9,144 m
6. For different column members = 1
7. Area, Depth and Inertia of the 1st Floor Columns (W12x79) = 0,015 m^2, 0.20 m, 0,0002755 m^4
8. Area, Depth and Inertia of the 2nd, 3rd and 4th Floor Columns (W12x65) = 0,0123 m^2, 0.20 m, 0,0002219 m^4
   (Different from the reference question, the mid-columns are input as W12x65 instead of W12x79 due to a restriction in the program)
9. For different beam members = 1
10. Area, Depth and Inertia of the 1st, 2nd and 3rd Bay Beams (W16x40) = 0.007613 m^2, 0.20 m, 0,0002156 m^4
11. Area, Depth and Inertia of the 4th Bay Beams (W14x30) = 0,0571 m^2, 0.20 m, 0,0001211 m^4
12. Base Constraints of 1st Floor Columns = 2 for all of them (1=hinge, 2= fixed)

Frame Loading Data

13. To Input Point Loads = 1
14. Floor and Bay Number of the Relevant Joint =1,1
15. Direction of Loading (1 for X-dir, 2 for Y-dir, 3 for Moment) = 1
16. Value of Loading = 3,17485 t for 1st, 2nd and 3rd floors and 1,5874 for the 4th floor
17. To input point Load Again = 1 for proceed
    (All point load assignment is completed in the loop between steps 14 and 17.)
18. To Input Distributed Loads = 1
19. Floor and Bay Number of the Beam Member =1,1
20. Value of Loading = 2,6784 t/m 1st, 2nd and 3rd floors and 1,3392 for the 4th floor
21. To Input Distributed Loads Again = 1
    (All point load assignment is completed in the loop between steps 19 and 21.)
22. To Take Self Weight into Account = 0
23. To Take EQ Loads into Account =0

The input part for rigid-frame analysis ends at this point.

24. To Take P-d Effect Into Account = 1

The input part for rigid-frame analysis with P-D effect ends at this point.

Data for the analysis with Semi-Rigid Connection

25. To Input Semi-Rigid Connection Data = 1
26. Floor and Bay Numbers of the Semi-Rigid Connections = 1,1
27. Type of Semi-Rigid Connection = 3
28. Semi-Rigid Connection Parameters = 13 in. for *d* and 0,7875 in. for *t* (These parameters are given in terms of inch at this point in order to be used in the relevant formulas. Finally they're converted to SI units)
29. To Input Semi-Rigid Connection Data Again = 1 (All Semi-Rigid connection data assignment is completed in the loop between steps 26 and 28.)
30. Number of Load increments (Total Number of Iterations for nonlinear Analysis)

APPENDIX B

```
C     ***************************************************************
C     ***************************************************************
C     *************          SRFP            *********************
C     ***************************************************************
C     ***************************************************************
C     ***************************************************************
C     ***************************************************************
C     ***************************************************************
C     ***************************************************************
      IMPLICIT REAL*8 (A-H,O-Z)
      DIMENSION IBASE(5),SAYAC(100),EIHGHT(6),EIWDTH(6),AC(30,30),DC(30,
     *30),AIC(30,30),AB(30,30),DB(30,30),AIB(30,30),AKC(6,7,6,6),AKB(6,6
     *,6,6),ADEG(100,100),PAPP(100),PPOINT(100),PDIST(100),PSELF(100),PS
     *ELFC(100),EQWC(6),EQWB(6),EQW(6),EQKAT(6),PAPPEQ(100),TOPLAM(100),
     *D(100),PK(7,7,6),PB(6,6,6),GKC(6,6,6,6),PAPP2(100),PAPPSR(100),C1(
     *6,6),C2(6,6),C3(6,6),AKAPA(6,6),ROTAT3(6,6),ROTAT6(6,6),ROTZ3(6,6)
     *,ROTZ6(6,6),DELTA3(6,6),DELTA6(6,6),AMKIP3(6,6),AMKIP6(6,6),ROT3(6
     *,6),ROT6(6,6),KONTRO(6,6),PBSR(6,6,6),PKSR(6,7,6),DSRGD(100),SRSTF
     *3(6,6),SRSTF6(6,6),SWAY(100),SHEAR(100),PKIRIS(6,6,6),PDKRS(6,6,6)
     *,THGHT(6),UMC(6,6),DEFOR(6),LLOAD(6,6),SRS3(6,6),SRS6(6,6)

      OPEN (5,FILE='3X5.SOU',STATUS='OLD')
      OPEN (10,FILE='DATA.OUT',STATUS='UNKNOWN')
      OPEN (11,FILE='INPUT.OUT',STATUS='UNKNOWN')
      OPEN (12,FILE='EXCEL.OUT',STATUS='UNKNOWN')
      OPEN (14,FILE='MODLAR.TXT',STATUS='UNKNOWN')
      OPEN (15,FILE='YUK.TXT',STATUS='UNKNOWN')
      OPEN (16,FILE='KLN.TXT',STATUS='UNKNOWN')


      NITNO=0

      DO 271 I=1,100
      PAPP(I)=0.0
      PPOINT(I)=0.0
      PDIST(I)=0.0
      PSELF(I)=0.0
      PSELFC(I)=0.0
      PAPPEQ(I)=0.0
  271 CONTINUE

      DO 327 I=1,NFLR
      DO 328 J=1,NBAY+1
      DO 329 K=1,6
      DO 330 L=1,6
      GKC(I,J,K,L)=0
  330 CONTINUE
  329 CONTINUE
  328 CONTINUE
  327 CONTINUE
      DO 430 I=1,NFLR
      DO 431 J=1,NBAY
      DO 432 K=1,6
      PBSR(I,J,K)=0
  432 CONTINUE
  431 CONTINUE
  430 CONTINUE

      SAYAC(1)=1
      DO 116 I=2,100
  116 SAYAC(I)=SAYAC(I-1)+1
```

```
C     INPUT OF FLOOR AND BAY INFORMATION

      WRITE(6,117)
  117 FORMAT(5X,'PLEASE INPUT # OF FLOORS...(MAX IS 6)')
  153 READ(5,103) NFLR
      IF(NFLR.LE.6.AND.NFLR.NE.0) GO TO 152
      WRITE(6,147)
      GO TO 153
  147 FORMAT(/,5X,'MAX #IS 6!..INPUT AGAIN...')
  152 WRITE(6,118) NFLR
  118 FORMAT(5X,'# OF FLOOR...',I1)
      WRITE(6,119)
  119 FORMAT(/,5X,'PLEASE INPUT # OF BAYS...(MAX IS 6)')
  154 READ(5,103) NBAY
      IF(NBAY.LE.6) GO TO 149
      WRITE(6,147)
      GO TO 154
  149 WRITE(6,120) NFLR
  120 FORMAT(5X,'# OF BAY...',I1)
      NCOL=(NBAY+1)*NFLR
      NBEAM=NBAY*NFLR


C     MODULUS OF ELASTICITY

      WRITE(6,101)
  101 FORMAT(/,5X,44HPLEASE INPUT MODULUS OF ELASTICITY (t/m2)...)
  103 FORMAT(I1)
  275 FORMAT(F8.0)
      READ(5,275) E
  102 FORMAT(5X,'MODULUS OF ELASTICITY...',F8.0,' t/m2')
      WRITE(6,102) E

C     INPUT FOR HEIGHT  &  WIDTH

  123 FORMAT(/,10X,'FLOOR HEIGHTS',/,10X,'--------------')
      WRITE(6,123)

      DO 121 I=1,NFLR
      WRITE(6,104) SAYAC(I)
 1111 FORMAT(F6.4)
  104 FORMAT(/,5X,'PLEASE INPUT HEIGHT FOR FLOOR #',I2,'...(m)')
      READ(5,1111) EIHGHT(I)
  105 FORMAT(5X,'HEIGHT FOR FLOOR ',I2,'...',F6.4,' m.')
      WRITE(6,105) SAYAC(I),EIHGHT(I)
  121 CONTINUE

  124 FORMAT(/,10X,'BAY WIDTHS',/,10X,'-----------')
      WRITE(6,124)

      DO 122 I=1,NBAY
      WRITE(6,106) SAYAC(I)
  106 FORMAT(/,5X,'PLEASE INPUT WIDTH FOR BAY#',I2,'...(m)')
      READ(5,1111) EIWDTH(I)
  107 FORMAT(5X,'WIDTH FOR BAY',I2,'...',F6.4,' m.')
      WRITE(6,107) SAYAC(I),EIWDTH(I)
  122 CONTINUE

C     SECTION PROPERTIES

  125 FORMAT(/5X,'TOTAL # OF COLUMNS IS=',I2,/,5X,'TOTAL # OF BEAMS IS=
     *',I2,/,5X,'IF ALL COLUMN SECTION PROPERTIES ARE DIFFERENT, ENTER 1
     *...')
      WRITE(6,125) NCOL,NBEAM
      READ(5,103) ISECHKC
      IF(ISECHKC.EQ.1) GO TO 130

C     IF SECTIONS ARE SAME

  126 WRITE(6,108)
  108 FORMAT(/,5X,'PLEASE INPUT AREA(m2),DEPTH(m),AND INERTIA(m4) OF THE
```

```
     * COLUMN...',/,5X,'-----------------------------
     *-------------------',/,5X)
      READ(5,109) ARC,DEC,AINC
      DO 167 I=1,NFLR
      DO 166 J=1,(NBAY+1)
      AC(I,J)=ARC
      DC(I,J)=DEC
      AIC(I,J)=AINC
  166 CONTINUE
  167 CONTINUE
  110 FORMAT(/,15X,'F O R   A L L  C O L U M N S !', /,/,5X,'AREA(m2)= '
     *,F8.7,3X,'DEPTH(m)= ',F8.8,3X,'INERTIA(*m4)= ',F10.9)
      WRITE(6,110) ARC,DEC,AINC
  109 FORMAT(3F10.9)

  140 WRITE(6,127)
  127 FORMAT(/,5X,'IF ALL BEAM SECTION PROPERTIES ARE DIFFERENT, ENTER 1
     *...')
      READ(5,103) ISECHKB
      IF(ISECHKB.EQ.1) GO TO 142

  128 WRITE(6,111)
  111 FORMAT(/,5X,'PLEASE INPUT AREA(m2),DEPTH(m),AND INERTIA(m4) OF THE
     *BEAM...',/,5X,'---------------------------------------------------
     *-------------------',/,5X)
      READ(5,109) ARB,DEB,AINB
      DO 169 I=1,NFLR
      DO 168 J=1,NBAY
      AB(I,J)=ARB
      DB(I,J)=DEB
      AIB(I,J)=AINB
  168 CONTINUE
  169 CONTINUE
  129 FORMAT(/,15X,'F O R   A L L  B E A M S !',/, /,5X,'AREA(m2)= ',F8.
     *7,3X,'DEPTH(m)= ',F8.8,3X,'INERTIA(m4)= ',F10.9)
      WRITE(6,129) ARB,DEB,AINB
      GO TO 155

C     IF SECTIONS ARE DIFFERENT

C     FOR COLUMNS

  130 WRITE(6,131)
  131 FORMAT(/,/,20X,'I N  C A S E  O F  N U M B E R I N G ! . .      '
     *,/,/,/,5X,'FLOOR LEVELS ARE NUMBERED FROM BOTTOM TO TOP!..      ',/,
     */,5X,'BEAMS,BAYS AND COLUMNS ARE NUMBERED FROM LEFT TO RIGHT!..')

      DO 133 I=1,NFLR
  132 FORMAT(///,5X,'PLEASE INPUT AREA(m2),DEPTH(m),AND INERTIA(m4) FOR
     *THE COLUMNS OF FLOOR #',I2,'...',/,5X)
      WRITE(6,132) SAYAC(I)
      READ(5,109) AREAC,DEPTHC,AINRTC
      DO 141 J=1,(NBAY+1)
      AC(SAYAC(I),J)=AREAC
      DC(SAYAC(I),J)=DEPTHC
      AIC(SAYAC(I),J)=AINRTC

  139 FORMAT(/,5X,'FLOOR',3X,'COLUMN #',3X,'AREA(m2)',3X,'DEPTH(m)',3X,'
     *INERTIA(m4)',/5X,'-----',3X,'--------',3X,'--------',3X,'--------'
     *,3X,'-----------',/,5X,F5.0,3X,F5.0,3X,F9.3,2X,F9.3,4X,F9.4)
      WRITE(6,139) SAYAC(I),J,AC(SAYAC(I),J),DC(SAYAC(I),J),AIC(SAYAC(I)
     *,J)
  141 CONTINUE
  133 CONTINUE
      GO TO 140

C     FOR BEAMS

  142 WRITE(6,131)

      DO 143 I=1,NFLR
```

```
  144 FORMAT(///,5X,'PLEASE INPUT AREA(m2),DEPTH(m),AND INERTIA(m4) FOR
     *THE BEAMS OF FLOOR #',I2,'...',/,5X)
      WRITE(6,144) SAYAC(I)
      READ(5,109) AREAB,DEPTHB,AINRTB
      DO 145 J=1,NBAY
      AB(SAYAC(I),J)=AREAB
      DB(SAYAC(I),J)=DEPTHB
      AIB(SAYAC(I),J)=AINRTB
  146 FORMAT(/,5X,' FLOOR ',3X,' BEAM  #',3X,'AREA(m2)',3X,'DEPTH(m)',3X
     *,'INERTIA(m4)',/5X,'-----',3X,'--------',3X,'--------',3X,'-------
     *-',3X,'-----------',/,5X,F5.0,3X,F5.0,3X,F9.3,2X,F9.3,4X,F9.4)
      WRITE(6,146) SAYAC(I),J,AB(SAYAC(I),J),DB(SAYAC(I),J),AIB(SAYAC(I)
     *,J)
  145 CONTINUE
  143 CONTINUE

C     COLUMN BASE CONSTRAINTS

  155 WRITE(6,113)
  113 FORMAT(/,5X,'-------------------------------------------------------
     *-------',/,/,5X,'PLEASE ENTER COLUMN BASE CONSTRAINTS...(1 FOR HIN
     *GE,2 FOR FIXED)',/,5X)
      DO 112 I=1,(NBAY+1)
  156 READ(5,115) IBAS
      IF(IBAS.EQ.1) GO TO 150
      IF(IBAS.EQ.2) GO TO 150
  151 FORMAT(/,5X,'PLEASE INPUT EITHER 1 OR 2 !..')
      WRITE(6,151)
      GO TO 156
  150 IBASE(I)=IBAS
  115 FORMAT(I1)
  114 FORMAT(5X,'BASE OF COLUMN #',I1,' = ',I2,/,5X)
      WRITE(6,114) SAYAC(I),IBASE(I)
  112 CONTINUE

      NDEG2=0
      DO 305 J=1,NBAY+1
      IF(IBASE(J).EQ.1) THEN
         NDEG2=NDEG2+1
      ENDIF
  305 CONTINUE



C     ***************  # OF DOF  **************************
      NDEG3=0
      NDEG3=NFLR*(NBAY+1)*3
      NDEG4=NDEG3
      NDEG5=NDEG4
      NDEG=NFLR*(NBAY+1)*3+NDEG2
      WRITE(6,304) NDEG


C     ***********************************************************
C                              LOADS
C     ***********************************************************
      DO 231 I=1,NDEG
      PAPP(I)=0.0
  231 CONTINUE

C     *****     POINT LOADS  ***************************************

      WRITE(6,232)
  232 FORMAT(/,5X,'-------------------------------------------------------
     *-------',/,/,5X,'PLEASE PRESS (1) TO INPUT POINT LOADS...)',/,5X)
      READ(5,115) NPOINTL
      IF(NPOINTL.NE.1) GO TO 214
  215 FORMAT(/,5X,'INPUT FLOOR AND BAY NUMBERS RESPECTIVELY...')
  233 WRITE(6,215)
  216 FORMAT(2I2)
      READ(5,216) NFLORP,NBAYP
```

```
  217 FORMAT(/,5X,'INPUT 1,2 OR 3 FOR X,Y,Z DIRECTIONS RESPECTIVELY...')
      WRITE(6,217)
      READ(5,115) NDIRP
      NJOINT=NFLORP+((NFLORP-1)*NBAY)+NBAYP-1

      WRITE(6,218)
  218 FORMAT(/,5X,'ENTER FORCE OR MOMENT FOR THE SELECTED JOINT...')
  219 FORMAT(F8.4)
      READ(5,219) PLOAD
      PPOINT(3*NJOINT-3+NDIRP)=PLOAD
  220 FORMAT(/,5X,'TO INPUT AN INPUT LOAD AGAIN PRESS (1)')
      WRITE(6,220)
      READ(5,115) NPLCHK
      IF(NPLCHK.EQ.1) GO TO 233

C     ***** DISTRIBUTED LOADS **************************************

  214 WRITE(6,221)
  221 FORMAT(/,5X,'-------------------------------------------------------
     *-------',/,/,5X,'PLEASE PRESS (1) TO INPUT DISTRIBUTED LOADS...)',
     */,5X)
      READ(5,115) NDISPL
      IF(NDISPL.NE.1) GO TO 224
  237 WRITE(6,215)
      READ(5,216) NFLORP,NBAYP
      WRITE(6,222)
  222 FORMAT(/,5X,'ENTER FORCE FOR THE SELECTED SPAN (t/m)...')
      READ(5,219) DILOAD
      DIDISP=DILOAD
      LLOAD(NFLORP,NBAYP)=DILOAD
c      DDLOAD(NFLORP,NBAYP)=DILOAD
      JOINT=NFLORP+(NFLORP-1)*NBAY+NBAYP-1

  236 FORMAT(/,5X,'--------',F9.4)
      PDIST(3*JOINT)=PDIST(3*JOINT)-1*DIDISP*EIWDTH(NBAYP)*EIWDTH(NBAYP)
     */12
      PDIST(3*(JOINT+1))=PDIST(3*(JOINT+1))+DIDISP*EIWDTH(NBAYP)*EIWDTH(
     *NBAYP)/12
      PDIST(3*JOINT-1)=PDIST(3*JOINT-1)-1*DIDISP*EIWDTH(NBAYP)/2
      PDIST(3*(JOINT+1)-1)=PDIST(3*(JOINT+1)-1)-1*DIDISP*EIWDTH(NBAYP)/2

      PDKRS(NFLORP,NBAYP,3)=-1*DIDISP*EIWDTH(NBAYP)*EIWDTH(NBAYP)/12
      PDKRS(NFLORP,NBAYP,6)=DIDISP*EIWDTH(NBAYP)*EIWDTH(NBAYP)/12
      PDKRS(NFLORP,NBAYP,2)=-1*DIDISP*EIWDTH(NBAYP)/2
      PDKRS(NFLORP,NBAYP,5)=-1*DIDISP*EIWDTH(NBAYP)/2

  223 FORMAT(/,5X,'TO INPUT AN DISTRIBUTED LOAD AGAIN, PRESS (1)...')
      WRITE(6,223)
      READ(5,115) NPLCHK
      IF(NPLCHK.EQ.1) GO TO 237

C     *****  DEAD LOADS  *****************************************

  224 WRITE(6,225)
  225 FORMAT(/,5X,'-------------------------------------------------------
     *-------',/,/,5X,'PRESS (1) TO TAKE SELF WEIGHT INTO ACCOUNT...)',
     */,5X)
      READ(5,115) NSELFL
      IF(NSELFL.NE.1) GO TO 260
      DO 226 I=1,NFLR
      DO 227 J=1,NBAY
      DILOAD=AB(I,J)*7.86
      DIDISP=DILOAD
      JOINT=I+(I-1)*NBAY+J-1
      PSELF(3*JOINT)=PSELF(3*JOINT)+(-1)*DIDISP*EIWDTH(J)*EIWDTH(J)/12
      PSELF(3*(JOINT+1))=PSELF(3*(JOINT+1))+DIDISP*EIWDTH(J)*EIWDTH(J)/1
     *2
      PSELF(3*JOINT-1)=PSELF(3*JOINT-1)+(-1)*DIDISP*EIWDTH(J)/2
      PSELF(3*(JOINT+1)-1)=PSELF(3*(JOINT+1)-1)+(-1)*DIDISP*EIWDTH(J)/2

      PKIRIS(I,J,3)=(-1)*DIDISP*EIWDTH(J)*EIWDTH(J)/12
```

```
      PKIRIS(I,J,6)=DIDISP*EIWDTH(J)*EIWDTH(J)/12
      PKIRIS(I,J,2)=(-1)*DIDISP*EIWDTH(J)/2
      PKIRIS(I,J,5)=(-1)*DIDISP*EIWDTH(J)/2

  227 CONTINUE
  226 CONTINUE

C     *********DEAD LOAD CALCULATION FOR COLUMNS*********************

      DO 228 I=1,NFLR
      DO 229 J=1,(NBAY+1)
      DILOAD=AC(I,J)*7.86
      DIDISP=DILOAD
      JOINT=I+(I-1)*NBAY+J-1
      PSELFC(3*JOINT-1)=PSELFC(3*JOINT-1)+(-1)*DIDISP*EIHGHT(I)/2
      PSELFC(3*(JOINT-1-NBAY)-1)=PSELF(3*(JOINT-1-NBAY)-1)+(-1)*DIDISP*E
     *IHGHT(I)/2

  229 CONTINUE
  228 CONTINUE

C     ************************************************************
C     ***********      PARTICULAR EQ LOADS    *********************
C     ************************************************************

 260 WRITE(6,261)
  261 FORMAT(/,5X,'----------------------------------------------------
     *-------',/,/,5X,'PRESS (1) TO TAKE EARTHQUAKE LOADS INTO ACCOUNT..
     *.)',/,5X)
      READ(5,115) NEQ
      IF(NEQ.NE.1) GO TO 262

  262

  230 DO 235 I=1,NDEG
      PAPP(I)=PSELFC(I)+PSELF(I)+PDIST(I)+PPOINT(I)+PAPPEQ(I)
  235 CONTINUE

      WRITE(6,*) 'PAPP(22)'
      WRITE(6,*) PAPP(22)
      WRITE(6,*) 'PAPP(23)'
      WRITE(6,*) PAPP(23)
      WRITE(6,*) 'PAPP(24)'
      WRITE(6,*) PAPP(24)

      DO 369 I=1,NDEG
      PAPPSR(I)=PAPP(I)
  369 CONTINUE

      DO 340 I=1,NDEG
      PAPP2(I)=PAPPSR(I)
  340 CONTINUE
      DO 487 I=1,NDEG
  487 CONTINUE

C     ***************************************************************
C     *****************  MEMBER STIFFNESS MATRIX  ********************
C     ***************************************************************

C     STIFFNESS FOR COLUMNS

      DO 205 I=1,NFLR
      DO 206 J=1,(NBAY+1)
      DO 207 K=1,6
      DO 208 L=1,6
      AKC(I,J,K,L)=0
  208 CONTINUE
  207 CONTINUE
  206 CONTINUE
  205 CONTINUE
```

```
      DO 160 I=1,NFLR
      DO 161 J=1,(NBAY+1)
      OEA=E*AC(I,J)
      EI=E*AIC(I,J)
      RL=EIHGHT(I)
      AKC(I,J,1,1)=12*EI/(RL*RL*RL)
      AKC(I,J,1,2)=0
      AKC(I,J,1,3)=(-1)*6*EI/(RL*RL)
      AKC(I,J,1,4)=12*(-1)*EI/(RL*RL*RL)
      AKC(I,J,1,5)=0
      AKC(I,J,1,6)=(-1)*6*EI/(RL*RL)
      AKC(I,J,2,1)=0
      AKC(I,J,2,2)=OEA/RL
      AKC(I,J,2,3)=0
      AKC(I,J,2,4)=0
      AKC(I,J,2,5)=(-1)*OEA/RL
      AKC(I,J,2,6)=0
      AKC(I,J,3,1)=(-1)*6*EI/(RL*RL)
      AKC(I,J,3,2)=0
      AKC(I,J,3,3)=4*EI/RL
      AKC(I,J,3,4)=6*EI/(RL*RL)
      AKC(I,J,3,5)=0
      AKC(I,J,3,6)=2*EI/RL
      AKC(I,J,4,1)=12*(-1)*EI/(RL*RL*RL)
      AKC(I,J,4,2)=0
      AKC(I,J,4,3)=6*EI/(RL*RL)
      AKC(I,J,4,4)=12*EI/(RL*RL*RL)
      AKC(I,J,4,5)=0
      AKC(I,J,4,6)=6*EI/(RL*RL)
      AKC(I,J,5,1)=0
      AKC(I,J,5,2)=(-1)*OEA/RL
      AKC(I,J,5,3)=0
      AKC(I,J,5,4)=0
      AKC(I,J,5,5)=OEA/RL
      AKC(I,J,5,6)=0
      AKC(I,J,6,1)=(-1)*6*EI/(RL*RL)
      AKC(I,J,6,2)=0
      AKC(I,J,6,3)=2*EI/RL
      AKC(I,J,6,4)=6*EI/(RL*RL)
      AKC(I,J,6,5)=0
      AKC(I,J,6,6)=4*EI/RL
  161 CONTINUE
  160 CONTINUE

C     STIFFNESS FOR BEAMS

      DO 209 I=1,NFLR
      DO 210 J=1,NBAY
      DO 211 K=1,6
      DO 212 L=1,6
      AKB(I,J,K,L)=0
  212 CONTINUE
  211 CONTINUE
  210 CONTINUE
  209 CONTINUE
      DO 164 I=1,NFLR
      DO 165 J=1,NBAY
      OEA=E*AB(I,J)
      EI=E*AIB(I,J)
      RL=EIWDTH(J)
      AKB(I,J,1,1)=OEA/RL
      AKB(I,J,1,2)=0
      AKB(I,J,1,3)=0
      AKB(I,J,1,4)=OEA*(-1)/RL
      AKB(I,J,1,5)=0
      AKB(I,J,1,6)=0
      AKB(I,J,2,1)=0
      AKB(I,J,2,2)=12*EI/(RL*RL*RL)
      AKB(I,J,2,3)=6*EI/(RL*RL)
      AKB(I,J,2,4)=0
      AKB(I,J,2,5)=12*(-1)*EI/(RL*RL*RL)
```

```
      AKB(I,J,2,6)=6*EI/(RL*RL)
      AKB(I,J,3,1)=0
      AKB(I,J,3,2)=6*EI/(RL*RL)
      AKB(I,J,3,3)=4*EI/RL
      AKB(I,J,3,4)=0
      AKB(I,J,3,5)=(-1)*6*EI/(RL*RL)
      AKB(I,J,3,6)=2*EI/RL
      AKB(I,J,4,1)=(-1)*OEA/RL
      AKB(I,J,4,2)=0
      AKB(I,J,4,3)=0
      AKB(I,J,4,4)=OEA/RL
      AKB(I,J,4,5)=0
      AKB(I,J,4,6)=0
      AKB(I,J,5,1)=0
      AKB(I,J,5,2)=12*(-1)*EI/(RL*RL*RL)
      AKB(I,J,5,3)=(-1)*6*EI/(RL*RL)
      AKB(I,J,5,4)=0
      AKB(I,J,5,5)=12*EI/(RL*RL*RL)
      AKB(I,J,5,6)=6*(-1)*EI/(RL*RL)
      AKB(I,J,6,1)=0
      AKB(I,J,6,2)=6*EI/(RL*RL)
      AKB(I,J,6,3)=2*EI/RL
      AKB(I,J,6,4)=0
      AKB(I,J,6,5)=6*(-1)*EI/(RL*RL)
      AKB(I,J,6,6)=4*EI/RL
  165 CONTINUE
  164 CONTINUE

  320 FORMAT(/,5X,'TO INCLUDE P-DELTA EFFECT PRESS (1)...')
      WRITE(6,320)
      READ(5,103) NPCHK
      IF(NPCHK.EQ.1) THEN
      GO TO 321
      ELSE
      GO TO 326
      ENDIF

C     ****************************************************************
C     ****************************************************************
C     ******************  GEOMETRIC STIFFNESS   **********************
C     ****************************************************************
C     ****************************************************************

  322 NPCHK=NPCHK+1


      DO 323 I=1,NFLR
      DO 324 J=1,(NBAY+1)
      RL=EIHGHT(I)
      IF(NITNO.LT.2) THEN
      PL=PK(I,J,2)/RL
      ELSE
      PL=PKSR(I,J,2)/RL
      ENDIF
      GKC(I,J,1,1)=PL*6/5
      GKC(I,J,1,2)=0
      GKC(I,J,1,3)=(-1)*PL*RL/10
      GKC(I,J,1,4)=(-1)*PL*6/5
      GKC(I,J,1,5)=0
      GKC(I,J,1,6)=(-1)*PL*RL/10
      GKC(I,J,2,1)=0
      GKC(I,J,2,2)=0
      GKC(I,J,2,3)=0
      GKC(I,J,2,4)=0
      GKC(I,J,2,5)=0
      GKC(I,J,2,6)=0
      GKC(I,J,3,1)=(-1)*PL*RL/10
      GKC(I,J,3,2)=0
      GKC(I,J,3,3)=2*PL*RL*RL/15
      GKC(I,J,3,4)=PL*RL/10
      GKC(I,J,3,5)=0
```

```
      GKC(I,J,3,6)=(-1)*PL*RL*RL/30
      GKC(I,J,4,1)=(-1)*PL*6/5
      GKC(I,J,4,2)=0
      GKC(I,J,4,3)=PL*RL/10
      GKC(I,J,4,4)=PL*6/5
      GKC(I,J,4,5)=0
      GKC(I,J,4,6)=PL*RL/10
      GKC(I,J,5,1)=0
      GKC(I,J,5,2)=0
      GKC(I,J,5,3)=0
      GKC(I,J,5,4)=0
      GKC(I,J,5,5)=0
      GKC(I,J,5,6)=0
      GKC(I,J,6,1)=(-1)*PL*RL/10
      GKC(I,J,6,2)=0
      GKC(I,J,6,3)=(-1)*PL*RL*RL/30
      GKC(I,J,6,4)=PL*RL/10
      GKC(I,J,6,5)=0
      GKC(I,J,6,6)=2*RL*RL*PL/15
  324 CONTINUE
  323 CONTINUE


      DO 315 I=1,NFLR
      DO 316 J=1,NBAY+1
      DO 317 K=1,6
      DO 318 L=1,6
      AKC(I,J,K,L)=AKC(I,J,K,L)-GKC(I,J,K,L)
  318 CONTINUE
  317 CONTINUE
  316 CONTINUE
  315 CONTINUE

      GO TO 326
  321 DENEME=0
  326 DENEME=1
  555
      IF(NPCHK.NE.1) GO TO 381

C     *************** SEMI-RIGID CONNECTIONS   ************************

  360 FORMAT(/,5X,'IS THERE ANY SEMI-RIGID CONNECTION IN THE FRAME?..',/
     *,15X,'IF YES, PRESS(1)...')
      WRITE(6,360)
      READ(5,103) NSRCHK
      IF(NSRCHK.NE.1) GO TO 361

  362
      READ(5,216) NFLORP,NBAYP


  350 FORMAT(/,10X,'(1) SINGLE WEB ANGLE',/,10X,'(2) DOUBL WEB ANGLE',/,
     *10X,'(3) TOP & SEAT ANGLE  WITH DOUBLE WEB ANGLE',/,10X,'(4) TOP &
     * SEAT ANGLE WITHOUT DOUBLE WEB ANGLE',/,10X,'(5) END PLATE WITHOUT
     * STIFFENER',/,10X,'(6) END PLATE WITH STIFFENERS',/,10X,'(7) T-STU
     *B',/,10X,'(8) HEADER PLATE',/,/,5X,'PLEASE ENTER CONNECTION TYPE!.
     *.')
      WRITE(6,350)
      WRITE(6,*) NFLORP,NBAYP
      READ(5,103) NCTYP


      IF(NCTYP.EQ.1) THEN
  351 FORMAT(/,5X,'ENTER da,ta & g VALUES RESPECTIVELY FOR SINGLE WEB AN
     *GLE CONNECTION...')
      WRITE(6,351)
  352 FORMAT(F6.5)
      READ(5,352) DA1,TA1,G1
      C1(NFLORP,NBAYP)=0.00428
      C2(NFLORP,NBAYP)=0.00000000145**(1./3.)
      C3(NFLORP,NBAYP)=0.000000000000000151**(0.2)
```

```
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(DA1**(-2.4))*(TA1**(-1.81))*(G1**(0.15))
      ELSE
      ENDIF

      IF(NCTYP.EQ.2) THEN
  353 FORMAT(/,5X,'ENTER da,ta & g VALUES RESPECTIVELY FOR DOUBLE WEB AN
     *GLE CONNECTION...')
      WRITE(6,353)
      READ(5,352) DA2
      READ(5,352) TA2
      READ(5,352) G2
      C1(NFLORP,NBAYP)=0.000366
      C2(NFLORP,NBAYP)=0.00000115**(1./3.)
      C3(NFLORP,NBAYP)=0.0000000457**(0.2)
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(DA2**(-2.4))*(TA2**(-1.81))*(G2**(0.15))
      ELSE
      ENDIF

      IF(NCTYP.EQ.3) THEN
  354 FORMAT(/,5X,'ENTER d,t,tc,la & g VALUES RESPECTIVELY FOR TOP & SEA
     *T ANGLE WITH DOUBLE WEB ANGLE CONNECTION...')
      WRITE(6,354)
      READ(5,352) D3
      READ(5,352) T3
      READ(5,352) TC3
      READ(5,352) ALA3
      READ(5,352) G3
      READ(5,352) UMC(NFLORP,NBAYP)
      C1(NFLORP,NBAYP)=0.0000223
      C2(NFLORP,NBAYP)=(0.0000000185)**(1./3.)
      C3(NFLORP,NBAYP)=(0.00000000000319)**(0.2)
      AKAPA(NFLORP,NBAYP)=(D3**(-1.287))*(T3**(-1.128))*(TC3**(-0.415))*
     *(ALA3**(-0.694))*(G3**(1.35))
C       WRITE(6,1001) AKAPA(NFLORP,NBAYP)
      ELSE
      ENDIF
      IF(NCTYP.EQ.4) THEN
  355 FORMAT(/,5X,'ENTER d,t,la & db VALUES RESPECTIVELY FOR TOP & SEAT
     *ANGLE WITHOUT DOUBLE WEB ANGLE CONNECTION...')
      WRITE(6,355)
      READ(5,352) D4
      READ(5,352) T4
      READ(5,352) ALA4
      READ(5,352) DB4
      C1(NFLORP,NBAYP)=0.000846
      C2(NFLORP,NBAYP)=0.000101**(1./3.)
      C3(NFLORP,NBAYP)=0.0000000124**(0.2)
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(D4**(-1.5))*(T4**(-0.5))*(ALA4**(-0.7))*(DB4*
     ***(-1.5))
      ELSE
      ENDIF
      IF(NCTYP.EQ.5) THEN
  356 FORMAT(/,5X,'ENTER dg,tp & db VALUES RESPECTIVELY FOR END PLATE WI
     *THOUT COLUMN STIFFENER CONNECTION...')
      WRITE(6,356)
      READ(5,352) DG5,TP5,DB5
      C1(NFLORP,NBAYP)=0.00183
      C2(NFLORP,NBAYP)=0.000104**(1./3.)
      C3(NFLORP,NBAYP)=0.00000638**(0.2)
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(DG5**(-2.4))*(TP5**(-0.4))*(DB5**(-1.5))
      ELSE
      ENDIF

      IF(NCTYP.EQ.6) THEN
  357 FORMAT(/,5X,'ENTER dg & tp VALUES RESPECTIVELY FOR END PLATE WITH
     *COLUMN STIFFENER CONNECTION...')
      WRITE(6,357)
```

```
      READ(5,352) DG6
      READ(5,352) TP6
      C1(NFLORP,NBAYP)=0.00179
      C2(NFLORP,NBAYP)=0.000176**(1./3.)
      C3(NFLORP,NBAYP)=0.00000638**(0.2)
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(DG6**(-2.4))*(TP6**(-0.6))
      ELSE
      ENDIF
      IF(NCTYP.EQ.7) THEN

  358 FORMAT(/,5X,'ENTER d,t,lt & db VALUES RESPECTIVELY FOR T-STUB CONN
     *ECTION...')
      WRITE(6,358)
      READ(5,352) D7,T7,ALT7,DB7
      C1(NFLORP,NBAYP)=0.000204
      C2(NFLORP,NBAYP)=0.000210**(1./3.)
      C3(NFLORP,NBAYP)=-0.0000000076**(0.2)
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(D7**(-1.5))*(T7**(-0.5))*(ALT7**(-0.7))*(DB7*
     **(-1.1))
      ELSE
      ENDIF
      IF(NCTYP.EQ.8) THEN
  359 FORMAT(/,5X,'ENTER dp,tp,tw & g VALUES RESPECTIVELY FOR END PLATE
     *WITHOUT COLUMN STIFFENER CONNECTION...')
      WRITE(6,359)
      READ(5,352) DP8,TP8,TW8,G8
      C1(NFLORP,NBAYP)=0.0000510
      C2(NFLORP,NBAYP)=0.00000000062**(1./3.)
      C3(NFLORP,NBAYP)=0.00000000000024**(0.2)
      READ(5,352) UMC(NFLORP,NBAYP)
      AKAPA(NFLORP,NBAYP)=(DP8**(-2.3))*(TP8**(-1.6))*(TW8**(-0.5))*(G8*
     **(1.6))
      ELSE
      ENDIF

  375 FORMAT(/,5X,'TO ADD A SEMI-RIGID CONNECTION AGAIN PRESS (1)...')
      WRITE(6,375)
      READ(5,103) NSRADD
      IF(NSRADD.EQ.1) GO TO 362

  367 FORMAT(/,5X,'PLEASE ENTER # OF ITERATIONS FOR LOAD INCREMENT...')
      WRITE(6,367)
  368 FORMAT(I4)
      READ(5,368) NITER

C     ********************************REN****************************

 1367 FORMAT(/,5X,'PLEASE ENTER LOAD INCREMENT FOR EACH STEP...')
      WRITE(6,1367)
 1368 FORMAT(F9.3)
      READ(5,) ANLOAD

C     ********************************REN****************************
  381
      IF(NPCHK.EQ.0) GO TO 361
      IF(NPCHK.EQ.1) GO TO 423
      IF(NPCHK.EQ.2) GO TO 361

  386
      KONT=0
      DO 413 I=1,6
      DO 414 J=1,6
      KONTRO(I,J)=0
  414 CONTINUE
  413 CONTINUE

      DO 390 I=1,NFLR
      DO 391 J=1,NBAY
      IF(AKAPA(I,J).EQ.0) GO TO 402
```

```
      ROT3(I,J)=ROTAT3(I,J)
      ROT6(I,J)=ROTAT6(I,J)
      AMKIP3(I,J)=(PB(I,J,3)+PBSR(I,J,3))*86.8043
      AMKIP6(I,J)=(PB(I,J,6)+PBSR(I,J,6))*86.8043
      ROTAT3(I,J)=C1(I,J)*AKAPA(I,J)*AMKIP3(I,J)+(C2(I,J)*AKAPA(I,J)*AMK
     *IP3(I,J))**3.+(C3(I,J)*AKAPA(I,J)*AMKIP3(I,J))**5.
      ROTAT6(I,J)=C1(I,J)*AKAPA(I,J)*AMKIP6(I,J)+(C2(I,J)*AKAPA(I,J)*AMK
     *IP6(I,J))**3.+(C3(I,J)*AKAPA(I,J)*AMKIP6(I,J))**5.
      DELTA3(I,J)=(ROTAT3(I,J)-ROT3(I,J))/ROTAT3(I,J)
      DELTA6(I,J)=(ROTAT6(I,J)-ROT6(I,J))/ROTAT6(I,J)
 1236 FORMAT(2X,I3,2X,F13.7,1X,F13.9,1X,F13.9,1X,F13.9,1X,F13.9)
 1000 FORMAT(25X,'KONTRO',F18.8)
 1001 FORMAT(10X,'---------------  ',F20.15)
      IF(DELTA3(I,J).LT.(-0.0001).OR.DELTA3(I,J).GT.0.0001.OR.DELTA6(I,J
     *).LT.(-0.0001).OR.DELTA6(I,J).GT.0.0001) THEN
      KONTRO(I,J)=KONTRO(I,J)+1
      ELSE
      ENDIF

 1238 FORMAT(2X,I3,2X,F10.3,1X,F10.3,1X,F10.3,1X,F10.3,1X,F10.3,1X,F10.3
     *)
      KONT=KONT+KONTRO(I,J)
      SRSTF3(I,J)=PB(I,J,3)/(ROTAT3(I,J)-ROTZ3(I,J))
      SRSTF6(I,J)=PB(I,J,6)/(ROTAT6(I,J)-ROTZ6(I,J))

      OEA=E*AB(I,J)
      EI=E*AIB(I,J)
      RL=EIWDTH(J)
      GAMA1=RL/(RL+3*EI/SRSTF3(I,J))
      GAMA2=RL/(RL+3*EI/SRSTF6(I,J))
      ASR=GAMA1+GAMA2+GAMA1*GAMA2
      BSR=GAMA1*(2+GAMA2)
      CSR=3*GAMA1
      DSR=3*GAMA2
      FSR=3*GAMA1*GAMA2
      GSR=GAMA2*(2+GAMA1)
      AHSR=4-GAMA1*GAMA2
      AKB(I,J,1,1)=OEA/RL
      AKB(I,J,1,2)=0
      AKB(I,J,1,3)=0
      AKB(I,J,1,4)=OEA*(-1)/RL
      AKB(I,J,1,5)=0
      AKB(I,J,1,6)=0
      AKB(I,J,2,1)=0
      AKB(I,J,2,2)=12*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,2,3)=6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,2,4)=0
      AKB(I,J,2,5)=12*(-1)*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,2,6)=6*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,3,1)=0
      AKB(I,J,3,2)=6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,3,3)=4*EI*CSR/(RL*AHSR)
      AKB(I,J,3,4)=0
      AKB(I,J,3,5)=(-1)*6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,3,6)=2*EI*FSR/(RL*AHSR)
      AKB(I,J,4,1)=(-1)*OEA/RL
      AKB(I,J,4,2)=0
      AKB(I,J,4,3)=0
      AKB(I,J,4,4)=OEA/RL
      AKB(I,J,4,5)=0
      AKB(I,J,4,6)=0
      AKB(I,J,5,1)=0
      AKB(I,J,5,2)=12*(-1)*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,5,3)=(-1)*6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,5,4)=0
      AKB(I,J,5,5)=12*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,5,6)=6*(-1)*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,6,1)=0
      AKB(I,J,6,2)=6*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,6,3)=2*EI*FSR/(RL*AHSR)
      AKB(I,J,6,4)=0
```

```
      AKB(I,J,6,5)=6*(-1)*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,6,6)=4*EI*DSR/(AHSR*RL)

  402
  391 CONTINUE
  390 CONTINUE

 9237 FORMAT(2X,I3,2X,F10.3,1X,F10.5,1X,F10.5,1X,F10.3,1X,F10.5,1X,F10.5
     *)

      IF(NPCHK.NE.1) GO TO 559
  423
      NITNO=1
C      WRITE(10,*) NITNO
  559
      DO 447 I=1,NDEG
      PAPP(I)=PAPPSR(I)/NITER
      PAPP2(I)=PAPP(I)
  447 CONTINUE

      DO 687 I=1,NFLR
      DO 686 J=1,NBAY
      DO 685 K=1,6
      PKIRIS(I,J,K)=PKIRIS(I,J,K)/NITER
      PDKRS(I,J,K)=PDKRS(I,J,K)/NITER
  685 CONTINUE
  686 CONTINUE
  687 CONTINUE

      IF(NPCHK.LT.3) GO TO 361
      IF(NSRCHK.EQ.1.AND.NITNO.LE.NITER.AND.KONT.EQ.0) THEN
      NNOO=NNOO+1
      NITNO=NITNO+1

      WRITE(15,*) SRSTF3(4,2)


      DO 392 I=1,NFLR
      DO 393 J=1,NBAY
      IF(AKAPA(I,J).EQ.0) GO TO 415
      PBSR(I,J,1)=PBSR(I,J,1)+PB(I,J,1)
      PBSR(I,J,2)=PBSR(I,J,2)+PB(I,J,2)
      PBSR(I,J,3)=PBSR(I,J,3)+PB(I,J,3)
      PBSR(I,J,4)=PBSR(I,J,4)+PB(I,J,4)
      PBSR(I,J,5)=PBSR(I,J,5)+PB(I,J,5)
      PBSR(I,J,6)=PBSR(I,J,6)+PB(I,J,6)
      ROTZ3(I,J)=ROTAT3(I,J)
      ROTZ6(I,J)=ROTAT6(I,J)
  415
  393 CONTINUE
  392 CONTINUE

      DO 417 I=1,NFLR
      DO 418 J=1,NBAY+1
      DO 419 K=1,6
      PKSR(I,J,K)=PKSR(I,J,K)+PK(I,J,K)
  419 CONTINUE
  418 CONTINUE
  417 CONTINUE

      DO 420 I=1,NDEG
      DSRGD(I)=DSRGD(I)+D(I)
  420 CONTINUE

      DO 439 I=1,NDEG
      PAPP(I)=PAPPSR(I)/NITER
  439 CONTINUE
      ELSE
      ENDIF
      IF(NITNO.EQ.(NITER+1)) GO TO 421
```

```
      TTJ=NFLR+(NFLR-1)*NBAY
      TTOP=DSRGD(3*TTJ-2)
      TJ=NFLR+(NFLR-1)*NBAY

  361

C    ***************************************************************
C    ***************************************************************
C    ***************************************************************
C    ********************   GLOBAL STIFFNESS   *********************
C    ***************************************************************
C    ***************************************************************
C    ***************************************************************

      DO 202 J=1,NDEG
      DO 203 I=1,NDEG
      ADEG(I,J)=0.0
  203 CONTINUE
  202 CONTINUE

C    ********************DIAGONAL ELEMENTS*************************

      NFLRC=1
      NBAYC=0
      DO 172 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K)=AKC(I,J,4,4)+AKC((I+1),J,1,1)+AKB(I,J,1,1)+AKB(I,(J-1),4
     *,4)
  172 CONTINUE

      NFLRC=1
      NBAYC=0
      DO 175 K=3,NDEG,3
      NBAYC=NBAYC+1
      IF((K-3)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
```

```
      ENDIF
      IF((K-3)/3.EQ.(4*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K)=AKC(I,J,6,6)+AKC((I+1),J,3,3)+AKB(I,J,3,3)+AKB(I,(J-1),6
     *,6)
  175 CONTINUE

      NFLRC=1
      NBAYC=0
      DO 178 K=2,(NDEG-1),3
      NBAYC=NBAYC+1
      IF((K-2)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K)=AKC(I,J,5,5)+AKC((I+1),J,2,2)+AKB(I,J,2,2)+AKB(I,(J-1),5
     *,5)
  178 CONTINUE

C     ***************** SAME JOINT **************

      NFLRC=1
      NBAYC=0
      DO 192 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K+1)=AKC(I,J,4,5)+AKC((I+1),J,1,2)+AKB(I,(J-1),4,5)+AKB(I,J
     *,1,2)
```

```
      ADEG(K+1,K)=ADEG(K,K+1)
      ADEG(K,K+2)=AKC(I,J,4,6)+AKC((I+1),J,1,3)+AKB(I,(J-1),4,6)+AKB(I,J
     *,1,3)
      ADEG(K+2,K)=ADEG(K,K+2)
      ADEG(K+1,K+2)=AKC(I,J,5,6)+AKC((I+1),J,2,3)+AKB(I,(J-1),5,6)+AKB(I
     *,J,2,3)
      ADEG(K+2,K+1)=ADEG(K+1,K+2)

  192 CONTINUE

C     ***** ON THE RIGHT *******

      NFLRC=1
      NBAYC=0
      DO 195 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K+3)=AKB(I,J,1,4)
      ADEG(K+3,K)=ADEG(K,K+3)
      ADEG(K+1,K+3)=AKB(I,J,2,4)
      ADEG(K+3,K+1)=ADEG(K+1,K+3)
      ADEG(K+2,K+3)=AKB(I,J,3,4)
      ADEG(K+3,K+2)=ADEG(K+2,K+3)


      ADEG(K,K+4)=AKB(I,J,1,5)
      ADEG(K+4,K)=ADEG(K,K+4)
      ADEG(K+1,K+4)=AKB(I,J,2,5)
      ADEG(K+4,K+1)=ADEG(K+1,K+4)
      ADEG(K+2,K+4)=AKB(I,J,3,5)
      ADEG(K+4,K+2)=ADEG(K+2,K+4)


      ADEG(K,K+5)=AKB(I,J,1,6)
      ADEG(K+5,K)=ADEG(K,K+5)
      ADEG(K+1,K+5)=AKB(I,J,2,6)
      ADEG(K+5,K+1)=ADEG(K+1,K+5)
      ADEG(K+2,K+5)=AKB(I,J,3,6)
      ADEG(K+5,K+2)=ADEG(K+2,K+5)

  195 CONTINUE
C     *************** COLUMN TOP AND BOTTOM ****************************
      NFLRC=2
      NBAYC=0
      DO 194 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
```

```
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC

      ADEG(K,(K+3*(NBAY+1)))=AKC(I,J,1,4)
      ADEG((K+3*(NBAY+1)),K)=ADEG(K,(K+3*(NBAY+1)))
      ADEG(K,(K+1+3*(NBAY+1)))=AKC(I,J,1,5)
      ADEG((K+1+3*(NBAY+1)),K)=ADEG(K,(K+1+3*(NBAY+1)))
      ADEG(K,(K+2+3*(NBAY+1)))=AKC(I,J,1,6)
      ADEG((K+2+3*(NBAY+1)),K)=ADEG(K,(K+2+3*(NBAY+1)))

      ADEG(K+1,(K+3*(NBAY+1)))=AKC(I,J,2,4)
      ADEG((K+3*(NBAY+1)),K+1)=ADEG(K+1,(K+3*(NBAY+1)))
      ADEG(K+1,(K+1+3*(NBAY+1)))=AKC(I,J,2,5)
      ADEG((K+1+3*(NBAY+1)),K+1)=ADEG(K+1,(K+1+3*(NBAY+1)))
      ADEG(K+1,(K+2+3*(NBAY+1)))=AKC(I,J,2,6)
      ADEG((K+2+3*(NBAY+1)),K+1)=ADEG(K+1,(K+2+3*(NBAY+1)))

      ADEG(K+2,(K+3*(NBAY+1)))=AKC(I,J,3,4)
      ADEG((K+3*(NBAY+1)),K+2)=ADEG(K+2,(K+3*(NBAY+1)))
      ADEG(K+2,(K+1+3*(NBAY+1)))=AKC(I,J,3,5)
      ADEG((K+1+3*(NBAY+1)),K+2)=ADEG(K+2,(K+1+3*(NBAY+1)))
      ADEG(K+2,(K+2+3*(NBAY+1)))=AKC(I,J,3,6)
      ADEG((K+2+3*(NBAY+1)),K+2)=ADEG(K+2,(K+2+3*(NBAY+1)))

  194 CONTINUE



C     ****************** FOR HINGE BASE **************************

      IF(NPCHK.NE.1) THEN
      NDEG4=NDEG5
      NDEG3=NDEG5
      ELSE
      ENDIF

      DO 306 J=1,NBAY+1
      IF(IBASE(J).EQ.1) THEN
      NDEG3=NDEG3+1
      ADEG(NDEG3,NDEG3)=AKC(1,J,3,3)
      ADEG(3*J-2,NDEG3)=AKC(1,J,3,4)
      ADEG(NDEG3,3*J-2)=ADEG(3*J-2,NDEG3)
      ADEG(3*J-1,NDEG3)=AKC(1,J,3,5)
      ADEG(NDEG3,3*J-1)=ADEG(3*J-1,NDEG3)
      ADEG(3*J,NDEG3)=AKC(1,J,3,6)
      ADEG(NDEG3,3*J)=ADEG(3*J,NDEG3)
      ENDIF
  306 CONTINUE


C     **************************************************************
C                           END OF GLOBAL STIFFNESS
C     **************************************************************
```

```
      IF(NPCHK.NE.1) THEN
      DO 341 I=1,NDEG
      PAPP(I)=PAPP2(I)
  341 CONTINUE
      ELSE
      ENDIF

C     ***************** GAUSS ELIMINATION ******************************

      DO 253 I=1,NDEG-1
      DO 265 K=NDEG,1+I,-1
      AKSAYI=ADEG(K,I)/ADEG(I,I)*(-1)
      DO 255 J=I,NDEG
      ADEG(K,J)=ADEG(K,J)+ADEG(I,J)*AKSAYI
  255 CONTINUE
      PAPP(K)=PAPP(K)+PAPP(I)*AKSAYI
  265 CONTINUE
  163 FORMAT(5X,F14.4)
  253 CONTINUE

C     ********************* DEPLASMANLARIN BULUNMASI******************

      D(NDEG)=PAPP(NDEG)/ADEG(NDEG,NDEG)

      DO 300 I=NDEG-1,1,-1
      TOPLAM(I)=0
      DO 303 J=NDEG,I+1,-1
      TOPLAM(I)=TOPLAM(I)+ADEG(I,J)*D(J)

  303 CONTINUE
      D(I)=(PAPP(I)-TOPLAM(I))/ADEG(I,I)
  300 CONTINUE


C     ******************* MEMBER FORCES ****************************

      DO 307 I=1,NFLR
      DO 308 J=1,NBAY+1
      IF(I.EQ.1.AND.IBASE(J).EQ.1) NDEG4=NDEG4+1
      DO 309 K=1,6
      JOINT=I+(I-1)*NBAY+J-1
      L=(JOINT-NBAY-1)*3-2
      IF(I.EQ.1.AND.IBASE(J).EQ.1) THEN
           PK(I,J,K)=AKC(I,J,K,3)*D(NDEG4)+AKC(I,J,K,4)*D(JOINT*3-2)+AKC
     *(I,J,K,5)*D(JOINT*3-1)+AKC(I,J,K,6)*D(JOINT*3)

      ELSE
           PK(I,J,K)=AKC(I,J,K,1)*D(L)+AKC(I,J,K,2)*D(L+1)+AKC(I,J,K,3)*
     *D(L+2)+AKC(I,J,K,4)*D(JOINT*3-2)+AKC(I,J,K,5)*D(JOINT*3-1)+AKC(I,J
     *,K,6)*D(JOINT*3)
      ENDIF

  309 CONTINUE
  308 CONTINUE
  307 CONTINUE

      IF(NPCHK.NE.1) THEN
      DO 333 I=1,NFLR
      DO 334 J=1,NBAY+1
      DO 335 K=1,6
      DO 336 L=1,6
      AKC(I,J,K,L)=AKC(I,J,K,L)+GKC(I,J,K,L)
  336 CONTINUE
  335 CONTINUE
  334 CONTINUE
  333 CONTINUE
      ELSE
      ENDIF
```

```
      IF(NPCHK.EQ.1) GO TO 322

C     ********************* BEAM FORCES ***************************

        DO 310 I=1,NFLR
      DO 311 J=1,NBAY
      DO 312 K=1,6
      JOINT=I+(I-1)*NBAY+J-1
      L=JOINT*3-2
        PB(I,J,K)=(AKB(I,J,K,1)*D(L)+AKB(I,J,K,2)*D(L+1)+AKB(I,J,K,3)*D(
     *L+2)+AKB(I,J,K,4)*D((JOINT+1)*3-2)+AKB(I,J,K,5)*D((JOINT+1)*3-1)+A
     *KB(I,J,K,6)*D((JOINT+1)*3))-PKIRIS(I,J,K)-PDKRS(I,J,K)

  312 CONTINUE
  311 CONTINUE
  310 CONTINUE


      IF(NSRCHK.NE.1) GO TO 421
      IF(NITNO.LT.(NITER+1)) GO TO 322
  421

 5360 FORMAT(/,5X,'EQ FORCES WILL NOT BEEN CONSIDERED...',/,15X,'IF YES,
     *PRESS(1)...')
      WRITE(6,5360)
      READ(5,103) NDEVAM
      IF(NDEVAM.NE.1) GO TO 3667


      go to 9328
C     *********************************************************
c     **************PERFORMANCE PART BEGINS HERE'**************
C     *********************************************************
C     *********************************************************

3667
      DO 1271 I=1,100
      PAPP(I)=0.0
      PSELF(I)=0.0
      PSELFC(I)=0.0
      PAPPEQ(I)=0.0
 1271 CONTINUE

C     ************************************************************
C     ***********       EQ LOADS             **********************
C     ************************************************************

 1260 WRITE(6,1261)
 1261 FORMAT(/,5X,'-------------------------------------------------------
     *-------',/,/,5X,'PRESS (1) TO TAKE EARTHQUAKE LOADS INTO ACCOUNT..
     *.)',/,5X)
      READ(5,115) NEQ
      IF(NEQ.NE.1) GO TO 1262

 1181 FORMAT (I1)

      WRITE(6,1239)
 1239 FORMAT(/,5X,'********** EARTHQUAKE LOADS **********',/,/,5X,' PLEA
     *SE ENTER EARTHQUAKE ZONE...')
 1242 THGHT(0)=0
      DO 1252 I=1,NFLR
      THGHT(I)=THGHT(I-1)+EIHGHT(I)
 1252 CONTINUE

      DO 1251 I=1,NFLR
      EQFARA=0
      DO 1245 J=1,NBAY
      EQWB(I)=EQFARA+7.86*AB(I,J)*EIWDTH(J)+LLOAD(I,J)*0.3*EIWDTH(J)
      EQFARA=EQWB(I)
 1245 CONTINUE
 1251 CONTINUE
```

```
C     *********COLUMN DEAD LOADS~*********************

      DO 1246 I=1,NFLR
      DO 1247 J=1,(NBAY+1)
      EQWC(I)=EQWC(I)+AC(I,J)*EIHGHT(I)*7.86
 1247 CONTINUE
 1246 CONTINUE

      DO 1248 I=1,NFLR
      EQW(I)=EQWC(I)+EQWB(I)
 1248 CONTINUE

      ALT=0
      DO 1657 I=1,NFLR
      ALT=ALT+EQW(I)*THGHT(I)
 1657 CONTINUE

      DO 1658 I=1,NFLR
      EQKAT(I)=(EQW(I)*THGHT(I))/ALT
 1658 CONTINUE
      DO 1258 I=1,NFLR
      DO 1257 J=1,NBAY+1
      JOINT=I+(I-1)*NBAY+J-1
      PAPPEQ(3*JOINT-2)=EQKAT(I)/(NBAY+1)
 1257 CONTINUE
 1258 CONTINUE

 1262

 1230 DO 1235 I=1,NDEG
      PAPP(I)=PAPPEQ(I)*100./1000.
 1235 CONTINUE

      DO 1369 I=1,NDEG
      PAPPSR(I)=PAPPEQ(I)
 1369 CONTINUE

      DO 1340 I=1,NDEG
      PAPP2(I)=PAPP(I)
 1340 CONTINUE
      DO 1487 I=1,NDEG
 1487 CONTINUE

C    ****************************************************************
C    ****************************************************************
C    ******************  GEOMETRIC STIFFNESS   **********************
C    ****************************************************************
C    ****************************************************************
      SECOND=1

 1322 NPCHK=NPCHK+1

c      TTJ=NFLR+(NFLR-1)*NBAY
c      TTOP=DSRGD(3*TTJ-2)

      DO 1323 I=1,NFLR
      DO 1324 J=1,(NBAY+1)
      RL=EIHGHT(I)
      IF(NITNO.LT.2) THEN
      PL=PK(I,J,2)/RL
      ELSE
      PL=PKSR(I,J,2)/RL
      ENDIF
      GKC(I,J,1,1)=PL*6/5
      GKC(I,J,1,2)=0
      GKC(I,J,1,3)=(-1)*PL*RL/10
      GKC(I,J,1,4)=(-1)*PL*6/5
      GKC(I,J,1,5)=0
      GKC(I,J,1,6)=(-1)*PL*RL/10
      GKC(I,J,2,1)=0
```

```
      GKC(I,J,2,2)=0
      GKC(I,J,2,3)=0
      GKC(I,J,2,4)=0
      GKC(I,J,2,5)=0
      GKC(I,J,2,6)=0
      GKC(I,J,3,1)=(-1)*PL*RL/10
      GKC(I,J,3,2)=0
      GKC(I,J,3,3)=2*PL*RL*RL/15
      GKC(I,J,3,4)=PL*RL/10
      GKC(I,J,3,5)=0
      GKC(I,J,3,6)=(-1)*PL*RL*RL/30
      GKC(I,J,4,1)=(-1)*PL*6/5
      GKC(I,J,4,2)=0
      GKC(I,J,4,3)=PL*RL/10
      GKC(I,J,4,4)=PL*6/5
      GKC(I,J,4,5)=0
      GKC(I,J,4,6)=PL*RL/10
      GKC(I,J,5,1)=0
      GKC(I,J,5,2)=0
      GKC(I,J,5,3)=0
      GKC(I,J,5,4)=0
      GKC(I,J,5,5)=0
      GKC(I,J,5,6)=0
      GKC(I,J,6,1)=(-1)*PL*RL/10
      GKC(I,J,6,2)=0
      GKC(I,J,6,3)=(-1)*PL*RL*RL/30
      GKC(I,J,6,4)=PL*RL/10
      GKC(I,J,6,5)=0
      GKC(I,J,6,6)=2*RL*RL*PL/15
 1324 CONTINUE
 1323 CONTINUE


      DO 1315 I=1,NFLR
      DO 1316 J=1,NBAY+1
      DO 1317 K=1,6
      DO 1318 L=1,6
      AKC(I,J,K,L)=AKC(I,J,K,L)-GKC(I,J,K,L)
 1318 CONTINUE
 1317 CONTINUE
 1316 CONTINUE
 1315 CONTINUE

      GO TO 1326

      DENEME=0
 1326 DENEME=1
 1555

 1386
      KONT=0
      DO 1413 I=1,6
      DO 1414 J=1,6
      KONTRO(I,J)=0
 1414 CONTINUE
 1413 CONTINUE

      DO 1390 I=1,NFLR
      DO 1391 J=1,NBAY
      IF(AKAPA(I,J).EQ.0) GO TO 1402
      ROT3(I,J)=ROTAT3(I,J)
      ROT6(I,J)=ROTAT6(I,J)
      AMKIP3(I,J)=(PB(I,J,3)+PBSR(I,J,3))*86.8043
      AMKIP6(I,J)=(PB(I,J,6)+PBSR(I,J,6))*86.8043
      ROTAT3(I,J)=C1(I,J)*AKAPA(I,J)*AMKIP3(I,J)+(C2(I,J)*AKAPA(I,J)*AMK
     *IP3(I,J))**3.+(C3(I,J)*AKAPA(I,J)*AMKIP3(I,J))**5.
      ROTAT6(I,J)=C1(I,J)*AKAPA(I,J)*AMKIP6(I,J)+(C2(I,J)*AKAPA(I,J)*AMK
     *IP6(I,J))**3.+(C3(I,J)*AKAPA(I,J)*AMKIP6(I,J))**5.
      DELTA3(I,J)=(ROTAT3(I,J)-ROT3(I,J))/ROTAT3(I,J)
      DELTA6(I,J)=(ROTAT6(I,J)-ROT6(I,J))/ROTAT6(I,J)
11235 FORMAT(2X,I3,2X,F13.11,1X,F13.11,1X,F13.11,1X,F13.11,1X,F13.9)
```

```
11236 FORMAT(2X,I3,2X,F13.7,1X,F13.9,1X,F13.9,1X,F13.9,1X,F13.9)
11000 FORMAT(25X,'KONTRO',F18.8)
11001 FORMAT(10X,'---------------  ',F20.15)
      IF(DELTA3(I,J).LT.(-0.0001).OR.DELTA3(I,J).GT.0.0001.OR.DELTA6(I,J
     *).LT.(-0.0001).OR.DELTA6(I,J).GT.0.0001) THEN
c      IF(DELTA3(I,J).LT.(-0.001).OR.DELTA3(I,J).GT.0.001.OR.DELTA6(I,J).
c     *LT.(-0.001).OR.DELTA6(I,J).GT.0.001) THEN
      KONTRO(I,J)=KONTRO(I,J)+1
      ELSE
      ENDIF

11238 FORMAT(2X,I3,2X,F10.3,1X,F10.3,1X,F10.3,1X,F10.3,1X,F10.3,1X,F10.3
     *)
      KONT=KONT+KONTRO(I,J)

      SRSTF3(I,J)=PB(I,J,3)/(ROTAT3(I,J)-ROTZ3(I,J))
      SRSTF6(I,J)=PB(I,J,6)/(ROTAT6(I,J)-ROTZ6(I,J))
      IF(PBSR(I,J,3).GT.(UMC(I,J)).OR.PBSR(I,J,3).LT.-(UMC(I,J))) THEN
      SRSTF3(I,J)=10
      SRS3(I,J)=SRS3(I,J)+1
      IF(SRS3(I,J).EQ.1) THEN
      WRITE(6,*) 'LEFT ',I,J,SECOND
      ELSE
      ENDIF
      ENDIF
      IF(PBSR(I,J,6).GT.(UMC(I,J)).OR.PBSR(I,J,6).LT.-(UMC(I,J))) THEN
      SRSTF6(I,J)=10
      SRS6(I,J)=SRS6(I,J)+1
      IF(SRS6(I,J).EQ.1) THEN
      WRITE(6,*) 'RIGHT',I,J,SECOND
      ELSE
      ENDIF
      ENDIF

      OEA=E*AB(I,J)
      EI=E*AIB(I,J)
      RL=EIWDTH(J)
      GAMA1=RL/(RL+3*EI/SRSTF3(I,J))
      GAMA2=RL/(RL+3*EI/SRSTF6(I,J))
      ASR=GAMA1+GAMA2+GAMA1*GAMA2
      BSR=GAMA1*(2+GAMA2)
      CSR=3*GAMA1
      DSR=3*GAMA2
      FSR=3*GAMA1*GAMA2
      GSR=GAMA2*(2+GAMA1)
      AHSR=4-GAMA1*GAMA2
      AKB(I,J,1,1)=OEA/RL
      AKB(I,J,1,2)=0
      AKB(I,J,1,3)=0
      AKB(I,J,1,4)=OEA*(-1)/RL
      AKB(I,J,1,5)=0
      AKB(I,J,1,6)=0
      AKB(I,J,2,1)=0
      AKB(I,J,2,2)=12*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,2,3)=6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,2,4)=0
      AKB(I,J,2,5)=12*(-1)*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,2,6)=6*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,3,1)=0
      AKB(I,J,3,2)=6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,3,3)=4*EI*CSR/(RL*AHSR)
      AKB(I,J,3,4)=0
      AKB(I,J,3,5)=(-1)*6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,3,6)=2*EI*FSR/(RL*AHSR)
      AKB(I,J,4,1)=(-1)*OEA/RL
      AKB(I,J,4,2)=0
      AKB(I,J,4,3)=0
      AKB(I,J,4,4)=OEA/RL
      AKB(I,J,4,5)=0
      AKB(I,J,4,6)=0
      AKB(I,J,5,1)=0
```

```
      AKB(I,J,5,2)=12*(-1)*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,5,3)=(-1)*6*EI*BSR/(AHSR*RL*RL)
      AKB(I,J,5,4)=0
      AKB(I,J,5,5)=12*EI*ASR/(AHSR*RL*RL*RL)
      AKB(I,J,5,6)=6*(-1)*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,6,1)=0
      AKB(I,J,6,2)=6*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,6,3)=2*EI*FSR/(RL*AHSR)
      AKB(I,J,6,4)=0
      AKB(I,J,6,5)=6*(-1)*EI*GSR/(AHSR*RL*RL)
      AKB(I,J,6,6)=4*EI*DSR/(AHSR*RL)

1402
1391 CONTINUE
1390 CONTINUE

2237 FORMAT(2X,I3,2X,F10.3,1X,F10.5,1X,F10.5,1X,F10.3,1X,F10.5,1X,F10.5
     *)

      IF(NPCHK.NE.1) GO TO 1559
1423
      NITNO=1
1559
      DO 1447 I=1,NDEG
      PAPP(I)=PAPPSR(I)*ANLOAD*0.001


      PAPP2(I)=PAPP(I)
1447 CONTINUE

      NITER=9999

      IF(NPCHK.LT.3) GO TO 1361
      IF(NSRCHK.EQ.1.AND.NITNO.LE.NITER.AND.KONT.EQ.0) THEN

      NNOO=NNOO+1
      NITNO=NITNO+1
      SECOND=SECOND+1

      DO 1392 I=1,NFLR
      DO 1393 J=1,NBAY
      IF(AKAPA(I,J).EQ.0) GO TO 1415
      PBSR(I,J,1)=PBSR(I,J,1)+PB(I,J,1)
      PBSR(I,J,2)=PBSR(I,J,2)+PB(I,J,2)
      PBSR(I,J,3)=PBSR(I,J,3)+PB(I,J,3)
      PBSR(I,J,4)=PBSR(I,J,4)+PB(I,J,4)
      PBSR(I,J,5)=PBSR(I,J,5)+PB(I,J,5)
      PBSR(I,J,6)=PBSR(I,J,6)+PB(I,J,6)
      ROTZ3(I,J)=ROTAT3(I,J)
      ROTZ6(I,J)=ROTAT6(I,J)
1415
1393 CONTINUE
1392 CONTINUE


      DO 1417 I=1,NFLR
      DO 1418 J=1,NBAY+1
      DO 1419 K=1,6
      PKSR(I,J,K)=PKSR(I,J,K)+PK(I,J,K)
1419 CONTINUE
1418 CONTINUE
1417 CONTINUE


      DO 1420 I=1,NDEG
      DSRGD(I)=DSRGD(I)+D(I)
1420 CONTINUE

      WRITE(15,*) SRSTF3(4,2)

      DO 1561 I=1,NBAY+1
```

```
      SHEAR(SECOND)=SHEAR(SECOND)+PKSR(1,I,1)
 1561 CONTINUE

      DO 1788 I=NFLR,1,-1
      JJO=NFLR+(NFLR-1)*NBAY
      JOINT=I+(I-1)*NBAY+1-1
      DEFOR(I)=DSRGD(3*JOINT-2)/DSRGD(3*JJO-2)
 1788 CONTINUE
      WRITE(14,*) DEFOR(NFLR),DEFOR(NFLR-1),DEFOR(NFLR-2)
      DO 1439 I=1,NDEG
      PAPP(I)=PAPPSR(I)*ANLOAD/1000
 1439 CONTINUE

c       WRITE(15,*) NITNO
c       WRITE(15,304) PKSR(1,1,1),PKSR(1,2,1),PBSR(1,1,1),PBSR(2,1,1)
c       WRITE(15,304) PKSR(1,1,2),PKSR(1,2,2),PBSR(1,1,2),PBSR(2,1,2)
c       WRITE(15,304) PKSR(1,1,3),PKSR(1,2,3),PBSR(1,1,3),PBSR(2,1,3)
c       WRITE(15,304) PKSR(1,1,4),PKSR(1,2,4),PBSR(1,1,4),PBSR(2,1,4)
c       WRITE(15,304) PKSR(1,1,5),PKSR(1,2,5),PBSR(1,1,5),PBSR(2,1,5)
c       WRITE(15,304) PKSR(1,1,6),PKSR(1,2,6),PBSR(1,1,6),PBSR(2,1,6)


      ELSE
      ENDIF

      DO 1781 I=1,NFLR
      DO 1782 J=1,NBAY
      DO 1783 K=1,6
      PKIRIS(I,J,K)=0
 1783 CONTINUE
 1782 CONTINUE
 1781 CONTINUE

      IF(NITNO.EQ.(NITER+1)) GO TO 1421


      IF(NITNO.EQ.25) THEN
c       WRITE(10,304) PKSR(1,1,1),PKSR(1,2,1),PKSR(1,3,1),PBSR(2,1,1)
c       WRITE(10,304) PKSR(1,1,2),PKSR(1,2,2),PKSR(1,3,2),PBSR(2,1,2)
c       WRITE(10,304) PKSR(1,1,3),PKSR(1,2,3),PKSR(1,3,3),PBSR(2,1,3)
c       WRITE(10,304) PKSR(1,1,4),PKSR(1,2,4),PKSR(1,3,4),PBSR(2,1,4)
c       WRITE(10,304) PKSR(1,1,5),PKSR(1,2,5),PKSR(1,3,5),PBSR(2,1,5)
c       WRITE(10,304) PKSR(1,1,6),PKSR(1,2,6),PKSR(1,3,6),PBSR(2,1,6)
      ELSE
      ENDIF


      TJ=NFLR+(NFLR-1)*NBAY
      SWAY(SECOND)=DSRGD(3*TJ-2)-TTOP
 1361

C     ****************************************************************
C     ****************************************************************
C     ****************************************************************
C     ********************   GLOBAL STIFFNESS   **********************
C     ****************************************************************
C     ****************************************************************
C     ****************************************************************

      DO 1202 J=1,NDEG
      DO 1203 I=1,NDEG
      ADEG(I,J)=0.0
 1203 CONTINUE
 1202 CONTINUE

C     *********************DIAGONAL ELEMENTS*************************

      NFLRC=1
      NBAYC=0
      DO 1172 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
```

```
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K)=AKC(I,J,4,4)+AKC((I+1),J,1,1)+AKB(I,J,1,1)+AKB(I,(J-1),4
     *,4)
 1172 CONTINUE

      NFLRC=1
      NBAYC=0
      DO 1175 K=3,NDEG,3
      NBAYC=NBAYC+1
      IF((K-3)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-3)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K)=AKC(I,J,6,6)+AKC((I+1),J,3,3)+AKB(I,J,3,3)+AKB(I,(J-1),6
     *,6)
 1175 CONTINUE

      NFLRC=1
      NBAYC=0
      DO 1178 K=2,(NDEG-1),3
      NBAYC=NBAYC+1
      IF((K-2)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
```

```
      ENDIF
      IF((K-2)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-2)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K)=AKC(I,J,5,5)+AKC((I+1),J,2,2)+AKB(I,J,2,2)+AKB(I,(J-1),5
     *,5)
 1178 CONTINUE

C     ***************** FOR SAME JOINT ***********

      NFLRC=1
      NBAYC=0
      DO 1192 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K+1)=AKC(I,J,4,5)+AKC((I+1),J,1,2)+AKB(I,(J-1),4,5)+AKB(I,J
     *,1,2)
      ADEG(K+1,K)=ADEG(K,K+1)
      ADEG(K,K+2)=AKC(I,J,4,6)+AKC((I+1),J,1,3)+AKB(I,(J-1),4,6)+AKB(I,J
     *,1,3)
      ADEG(K+2,K)=ADEG(K,K+2)
      ADEG(K+1,K+2)=AKC(I,J,5,6)+AKC((I+1),J,2,3)+AKB(I,(J-1),5,6)+AKB(I
     *,J,2,3)
      ADEG(K+2,K+1)=ADEG(K+1,K+2)

 1192 CONTINUE

C     ***** ON THE RIGHT *******

      NFLRC=1
      NBAYC=0
      DO 1195 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=2
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=4
```

```
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC
      ADEG(K,K+3)=AKB(I,J,1,4)
      ADEG(K+3,K)=ADEG(K,K+3)
      ADEG(K+1,K+3)=AKB(I,J,2,4)
      ADEG(K+3,K+1)=ADEG(K+1,K+3)
      ADEG(K+2,K+3)=AKB(I,J,3,4)
      ADEG(K+3,K+2)=ADEG(K+2,K+3)


      ADEG(K,K+4)=AKB(I,J,1,5)
      ADEG(K+4,K)=ADEG(K,K+4)
      ADEG(K+1,K+4)=AKB(I,J,2,5)
      ADEG(K+4,K+1)=ADEG(K+1,K+4)
      ADEG(K+2,K+4)=AKB(I,J,3,5)
      ADEG(K+4,K+2)=ADEG(K+2,K+4)


      ADEG(K,K+5)=AKB(I,J,1,6)
      ADEG(K+5,K)=ADEG(K,K+5)
      ADEG(K+1,K+5)=AKB(I,J,2,6)
      ADEG(K+5,K+1)=ADEG(K+1,K+5)
      ADEG(K+2,K+5)=AKB(I,J,3,6)
      ADEG(K+5,K+2)=ADEG(K+2,K+5)

 1195 CONTINUE
C     *************** COLUMN TOP AND BOTTOM ******************
      NFLRC=2
      NBAYC=0
      DO 1194 K=1,(NDEG-2),3
      NBAYC=NBAYC+1
      IF((K-1)/3.EQ.(NBAY+1)) THEN
        NFLRC=3
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(2*NBAY+2)) THEN
        NFLRC=4
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(3*NBAY+3)) THEN
        NFLRC=5
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(4*NBAY+4)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      IF((K-1)/3.EQ.(5*NBAY+5)) THEN
        NFLRC=6
        NBAYC=1
      ENDIF
      I=NFLRC
      J=NBAYC

      ADEG(K,(K+3*(NBAY+1)))=AKC(I,J,1,4)
      ADEG((K+3*(NBAY+1)),K)=ADEG(K,(K+3*(NBAY+1)))
      ADEG(K,(K+1+3*(NBAY+1)))=AKC(I,J,1,5)
      ADEG((K+1+3*(NBAY+1)),K)=ADEG(K,(K+1+3*(NBAY+1)))
      ADEG(K,(K+2+3*(NBAY+1)))=AKC(I,J,1,6)
      ADEG((K+2+3*(NBAY+1)),K)=ADEG(K,(K+2+3*(NBAY+1)))
```

```
      ADEG(K+1,(K+3*(NBAY+1)))=AKC(I,J,2,4)
      ADEG((K+3*(NBAY+1)),K+1)=ADEG(K+1,(K+3*(NBAY+1)))
      ADEG(K+1,(K+1+3*(NBAY+1)))=AKC(I,J,2,5)
      ADEG((K+1+3*(NBAY+1)),K+1)=ADEG(K+1,(K+1+3*(NBAY+1)))
      ADEG(K+1,(K+2+3*(NBAY+1)))=AKC(I,J,2,6)
      ADEG((K+2+3*(NBAY+1)),K+1)=ADEG(K+1,(K+2+3*(NBAY+1)))

      ADEG(K+2,(K+3*(NBAY+1)))=AKC(I,J,3,4)
      ADEG((K+3*(NBAY+1)),K+2)=ADEG(K+2,(K+3*(NBAY+1)))
      ADEG(K+2,(K+1+3*(NBAY+1)))=AKC(I,J,3,5)
      ADEG((K+1+3*(NBAY+1)),K+2)=ADEG(K+2,(K+1+3*(NBAY+1)))
      ADEG(K+2,(K+2+3*(NBAY+1)))=AKC(I,J,3,6)
      ADEG((K+2+3*(NBAY+1)),K+2)=ADEG(K+2,(K+2+3*(NBAY+1)))

 1194 CONTINUE



C     ****************** FOR HINGE BASE ***************************

      IF(NPCHK.NE.1) THEN
      NDEG4=NDEG5
      NDEG3=NDEG5
      ELSE
      ENDIF

      DO 1306 J=1,NBAY+1
      IF(IBASE(J).EQ.1) THEN
      NDEG3=NDEG3+1
      ADEG(NDEG3,NDEG3)=AKC(1,J,3,3)
      ADEG(3*J-2,NDEG3)=AKC(1,J,3,4)
      ADEG(NDEG3,3*J-2)=ADEG(3*J-2,NDEG3)
      ADEG(3*J-1,NDEG3)=AKC(1,J,3,5)
      ADEG(NDEG3,3*J-1)=ADEG(3*J-1,NDEG3)
      ADEG(3*J,NDEG3)=AKC(1,J,3,6)
      ADEG(NDEG3,3*J)=ADEG(3*J,NDEG3)
      ENDIF
 1306 CONTINUE



C     **************************************************************
C                           END OF GLOBAL STIFFNESS
C     **************************************************************

      IF(NPCHK.NE.1) THEN
      DO 1341 I=1,NDEG
      PAPP(I)=PAPP2(I)

 1341 CONTINUE
      ELSE
      ENDIF

C     ***************** GAUSS ELIMINATION ******************************

      DO 1253 I=1,NDEG-1
      DO 1265 K=NDEG,1+I,-1
      AKSAYI=ADEG(K,I)/ADEG(I,I)*(-1)
      DO 1255 J=I,NDEG
      ADEG(K,J)=ADEG(K,J)+ADEG(I,J)*AKSAYI
 1255 CONTINUE
      PAPP(K)=PAPP(K)+PAPP(I)*AKSAYI
 1265 CONTINUE
 1163 FORMAT(5X,F14.4)
 1253 CONTINUE

C     **********************   DISPLACEMENTS     ******************

      D(NDEG)=PAPP(NDEG)/ADEG(NDEG,NDEG)

      DO 1300 I=NDEG-1,1,-1
```

```
      TOPLAM(I)=0
      DO 1303 J=NDEG,I+1,-1
      TOPLAM(I)=TOPLAM(I)+ADEG(I,J)*D(J)

 1303 CONTINUE
      D(I)=(PAPP(I)-TOPLAM(I))/ADEG(I,I)
 1300 CONTINUE


      IF(NPCHK.EQ.2) THEN
      DO 1972 I=1,NDEG
      DSRGD(I)=D(I)
 1972 CONTINUE
      ELSE
      ENDIF


C     ****************** MEMBER FORCES ***************************

      DO 1307 I=1,NFLR
      DO 1308 J=1,NBAY+1
      IF(I.EQ.1.AND.IBASE(J).EQ.1) NDEG4=NDEG4+1
      DO 1309 K=1,6
      JOINT=I+(I-1)*NBAY+J-1
      L=(JOINT-NBAY-1)*3-2
      IF(I.EQ.1.AND.IBASE(J).EQ.1) THEN
           PK(I,J,K)=AKC(I,J,K,3)*D(NDEG4)+AKC(I,J,K,4)*D(JOINT*3-2)+AKC
     *(I,J,K,5)*D(JOINT*3-1)+AKC(I,J,K,6)*D(JOINT*3)
      IF(NPCHK.EQ.2) PKSR(I,J,K)=PK(I,J,K)
      ELSE
           PK(I,J,K)=AKC(I,J,K,1)*D(L)+AKC(I,J,K,2)*D(L+1)+AKC(I,J,K,3)*
     *D(L+2)+AKC(I,J,K,4)*D(JOINT*3-2)+AKC(I,J,K,5)*D(JOINT*3-1)+AKC(I,J
     *,K,6)*D(JOINT*3)
      ENDIF
 1309 CONTINUE
 1308 CONTINUE
 1307 CONTINUE

      IF(NPCHK.NE.1) THEN
      DO 1333 I=1,NFLR
      DO 1334 J=1,NBAY+1
      DO 1335 K=1,6
      DO 1336 L=1,6
      AKC(I,J,K,L)=AKC(I,J,K,L)+GKC(I,J,K,L)
 1336 CONTINUE
 1335 CONTINUE
 1334 CONTINUE
 1333 CONTINUE
      ELSE
      ENDIF

      IF(NPCHK.EQ.1) GO TO 1322

C     ******************** BEAM FORCES **************************
      DO 1310 I=1,NFLR
      DO 1311 J=1,NBAY
      DO 1312 K=1,6
      JOINT=I+(I-1)*NBAY+J-1
      L=JOINT*3-2
      PB(I,J,K)=(AKB(I,J,K,1)*D(L)+AKB(I,J,K,2)*D(L+1)+AKB(I,J,K,3)*D(
     *L+2)+AKB(I,J,K,4)*D((JOINT+1)*3-2)+AKB(I,J,K,5)*D((JOINT+1)*3-1)+A
     *KB(I,J,K,6)*D((JOINT+1)*3))
      IF(NPCHK.EQ.2) PBSR(I,J,K)=PB(I,J,K)

 1312 CONTINUE
 1311 CONTINUE
 1310 CONTINUE



 1998 FORMAT(F10.5)
```

```
      DO 4321 I=1,NBAY+1
      XX=0.0001*PKSR(1,I,2)/AC(1,I)
      YY=48000*AIC(1,I)/DC(1,I)
      SINIR=YY-XX
      IF(PKSR(1,I,3).GT.SINIR) GO TO 1421
      IF(PKSR(1,I,6).GT.SINIR) GO TO 1421
 4321 CONTINUE


      TOPDIS=THGHT(NFLR)*0.025
      IF(DSRGD((NFLR+(NFLR-1)*NBAY)*3-2).LT.TOPDIS) GO TO 1322
      GO TO 1421

 1421


C     ****************************************************************
 9328

  345 FORMAT(/,15X,'**  ANALYSIS IS COMPLETED!..  **',/,/,5X,'DISPLACEME
     *NTS ARE IN THE OUTPUT FILE ..DATA.OUT..!..')
      WRITE(6,345)
      WRITE(10,*) '    ***** DEFORMATIONS  *****'
      DO 346 I=1,NDEG
      IF(NSRCHK.EQ.1) THEN
      WRITE(10,304) I,DSRGD(I)
      ELSE
      WRITE(10,304) D(I)
      ENDIF
  346 CONTINUE
      WRITE(6,*) SECOND
  304 FORMAT(10X,F14.9,2X,F14.9,2X,F14.9,2X,F14.9,2X,F14.9,2X,F14.9)
      WRITE(10,*) '*****************'
      IF(NSRCHK.EQ.1) THEN
      WRITE(10,304) PKSR(1,1,1),PKSR(1,2,1),PKSR(1,3,1),PKSR(1,4,1)
      WRITE(10,304) PKSR(1,1,2),PKSR(1,2,2),PKSR(1,3,2),PKSR(1,4,2)
      WRITE(10,304) PKSR(1,1,3),PKSR(1,2,3),PKSR(1,3,3),PKSR(1,4,3)
      WRITE(10,304) PKSR(1,1,4),PKSR(1,2,4),PKSR(1,3,4),PKSR(1,4,4)
      WRITE(10,304) PKSR(1,1,5),PKSR(1,2,5),PKSR(1,3,5),PKSR(1,4,5)
      WRITE(10,304) PKSR(1,1,6),PKSR(1,2,6),PKSR(1,3,6),PKSR(1,4,6)
      WRITE(10,*) 'KOLON MOMENTLERI-SR'
      ELSE
      WRITE(10,304) PK(1,1,1),PK(1,2,1),PK(1,3,1),PK(1,4,1)
      WRITE(10,304) PK(1,1,2),PK(1,2,2),PK(1,3,2),PK(1,4,2)
      WRITE(10,304) PK(1,1,3),PK(1,2,3),PK(1,3,3),PK(1,4,3)
      WRITE(10,304) PK(1,1,4),PK(1,2,4),PK(1,3,4),PK(1,4,4)
      WRITE(10,304) PK(1,1,5),PK(1,2,5),PK(1,3,5),PK(1,4,5)
      WRITE(10,304) PK(1,1,6),PK(1,2,6),PK(1,3,6),PK(1,4,6)
      WRITE(10,*) 'KOLON MOMENTLERI-RGD'
      ENDIF
      WRITE(6,1001) NITNO

  562 FORMAT(5X,F16.8,5X,F16.8)
      DO 348 I=1,SECOND
      WRITE(12,562) SWAY(I),-SHEAR(I)
  348 CONTINUE

  601 FORMAT(5X,'******** INPUT DATA SRFP *********')
      WRITE(11,601)
  608 FORMAT(5X,'..!  UNITS ARE TONS AND METERS  !..')
      WRITE(11,608)
  607 FORMAT(/,5X,'MODULUS OF ELASTICITY........=',F10.1)
      WRITE(11,607) E
  602 FORMAT(5X,'NUMBER OF FLOORS............=',I1)
      WRITE(11,602) NFLR
  603 FORMAT(5X,'NUMBER OF BAYS..............=',I1)
      WRITE(11,603) NBAY
  604 FORMAT(/,5X,'FLOOR #',10X,'FLOOR HEIGHT',/,4X,'---------',8X,'----
     *---------')
      WRITE(11,604)
```

```
      DO 570 I=1,NFLR
 605  FORMAT(7X,I1,15X,F6.4)
      WRITE(11,605) I,EIHGHT(I)
 570  CONTINUE
 606  FORMAT(/,6X,'BAY #',11X,'BAY WIDTH',/,4X,'---------',8X,'---------
     *----')
      WRITE(11,606)
      DO 571 I=1,NBAY
 612  FORMAT(7X,I1,15X,F6.4)
      WRITE(11,612) I,EIWDTH(I)
 571  CONTINUE
 609  FORMAT(/,5X,'COLUMN FLOOR',5X,'COLUMN NUMBER',7X,'AREA',6X,'INERTI
     *A',/,4X,'------------',6X,'-------------',6X,'------',5X,'--------
     *-')
      WRITE(11,609)
 610  FORMAT(8X,I1,16X,I1,13X,F8.7,6X,F8.7)
      DO 572 I=1,NFLR
      DO 573 J=1,NBAY+1
      WRITE(11,610) I,J,AC(I,J),AIC(I,J)
 573  CONTINUE
 572  CONTINUE
 613  FORMAT(/,7X,'BEAM FLOOR',6X,'BEAM BAY',10X,'AREA',6X,'INERTIA',/,4
     *X,'------------',6X,'-------------',6X,'------',5X,'---------')
      WRITE(11,613)
 614  FORMAT(8X,I1,16X,I1,13X,F8.7,6X,F8.7)
      DO 574 I=1,NFLR
      DO 575 J=1,NBAY
      WRITE(11,614) I,J,AB(I,J),AIB(I,J)
 575  CONTINUE
 574  CONTINUE

1614  FORMAT(2X,F6.2,2X,F6.2,2X,F6.2,2X,F6.2)
      WRITE(16,*) 'FIRST FLOOR COLUMNS MOMENTS(DOWN)!'
      WRITE(16,1614) PKSR(1,1,3),PKSR(1,2,3),PKSR(1,3,3),PKSR(1,4,3)
      WRITE(16,*) 'FIRST FLOOR COLUMNS MOMENTS(UP)!'
      WRITE(16,1614) PKSR(1,1,6),PKSR(1,2,6),PKSR(1,3,6),PKSR(1,4,6)

      CLOSE(5)
      CLOSE(10)
      CLOSE(11)
      CLOSE(12)
      CLOSE(14)
      CLOSE(15)
      CLOSE(16)

      STOP
      END
```