A SIMPLIFIED NON-LINEAR SOIL-STRUCTURE INTERACTION (SSI) MODEL FOR THE EARTHQUAKE RESPONSE OF SURGE VESSELS RESTING ON SHALLOW FOUNDATIONS

by

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ABSTRACT

A SIMPLIFIED NON-LINEAR SOIL-STRUCTURE INTERACTION MODEL FOR THE EARTHQUAKE RESPONSE OF SURGE VESSELS RESTING ON SHALLOW FOUNDATIONS

In this study, earthquake response of a surge vessel which consists of 8 interconnected tanks is studied. This HPV unit is located in western Saudi Arabia with 600.000 m^3/day desalination capacity.

The focus of this study is to see and evaluate the response of the structure under soil structure interaction effects. Therefore the properties of the superstructure are kept the same for all the analysis. SSI parameters and earthquake input motions are changed in order to represent the various properties of soil and earthquake motions. The comparisons of analyses results to earthquake codes are made. Some suggestions are made to overcome the existing weakness and increase the performance of the structures. SSI model is developed and various non-linear analyses are conducted and the results are evaluated and compared. Also the difference in response of the structure is compared with and without SSI considered.

In the first chapter of this study, the scope of the work and general information about the surge vessel is given. In the second chapter, the methodology is explained. In the third chapter modelling and analyses are explained. Modelling and analyses are not kept as separate chapters in the name of being compatible with the development flow of the study. And the analyses results are evaluated and comparisons are made along with the development. In the fourth and final chapter conclusions are presented and suggestions for improving this study are proposed.

ÖZET

SIĞ TEMELLERE OTURMUŞ DENGELEME TANKLARININ DEPREM TEPKİSİ İÇİN BASİTLEŞTİRİLMİŞ, DOĞRUSAL OLMAYAN YAPI ZEMİN ETKİLEŞİM (SSI) MODELİ

Bu çalışmada birbirine bağlı 8 tanktan oluşan bir yüksek basınç dengeleme ünitesinin (HPV) deprem tepkisi çalışılmıştır. Bu HPV ünitesi Suudi Arabistan'ın batısında yer almaktadır ve 600.000 m^3/gün 'lük su arıtma kapasitesine sahiptir.

Bu çalışmada üzerine odaklanılan konu yapı zemin etkileşimi etkileri altında yapının tepkisini görmek ve değerlendirmektir. Bu sebeple tüm analizlerde üst yapı özellikleri sabit tutulmuştur. SSI parametreleri ve kullanılan deprem kayıtları, çeşitli zemin özellikleri ve deprem hareketlerini temsil etmek adına değiştirilmiştir. Analiz sonuçları ile deprem yönetmeliklerinin karşılaştırılmaları yapılmıştır. Yapıdaki mevcut sıkıntılar ve bu zayıflıkları gidermek ve yapının deprem performansını arttırmak adına bazı önerilerde bulunulmuştur. SSI modeli geliştirilmiş ve çeşitli doğrusal olmayan analizler de yapılarak sonuçlar değerlendirilmiş ve karşılaştırılmıştır. Aynı zamanda yapının SSI etkileri altında ve bu etkilerin göz önüne alınmadığı durumlardaki tepkileri karşılaştırılmıştır.

Bu çalışmanın ilk bölümünde işin kapsamı ile basınç ünitesi hakkında genel bilgiler verilmiştir. İkinci bölümde çalışmanın yöntembilimi anlatılmıştır. Üçüncü bölümde modelleme ve analizler anlatılmıştır. Çalışmanın gelişim akışı ile uyumlu olmak adına modelleme ve analizler farklı bölümler olarak ele alınmamıştır. Modelin geliştirilmesi süresince yapılan analizin sonuçları değerlendirilmiş ve karşılaştırmalar yapılmıştır. Dördüncü ve son bölümde ise çalışmanın sonuçları sunulmuş ve bu çalışmanın geliştirilmesi için önerilerde bulunulmuştur.

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LIST OF SYMBOLS / ABBREVIATIONS

B	Short dimension of rectangular foundation
c	Damping
d	Inside diameter
D	Outside diameter
E	Modulus of elasticity of the structure
F_a	Site coefficient (IBC)
F_v	Site coefficient (IBC)
G	Shear modulus of soil
h	Height of the structure
I	Moment of Inertia
k	Stiffness
k_R	Rotational spring stiffness for SSI
k_y	Translational spring stiffness for SSI
K_H	Translational spring stiffness for SSI
K_θ	Rotational spring stiffness for SSI
L	Long dimension of rectangular foundation
n_i	modal frequency of the mode “i"
m	mass
M	Mass per unit height
r_0	Equivalent radius of the foundation
S_{DS}	Design spectral acceleration for 0.2 seconds period (IBC)
S_{D1}	Design spectral acceleration for 1 second period (IBC)
S_a	Long period spectral acceleration (IBC)
S_s	0.2 seconds spectral acceleration
S_1	1 second spectral acceleration
t	Thickness of the tank
T_i	Modal period of the mode “i" (fixed base)
$\overline{T}$	Fundamental period of SSI system
u	relative displacement to the ground
u_f	ground displacement
V_s	Shear wave velocity

$\overline{\beta}$	Equivalent damping factor of SSI system
β_0	Foundation damping factor (radiation damping)
β	Damping ratio of the structure on fixed base
θ	Rotation
ν	Poisson ration of soil
ρ	Soil density

3D	Three Dimensional
API	American Petroleum Institute
AS / NZS	Australian / New Zealand Standard
ASCE	American Society of Civil Engineers
EL	Elastic Link
FE	Finite Elements
FEAP	Finite Element Analysis Program
FEMA	Federal Emergency Management Agency
HPV	High Pressure Vessel
IBC	International Building Code
NEHRP	National Earthquake Hazards Reduction Program
NL	Non-Linear
NZSEE	New Zealand Society for Earthquake Engineering
PEER	Pacific Earthquake Engineering Research Centre
PL	Plastic Link
SSD	Safe Shut Down
SSI	Soil-Structure Interaction

1. INTRODUCTION

1.1. Preliminary Information about High Pressure Vessels

Turkey is located on an important transit energy corridor that transfers oil and natural gas from Russia, Caucasia, Iran and Iraq into Europe via pipelines. On the other hand, in our country, with the increase in demand of water, very big capacity water conveyance lines are being constructed as well. High Pressure Vessels (HPV) is one of the critical support elements in such facilities.

High pressure vessels are used for safety purposes against water hammering due to emergency shutdown and re-feeding stages in pipe transmission lines. In the pipelines, due to fires and earthquakes, with the cutting off of liquid flow; significant pressure surging occurs. Although the duration of these cut offs may be short, their damaging effect on the overall system can be very devastating. Tank-pipe connections and supporting units may break, distribution systems can be interrupted. HPV's are critical industrial members used for regulating the pressure resulting from such kind of sudden changes or interruptions in the liquid flow rate. They protect from flange separation, leaks or major piping damage and are considered as "Safe Shut Down" (SSD) components for water/crude oil transmission systems. These members are widely used in crude oil and water transmissions lines as well as power plants, petrochemical facilities and sea-water desalination plants. Therefore the potential damages may cause cutting of water distribution and it is also dangerous for the safety of the facility itself.

Depending on the geometric and material properties of the soil-structure system and the hazard level, such tanks can be subjected to appreciable amounts of soil structure interaction effects (SSI) even during moderate earthquakes. Uplift of the base due to rocking motion may lead to early separations at the soil-foundation interface which might then cause permanent deformations in the structure. HPV's cannot be replaceable and cannot be fixed and rehabilitated in a short period of time. Therefore they need to be safe and should not undergo excessive deformations or rotations during earthquakes. So they need to be designed as per low damage levels.

Typically, an HPV unit consists of a number of tall, slender and rigid steel cylinders all supported by a common foundation and connected by a steel walkway / catwalk at the top. As the case study, the earthquake response of a surge vessel which consists of 8 interconnected tanks is studied. The site is located at a 600.000 m³/day water production capacity Desalination Plant, in the Western Saudi Arabia. It is an HPV unit of a sea water desalination plant located 200 kilometres north of Jeddah city, Saudi Arabia, desalinating the Yanbu Sea. (Figure 1) A simplified 3D representation drawing of the system is also presented. (Figures 1.2 1.3 1.4)

Figure 1.1. Different tank groups in HPV stations.

Desalination plants are facilities where salty sea water is resolved and potable water is produced. During the desalination process a great amount of steam energy is also released. Therefore such facilities are also used for generating power. For this reason these facilities are commonly named as Desalination and Power Plants. Aforementioned facility supplies potable water to the city of Madinah, with the capacity of 600,000 m³/day. The latest earthquake experienced in the area was in 2009 with the magnitude of 5.7 called “Al-Ais” earthquake. The distance from epicentre of the earthquake to the facility is approximately 140 kilometres. (Figure 1.5)

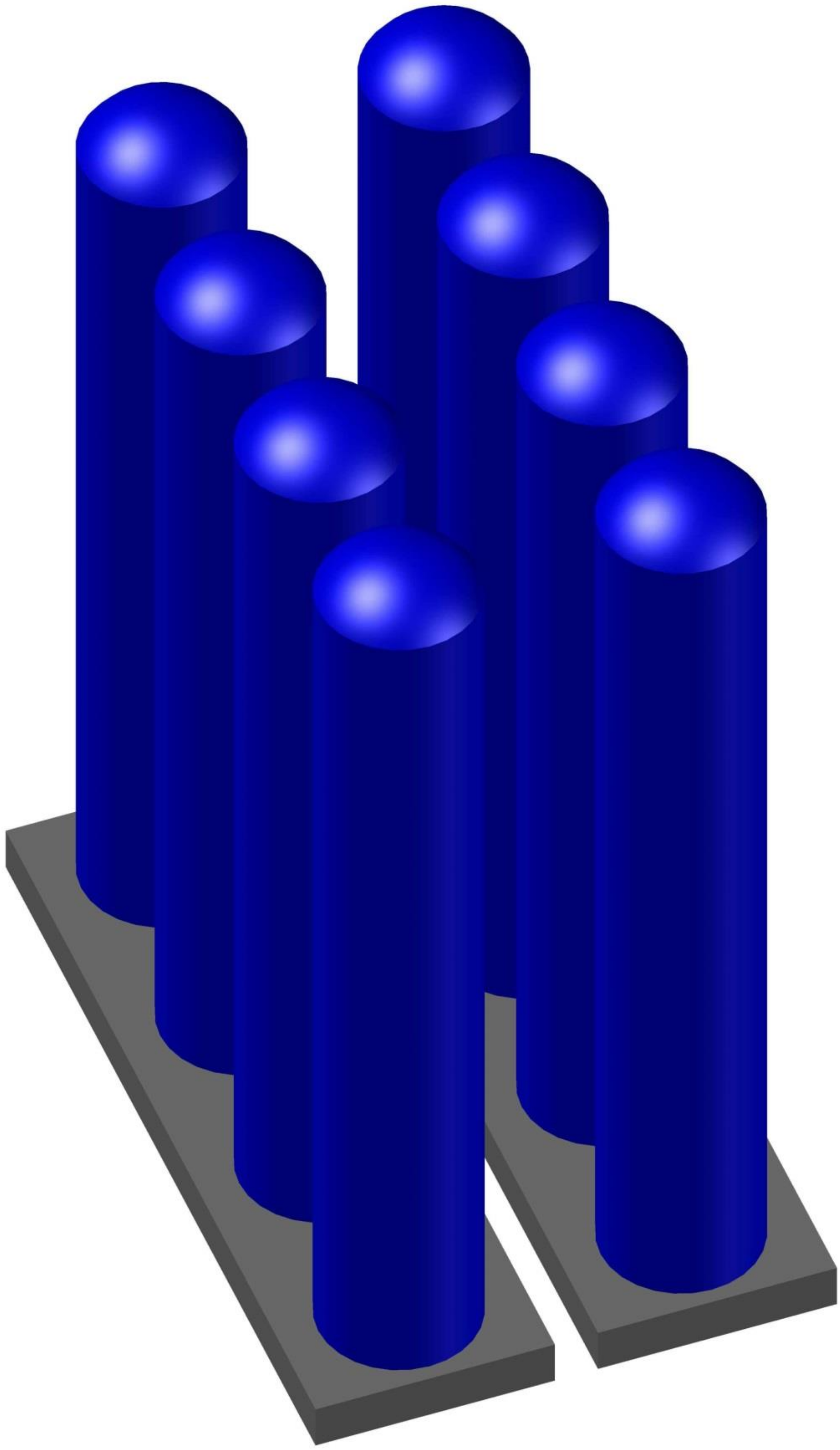


Figure 1.2. Simplified 3D drawing of the HPV.

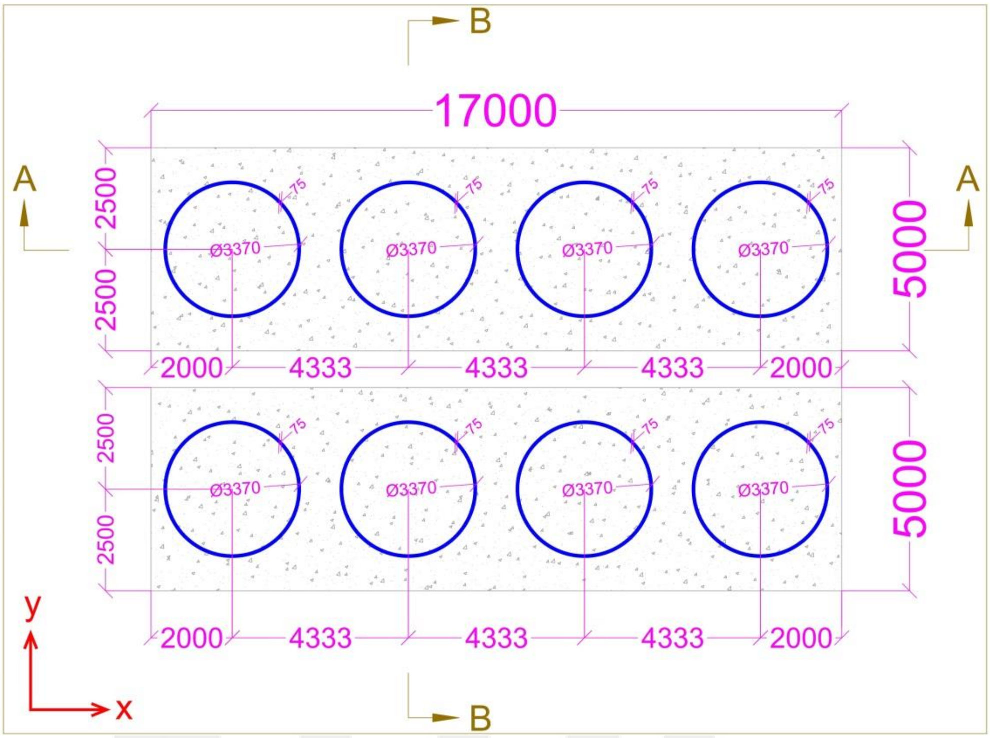


Figure 1.3. Plan view of simplified HPV model.

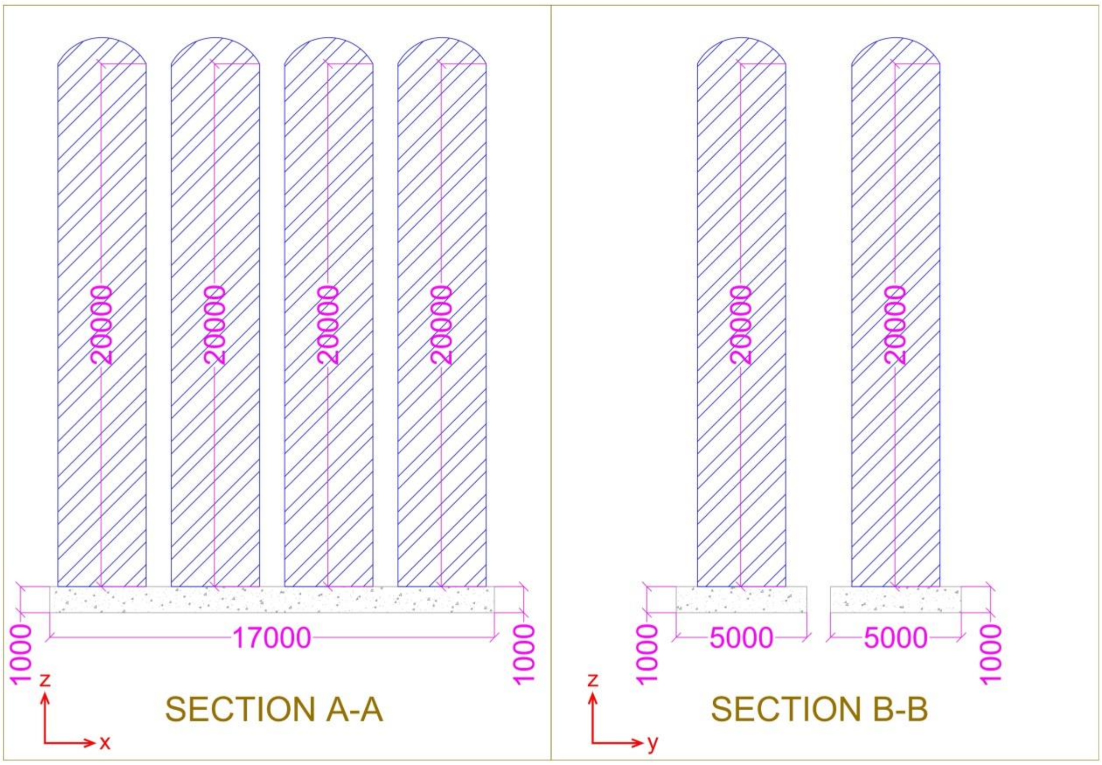


Figure 1.4. Cross-sections of simplified HPV model.

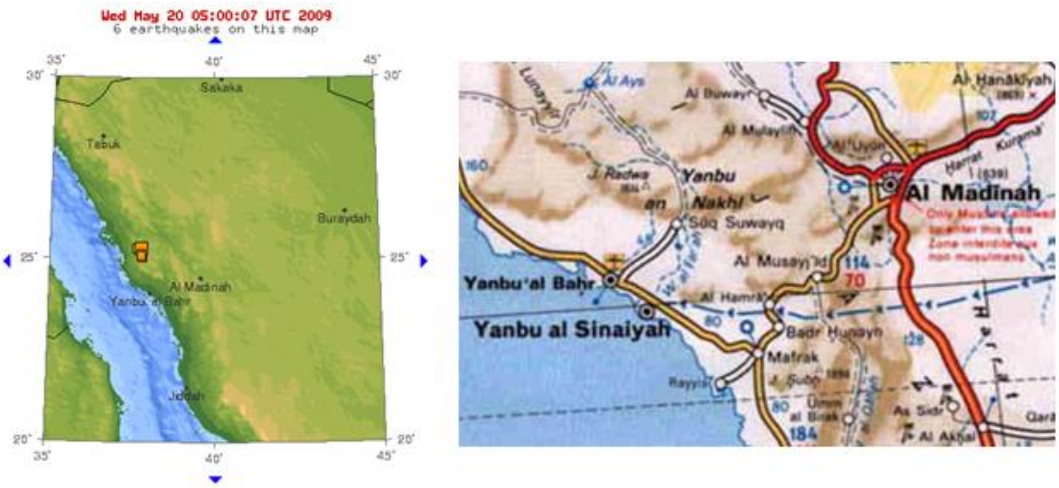


Figure 1.5. Location of HPV and Al-Ais earthquake, 2009.

The HPV station taken into account in this study consists of two sets of four HPV tanks resting on foundations with dimensions of 17m x 5m in plan and 1m in depth. (Figures 1.2, 1.3, 1.4) The tanks are anchored to the foundation with steel anchor bolts. A typical tank group consists of tanks, pipe connections and flow control unit as shown in (Figure 1.6) below.

Figure 1.6. Flexible pipe connection and typical deformation.

1.2. Effects of Soil-Structure Interaction (SSI)

Non-linear dynamic soil-foundation interaction effects can produce benefits to the overlying super-structure, protecting it from excessive plasticization and development of ductility and permanent displacements at the end of the excitation. However in return; deformations at the SSI interface may increase.

In the conventional design approach it is generally assumed that the superstructure and the soil connection is infinitely stiff, thus it is modelled as a fixed base support. Therefore it is assumed that the superstructure does not undergo any displacement relative to foundation-structure system. This approach is reasonable when foundation and the soil are rigid. However if the structure is supported on soft soil, when the soil’s elasticity is increased or when the foundation system’s rigidity is not so big; this assumption becomes no more reasonable or do not reflect the actual situation. In such cases the effect of foundation-soil system to the superstructure becomes different than the situation where the soil is assumed to be supported by a fixed support, resulting significant change of internal forces and displacement of the structure.

In this study, the earthquake response of a structure under the effects of SSI, considering different soil conditions is examined.

As damping in SSI system two damping mechanisms are considered;

- Hysteretic damping; due to plastic deformation of soil
- Geometric / radiation damping (viscous damping) representing the damping of vibrational energy radiating away from the foundation into the infinite soil medium

For the superstructure; Rayleigh damping which is proportional to mass and stiffness is taken into account.

Hysteretic damping is calculated from the area corresponding to hysteretic diagram of Moment versus Rotation response graph. Geometric damping is represented by a viscous damping constant as per the formulae presented in the literature. Damping of superstructure is represented by proportional damping terms.

Soil structure interaction (SSI) can simply be represented visually as given in Figure 1.7 proposed by Veletsos and Meek (1974) as a three-degree of freedom mathematical model. The effect of SSI is more significant for structures supported on soft soil. This condition is considered in FEMA 450 and NZSEE 1986 and relevant parameters are presented according to soil conditions. u_f, θ, and u represent ground displacement, rotation and relative displacement to ground respectively. Equations of motions for each degree of freedom are written below.

$$m\left(\ddot{u_f} + h\ddot{\theta} + \ddot{u}\right) + c\dot{u} + ku = -m\ddot{u_g} \tag{1.1}$$

$$m\left(\ddot{u_f} + h\ddot{\theta} + \ddot{u}\right) + m_f\ddot{u_f} + c_u\dot{u_f} + k_u u_f = -(m + m_f)\ddot{u_g} \tag{1.2}$$

$$mh\left(\ddot{u_f} + h\ddot{\theta} + \ddot{u}\right) + I\ddot{\theta} + c_\theta\dot{\theta} + k_\theta\theta = -mh\ddot{u_g} \tag{1.3}$$

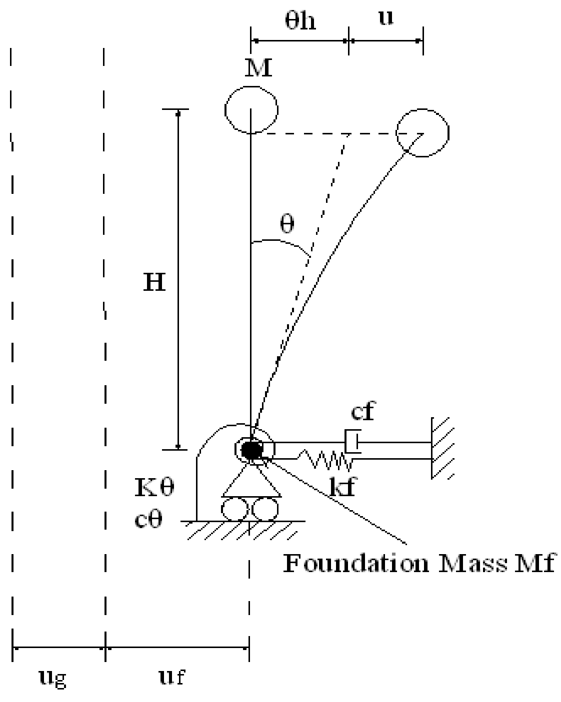


Figure 1.7. Simplified SSI model (FEMA 450).

In the literature; provisions and criteria for seismic response and design of tanks considering SSI effects are discussed and provided: Veletsos at al., 1988; FEMA 368-369 (2000), NEHRP (2003), FEMA 450, ASCE 7–02.7, AS/NZS 1170.3, API 650 and ASCE 41-06.

In ASCE – 41 for example at Section 11.10.2; possible failure cases are defined as below:

- Failure of anchor bolts supporting the vertical tanks
- Bucking at the base and failing of supports
- Overturning and rotation
- Anchor capacity

Although the superstructure and anchor connections of such systems are designed to resist very high level loadings, they can be subjected to significant rotations due to soil structure interaction. These deformations can even be non-linear and plastic. Also depending on soil conditions, as a result of soil structure interaction; foundation rotations may increase furthermore.

According to Dowrick (2007) the prerequisite for a system that SSI effects should be considered is given below:

$$V_s/(f.h) \leq 20 \qquad (1.4)$$

Here;

V_s : shear wave velocity (m/s)

f : Frequency of the superstructure supported by a fixed base (Hz)

h : height of the structure (m)

As a result of soil structure interaction, due to increase in period and damping, base shear is expected to be reduced. This relation is given via the formula provided below (NEHRP, 97) proposed by *Mylonakis and Gazetas*:

$$\Delta V = \left[C_s(T,\beta) - C_s(\overline{T},\beta) \left(\frac{\beta}{\overline{\beta}} \right)^{0.4} \right] W \tag{1.5}$$

C_s : *Seismic Response Coefficient*

T : *Natural Period of fixed base system*

$\overline{T}$: *Vibration period of SSI system (sec.)*

$\overline{\beta}$: *Equivalent damping factor of SSI system*

β_0 : *Foundation damping factor (radiation damping)*

W : *Equivalent seismic weight*

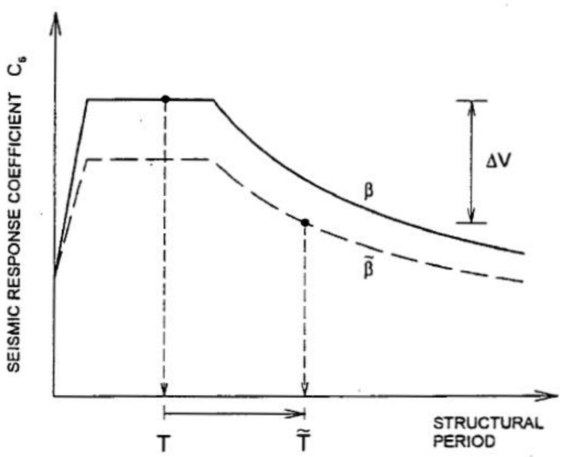


Figure 1.8. Base shear reduction due to SSI (NEHRP, 97).

A simplified representation of SSI and comparison of structural characteristics according to NEHRP, 97 is presented below:

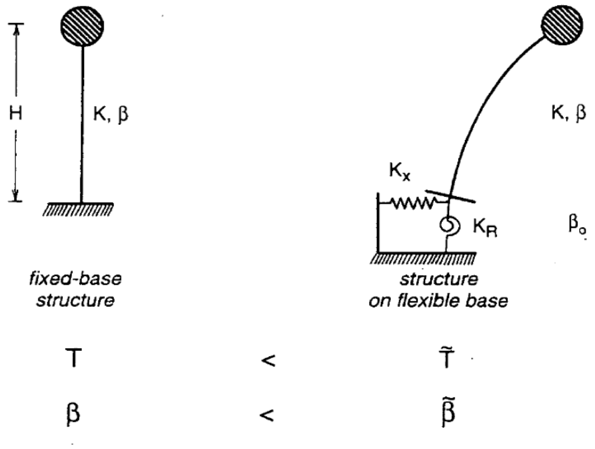


Figure 1.9. SSI according to NEHRP 97.

Fundamental structural characteristics of SSI system (period, damping, stiffness) and their relationship to fixed based system's fundamental characteristics are given in the graphic below. (NEHRP, 97)

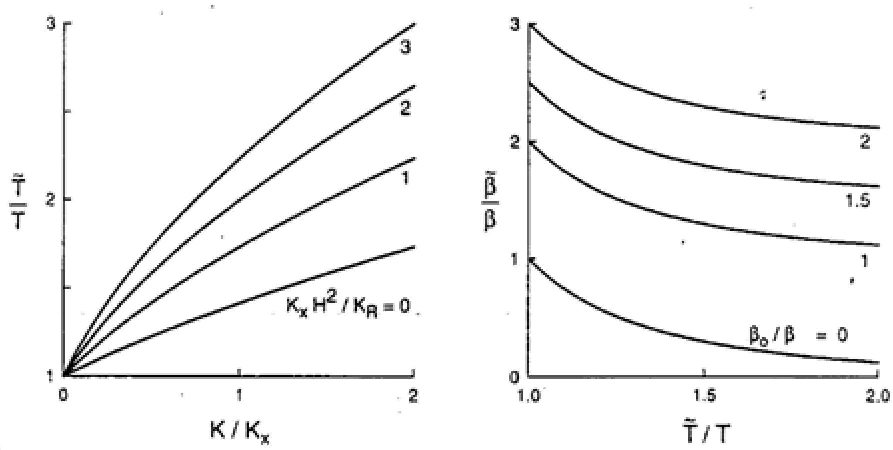


Figure 1.10. Relationship of structural characteristics for SSI (NEHRP 97).

1.3. Scope and Objective

In this study, the difference in response and deformations of the system with and without the effects of SSI is examined. It is also studied that how the structural response is changing under different soil conditions when SSI is considered. Non-linear behaviour of soil and how the structural response and deformations are changing is also studied. And finally the additional effect of foundation uplift and the change in structural response is examined. The results are compared and evaluated.

The main assumptions made for this study are noted below:

- Base rotation is considered as the main engineering demand parameter
- It is assumed that the super structure is assumed to behave linearly elastic and all nonlinearity occurs at the SSI interface.

Due to non-linear effects; permanent plastic foundation deformations may increase uncontrollably at structures resting on shallow foundations. This situation may result in failures and breakings at tank-pipe connection points. In this study, the conditions which the effects of plastic behaviour and foundation uplift will be effective are examined as well as their effects on total base rotation. With some modifications, it is suitable to note that the developed mathematical model can be adapted for building-type structures as well.

In order to be able to focus on soil-structure interaction, all the numerical parameters and properties of the superstructure kept the same throughout the entire study. The changed parameters are of the SSI level in order to represent different soil parameters and earthquake excitations. It is noted that this study may stand as an example for facilities of similar capacities.

In the earlier parts of the study, the execution of relevant codes and formulas presented in the literature are applied to the considered structure and the results are evaluated. Some practical and design-based precautions are proposed in order to get rid of possible weaknesses and failures of the system. Then the mathematical model is developed and various linear analyses are conducted. Finally model is further developed and various non-linear time history analyses are conducted. Analyses results are evaluated and compared.

2. METHODOLOGY

2.1. Aim and Scope

The objective of this thesis is to evaluate and compare the earthquake response of a surge vessel structure when SSI is considered. A simplified mathematical model is used and it is developed to represent and take into account of various linear and non-linear properties of SSI interface. The foundation-soil system is first modelled as a fixed support (without the effects of SSI) then with linear elastic links (to consider soil stiffness and SSI) and developed into non-linear plastic and non-linear elastic links to take into account the plastic behaviour of soil and uplift effects respectively.

The earthquake response of the structure is calculated via response spectrum analyses and non-linear time history analyses. Various real earthquake ground motions are used provided from PEER ground motion database. Some of these ground motions are modified in order to match the response spectrum used in response spectrum analysis cases. These can be called as artificial or modified earthquakes. Various responses of the structure for different analysis cases and soil conditions are calculated, evaluated and compared. Based on the results of these various parametric analyses some conclusions are made and suggestions are proposed.

The detailed steps of the analyses and development of the model is presented in the next chapter. As this is a parametric study, it is more suitable to present the development of the model and analyses concurrently.

2.2. Development

In this study a macro-element formulation has been introduced, specifically suited for the modelling of the nonlinear dynamic soil-structure interaction of shallow foundations. Two sources of non-linearity are considered at the soil-foundation interface:

a) The one due to the irreversible elastoplastic soil behaviour (material nonlinearity)
b) The one due to possible foundation uplift (geometric non-linearity)

Initially the soil is assumed to be in full contact with the foundation. Next, the effects of soil plasticity (nonlinear plastic link element) and uplift (multilinear elastic element) are added. Overall behaviour is assumed to be the combination of these elements. Yield strength for the plastic element and threshold overturning moment for the uplift element are defined. These two elements are connected in series to represent their coupled behaviour. The response of the model to real earthquake records is calculated. Due to shallow depth of the foundation kinematic interaction effects are ignored. This indicates that the recorded ground motion can be used as the input acceleration for the SSI model without any requirement for modification.

SAP2000 FE software is used to solve the equations of motions. Initially frequencies of the fixed and flexible base models are computed. Later, a series of detailed linear and nonlinear analyses are performed.

In this study, various simplified linear and nonlinear analyses are conducted for an HPV unit subjected to soil-structure interaction. As the critical engineering demand parameter base rotation angle is considered. The development steps of the study are given below:

1. Survey of relevant codes and literature
2. Creation of general mathematical model on SAP2000 FE software
3. Developing fixed base model and linear elastic SSI model
4. Defining the Response Spectrum and Analyses
5. Evaluation and comparison of results
6. Developing the SSI model (NL-plastic link member)
7. Non-linear time history analyses using the plastic link member
8. Developing the SSI model by adding uplift effects (NL-elastic link member)
9. Non-linear time history analyses using the plastic + elastic link members
10. Evaluation and comparison of analyses results

3. MODELLING AND ANALYSES

3.1. Preliminary Calculations

3.1.1. Superstructure Natural Frequency (fixed base)

First of all, the natural vibration period and damping ratio of the superstructure supported by a fixed base is calculated and determined.

For steel tanks supported by a fixed base, natural vibration frequency is calculated via the formula given below. (ASCE Guide for Seismic Evaluation and Design of Petrochemical Facilities 4.A-4)

$$n_i = \frac{k_i}{h^2}\left(\sqrt{\frac{EI}{M}}\right) \tag{3.1}$$

In this formulation parameters are;

n_i : *modal frequency of the mode "i" (Hz)*

k_i : *mode constant*

Table 3.1. Mode constants (ASCE-4A-4).

Mode Constant	
Mode	Constant
1	0.56
2	3.51
3	9.82
4	19.20

h : *Height of the structure (m)*

E : *Modulus of elasticity of the structure (Pa)*

I : *Moment of Inertia (m^4) [$I= \pi.d^3.t / 8$]*

M : *Mass per unit height (kg/m or $N/s^2/m^2$)*

d : *Inside diameter (m)*

t : *Thickness of the tank (m)*

Physical properties of the system:

h: 20m, E: 2E+11 Pa, I:0.983m^4, t: 0.075m, d:3.22m, m:100t, M:5t/m

Using the formulation presented above fundamental natural frequency of the system supported by a fixed base is calculated as 8.78 Hz. and period (1/T) is 0.11 seconds.

Period can also be calculated alternatively using the formula below:

$$T_i = 7.78\ x\ 10^{-6} \left(\frac{h}{D}\right)^2 \left(\sqrt{\frac{12MD}{t}}\right) \tag{3.2}$$

In this formulation parameters are;

T_i : *modal period of the mode "i" (seconds)*

h : *Height of the structure (ft.)*

D : *Outside diameter (ft.)*

M : *Mass per unit height (lb/ft)*

t : *Thickness of the tank (in.)*

3.1.2. Damping Ratio (fixed base)

When the literature is reviewed it can be observed that typically the elastic damping ratio of similar steel structures is around %1 - %2. Smaller values are for rock and/or stiff soils whereas higher damping values are for softer soils.

In this study, superstructure damping ratio (β) for the structure on fixed base is selected as 0.02 (2%).

3.1.3. Spring Constants (SSI)

Foundation system consists of two independent rectangular shallow mat foundations. Each foundation supports four tanks. (Figures 1.2, 1.3 and 1.4)

Foundation system is critical for rotations about the x-axis. (Figures 1.3 and 1.4) Therefore the rotational spring stiffness "k_Θ" is considered for the rotation about the x-axis and the translational spring stiffness k_y is considered for the direction of y-axis.

The spring stiffness values used for foundation-soil interaction are calculated via the empirical formulae proposed by *Pais and Kausel*. As the considered system is a shallow and mat foundation, it is suitable to use these spring stiffness values for rectangular foundations.

Translational spring stiffness terms:

$$K_{Hy}^{o} = \frac{GB}{(2-\nu)}\left(6.8\left(\frac{L}{B}\right)^{0.65} + 2.4\right) \tag{3.3}$$

$$K_{Hx}^{o} = \frac{GB}{(2-\nu)}\left(6.8\left(\frac{L}{B}\right)^{0.65} + 0.8\frac{L}{B} + 1.6\right) \tag{3.4}$$

Rotational spring stiffness terms:

$$K_{Rx}^{o} = \frac{GB^3}{8(1-\nu)}\left(3.2\ \frac{L}{B} + 0.8\right) \tag{3.5}$$

$$K_{Ry}^{o} = \frac{GB^3}{8(1-\nu)}\left(3.73\left(\frac{L}{B}\right)^{2.4} + 0.27\right) \tag{3.6}$$

In this formulation parameters are;

K_H : *Translational spring stiffness (kN/m)*

K_R : *Rotational stiffness (kN.m./rad)*

G : *Shear modulus of soil (Pa)*

B : *Short dimension of rectangular foundation (m)*

L : *Long dimension of rectangular foundation (m)*

ν : *Poisson ratio of soil*

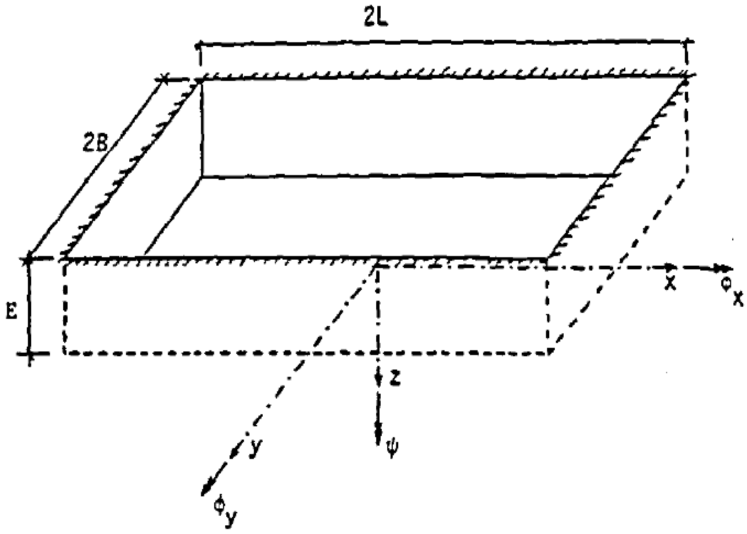


Figure 3.1. Static spring stiffness values for rectangular foundations (L≥B) (A. Pais and E. Kausel).

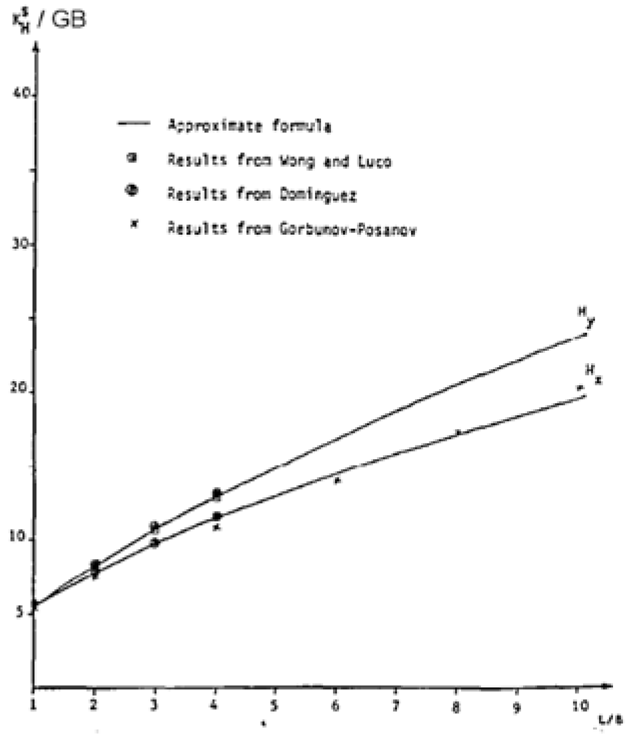


Figure 3.2. Static spring stiffness values for rectangular foundations (L≥B) (A. Pais and E. Kausel).

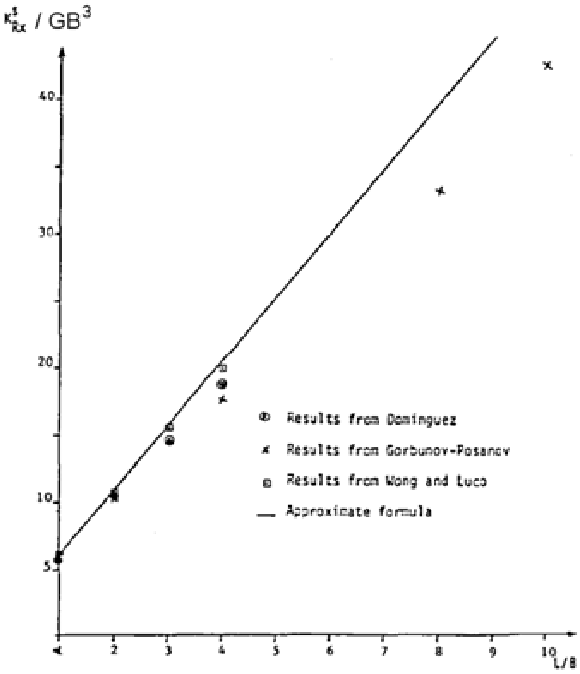


Figure 3.3. Static spring stiffness values for rectangular foundations (L≥B) (A. Pais and E. Kausel).

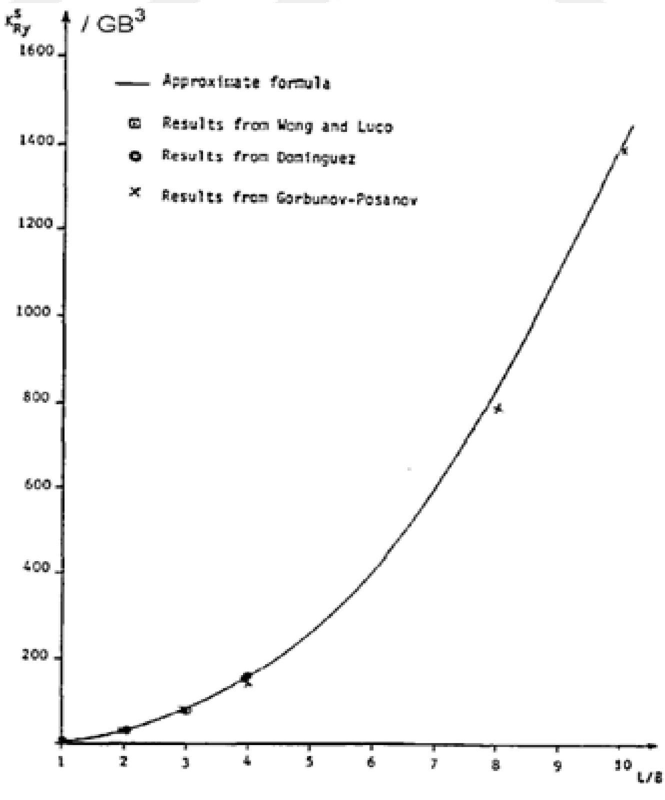


Figure 3.4. Static spring stiffness values for rectangular foundations (L≥B) (A. Pais and E. Kausel).

Table 3.2. Approximate formulas for dynamic stiffness of rigid foundations

(A. Pais and E. Kausel)

MODE		L/B=1	L/B=2	L/B=3	L/B=4
Vertical	Wong (v=1/3)	4.66	6.73	8.56	10.22
$\frac{K_v(1-\nu)}{GB}$	Dominguez	4.88	7.00	8.90	10.70
	Formula	4.70	6.81	8.66	10.36
Horizontal -x	Wong (v=1/3)	9.22	12.95	16.19	19.15
$\frac{K_{Hx}(2-\nu)}{GB}$	Dominguez	9.47	13.10	16.30	19.30
	Formula	9.20	13.07	16.29	19.14
Horizontal -y	Wong (v=1/3)	9.22	13.75	17.79	21.48
$\frac{K_{Hy}(2-\nu)}{GB}$	Dominguez	9.47	14.00	18.10	21.80
	Formula	9.20	13.87	17.89	21.54
Rocking-x	Wong (v=1/3)	4.17	7.18	10.30	13.18
$\frac{K_{Rx}(1-\nu)}{GB^3}$	Dominguez	3.85	6.80	9.75	12.80
	Formula	4.00	7.20	10.40	13.60
Rocking-y	Wong (v=1/3)	4.17	20.21	52.26	104.21
$\frac{K_{Ry}(1-\nu)}{GB^3}$	Dominguez	3.85	19.60	50.60	105.30
	Formula	4.00	19.96	52.37	104.18

The values shown in Figure 3.1, 3.2, 3.3 and 3.4 are given numerically in Table 3.2. The spring stiffness values are marked as well in Table 3.2 for the studied system.

The spring stiffness values calculated for this study are given below:

k_y : *5,000,000 kN/m*

k_θ : *20,000,000 kN.m / rad*

3.1.4. Superstructure Natural Period (SSI)

Using the spring stiffness values calculated in the previous section, fundamental vibration period of the superstructure is recalculated taking into account the SSI. SSI Period is calculated using the formula given below:

Rotational spring stiffness terms:

$$\frac{\overline{T}}{T} = \sqrt{1 + \frac{k}{K_H}\left(1 + \frac{K_H h^2}{K_\theta}\right)} \tag{3.7}$$

In this formulation parameters are;

$\overline{T}$: *Vibration period of SSI system (sec.)*

T : *Vibration period of system on a fixed base (sec.)*

K_H : *Translational spring stiffness (kN/m)*

K_R : *Rotational stiffness (kN.m./rad)*

h : *Height of the structure (m)*

SSI period of the system is calculated as 0.20 seconds. It should be noted that fixed base period was 0.11 seconds. Effects of this change will be examined in further sections.

3.1.5. Equivalent Damping for SSI System

After calculating the SSI period, the equivalent damping for the SSI system is calculated via the formula given below (NEHRP, 2009):

$$\overline{\beta} = \beta_0 + \frac{0.05}{\left(\frac{\overline{T}}{T}\right)^3} \tag{3.8}$$

Here;

$\overline{\beta}$: *Equivalent damping factor of SSI system*

β_0 : *Foundation damping factor (radiation damping)*

$\overline{T}$: *Vibration period of SSI system (sec.)*

T : *Vibration period of system on a fixed base (sec.)*

T and $\overline{T}$ was calculated in previous sections. β_0 can be selected using the tables presented in NEHRP, 2009 on Chapter 19.

In the studied case, site class corresponds to Class C. (NEHRP, 2009 and IBC, 2006) and corresponding S_{DS} is calculated to be lower than 0.2. Therefore β_0 is found as 0.03 from the table presented below. (Figure 19.2-1 – NEHRP, 2009) When all values are substituted $\overline{\beta}$ is calculated about 5%.

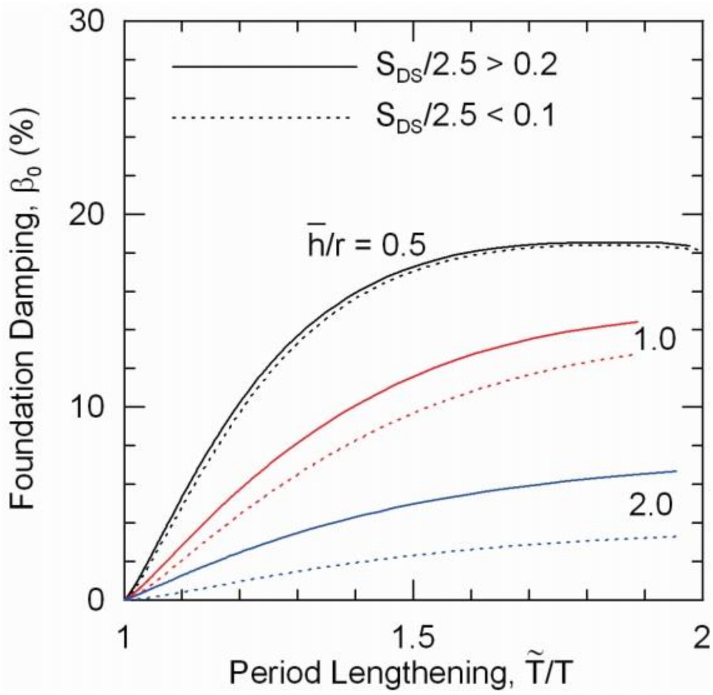


Figure 3.5. Foundation damping β_0 (%) (NEHRP, 2009).

It should be noted that damping ratio of the system supported by a rigid base was selected as 0.02. $\bar{\beta}$ is calculated as 0.05. It is important to note that damping ratio of the system is increased from 2% to 5% when fixed base and SSI systems are compared.

Damping of the wave amplitude in the subsoil due to movement is caused by hysteretic behaviour of the ground. This damping phenomenon is represented by various empirical formulae in the literature. For example damping characteristics are presented by *Witmann* by empirical formulations below:

$$c_x = 0.576 k_x r_0 \sqrt{\rho / G} \tag{3.9}$$

$$c_\theta = \frac{0.3}{1 + \frac{3(1-\nu)m}{8 r_{xx}^5 \rho}} k_\theta r_\theta \sqrt{\rho / G} \tag{3.10}$$

Here;

k_x : *Translational spring constant*

k_θ : *Rotational spring constant*

r_0 : *Equivalent radius of the foundation*

ρ : *Soil density*

G : *Shear modulus of soil*

v : *Poisson ratio of soil*

m : *foundation mass*

3.2. Modelling of the Superstructure

3.2.1. Material Properties

In SAP2000 FE, first materials properties are defined. Material is chosen as steel and mechanical properties are given as below:

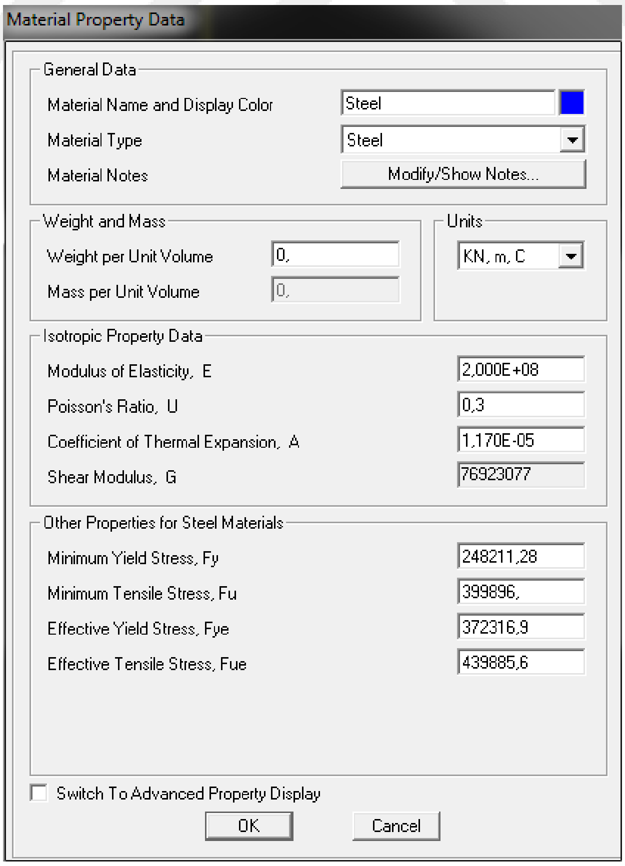


Figure 3.6. Mechanical properties of steel.

It is important to note that the material is defined massless. In the name of simplicity lumped mass assumption is taken into consideration for the superstructure. This modelling method is explained briefly in the coming sections.

3.2.2. Cross-Sectional Properties

Circular hollow frame section is used to represent the superstructure i.e., the HPV tank. Material is selected as steel as defined in the previous section. The outside diameter and thickness of the cross section are selected as 3.37m and 75mm, respectively as given in Figure 3.7 below:

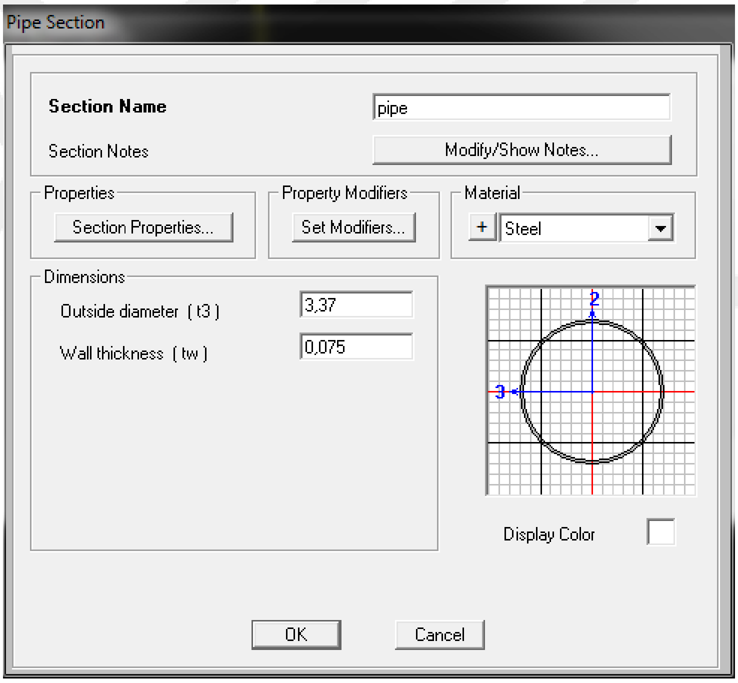


Figure 3.7. Cross-section properties.

3.2.3. General Superstructure Model

The properties of the superstructure are defined and modelled as given below:

- Height of the tank, h: 20 meters
- Mass of the tank, m: 100 tons = 10 x 10tons lumped masses with 2m spacing (global y direction) (Figure 3.8)
- Foundation dimensions, 17meters x 5meters x 1meter,

- Mass of the foundation, m_f= 200 tons (in the global y direction)
- Mass moment of inertia of the foundation: I_m=400 ton.m² (global x direction)
- Frame section is placed as a longitudinal frame element with lumped masses assigned on each end. Uniformly distributed lumped masses are assigned along the beam to represent the mass of the superstructure. (Figure 3.8)

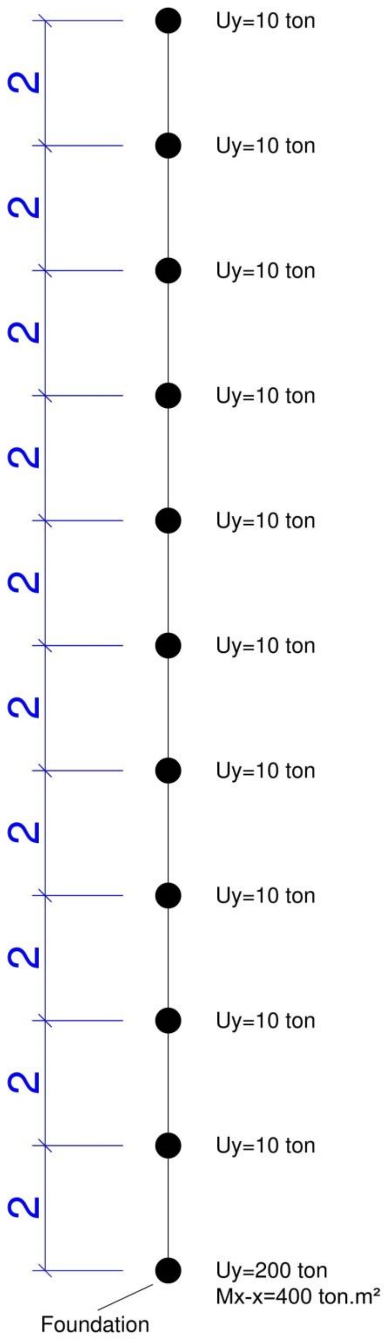


Figure 3.8. Simplified superstructure model.

3.2.4. Developing of "Model A" (Fixed Base)

Four general models are developed in this study. The first one is the model with a fixed base assumption. This model is named as "Model A". In this model no SSI effects are considered. Soil-foundation system is assumed to be infinitely stiff.

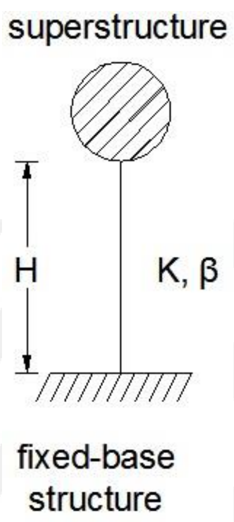


Figure 3.9. Visual representation of model A.

Modal damping ratio is selected and assigned to the system as 2% according to provisions suggested in the literature. This damping is assumed to represent the overall damping in the system.

First of all, for "Model A" modal analysis is conducted. The fundamental vibration period of the system is calculated as 0.125 seconds. This coincides with the result of hand calculation which was around 0.11 seconds.

3.2.5. Developing of "Model B" (Linear SSI)

In this part, the soil-structure interaction is taken into consideration. A simple linear elastic link member is defined. And as spring constants (spring stiffness) which are calculated on Section 3.3 according to empirical formulations proposed by "*Pais* and *Kausel"* are defined as translational and rotational stiffness properties of this Linear Link member.

The spring stiffness values calculated on Section 3.3 for this study are given below:

k_y : 5,000,000 kN/m

k_θ : 20,000,000 kN.m / rad

The other 4 degrees of freedom are kept infinitely stiff for the sake of simplicity. This Link member is assigned to the base of the structure to represent the interaction between superstructure and foundation-soil system. This model is named as “Model B”. Below is a visual representation of Model B.

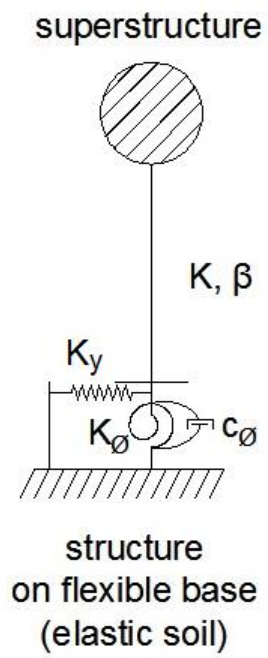


Figure 3.10. Visual representation of model B.

Modal damping ratio is selected and assigned to the system as 5% as calculated in Section 3.5 as proposed in NEHRP, 2009. This damping is assumed to represent the overall damping in the system.

Same as previous section, for “Model B” modal analysis is conducted. The fundamental vibration period of the system is calculated as 0.216 seconds. This coincides with the result of hand calculation which was around 0.20 seconds.

It is important to note that both damping and period of the system is increased when SSI is considered. The effects of these will be compared and evaluated in the next sections.

In Table 3.5 and Table 3.6 presented below, the comparison of fundamental structural parameters is made.

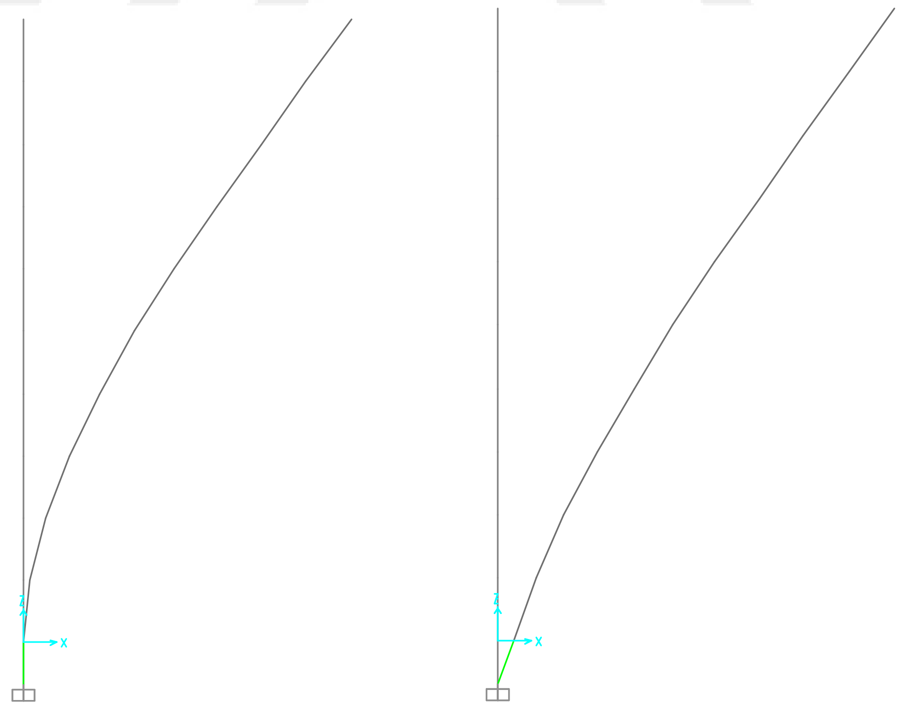
Table 3.3. Fundamental periods.

	Fundamental Period	
	Fixed Base	**SSI System**
Hand Calculation	0.11	0.20
SAP 2000	0.125	0.216

Table 3.4. Damping ratio.

Damping Ratio (β)	
Fixed Base	**SSI System**
2%	5%

In the Figure 3.9 below, fundamental mode shapes for both fixed base and SSI systems are presented visually.



(Fixed base) (Linear-Elastic SSI Base)

Figure 3.11. Fundamental mode shapes of fixed base and SSI models.

3.3. Response Spectrum Analyses

3.3.1. Response Spectrum

Response spectrum according to IBC, 2006 for the given soil and site conditions is plotted. The input site and soil parameters are given below:

S_s : 0.61g (0.2 seconds spectral acceleration)
S_1 : 0.41g (1 second spectral acceleration)
Site Class: C

Taking into account the parameters above IBC spectrum parameters are calculated and selected as below:

F_a : 1 (site coefficient for Site Class C)
F_v : 1.3 (site coefficient for Site Class C)
S_{DS} : 4 (design spectral acceleration for 0.2 seconds period)
S_{D1} : 3.47 (design spectral acceleration for 1 second period)

S_{DS} and S_{D1} are calculated using the formulae below as presented as IBC, 2006

$$S_{DS} = \frac{2}{3} F_a S_s \tag{3.11}$$

$$S_{D1} = \frac{2}{3} F_v S_1 \tag{3.12}$$

Long period spectral acceleration values are calculated via the formula below as presented in IBC.

$$S_a = \frac{S_{D1}}{T} \tag{3.13}$$

All the values are calculated and response spectrum for the given soil and site conditions is plotted. It is shown in Figure 3.10 below:

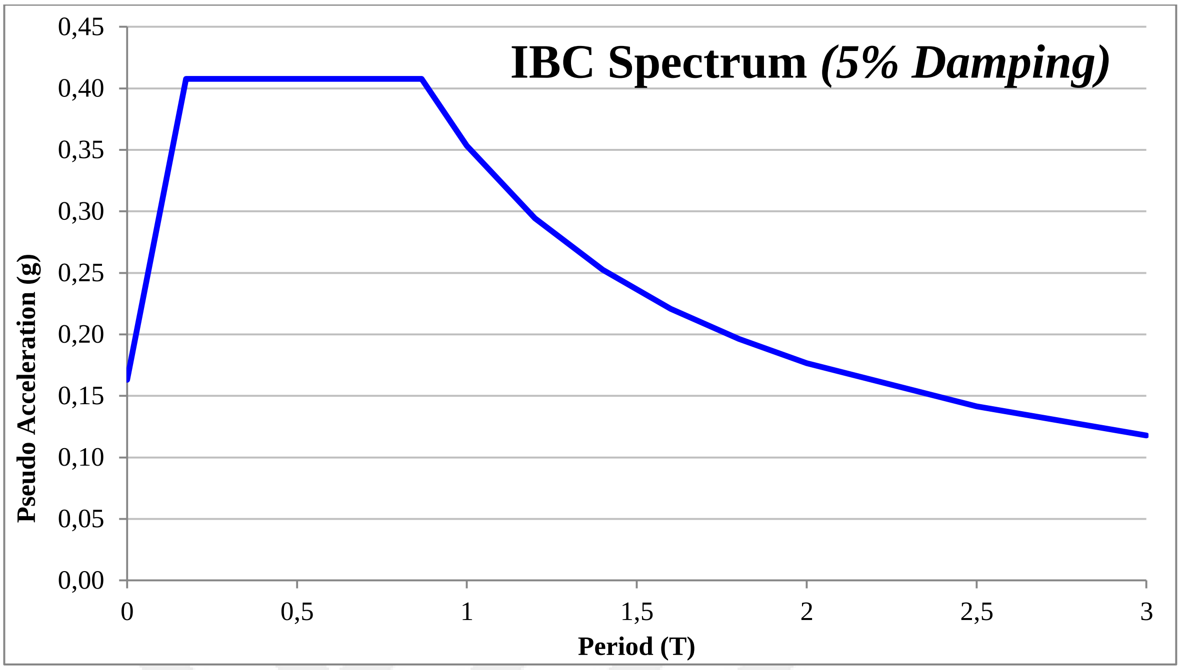


Figure 3.12. Response spectrum (IBC).

3.3.2. Analyses and Results

The response spectrum defined in the previous chapter is also defined in the SAP 2000 mathematical models: previously named as Model A and Model B.

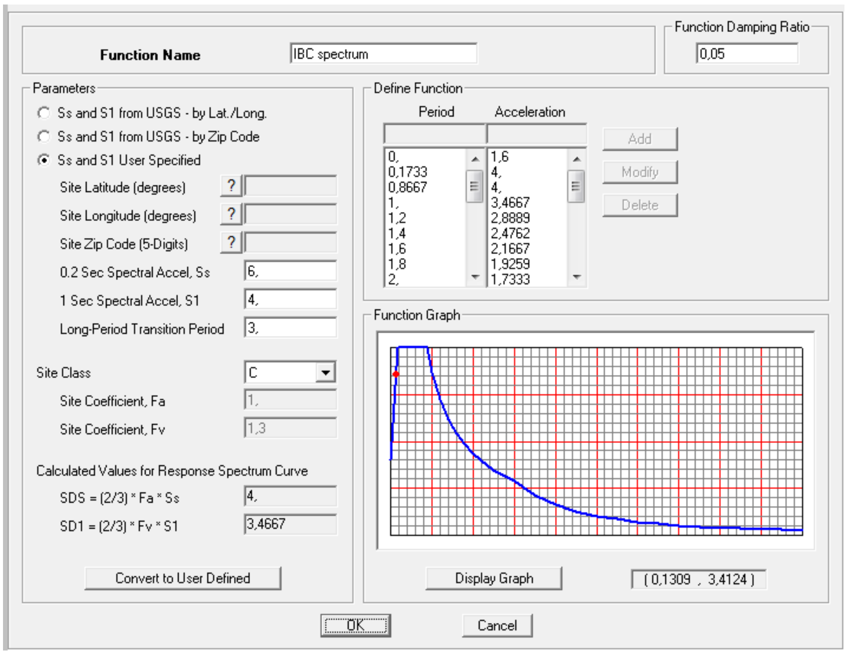


Figure 3.13. Response spectrum (SAP2000).

Using this response spectrum, response spectrum analyses are conducted for both Model A (fixed base system) and Model B (linear link – SSI system).

Analysis results in terms of top displacement, base displacement, base shear force and base moment are calculated and presented below:

Table 3.5. Displacements (response spectrum analyses).

Location	Displacement (mm)	
	Fixed Base (Model A)	SSI System (Model B)
Top	2.40	6.84
Base	0.00	0.10

Table 3.6. Base responses (response spectrum analyses).

Response	Fixed Base (Model A)	SSI System (Model B)
Base Shear Force (kN)	276	303
Base Moment (kN.m)	4091	4303

Displacement demand of the system is according to analysis results are considered and evaluated according to the displacement limits presented in seismic codes (IBC, 2006). It is concluded that displacement demands are below the critical limits.

Table 3.7. Displacement limits (IBC, 2006).

Piping Flexibility			
For the self anchored tanks, The Piping systems connected to tanks should provide sufficient flexibility to avoid release of product by failure of the piping system for the following displacement.			
Upward Vertical Displacement: For Anchorage ratio less than or equal to 0.785, Upward vertical displacement relative to support or foundation	=	23	mm
Downward Vertical Displacement: Downward vertical displacement relative to support or foundation for tanks with a ring wall	=	13	mm
Radial Displacement: Range of horizontal displacement (radial and tangential) relative to support or foundation	=	50	mm

Base moment and shear force response profiles are collected and shown visually as below:

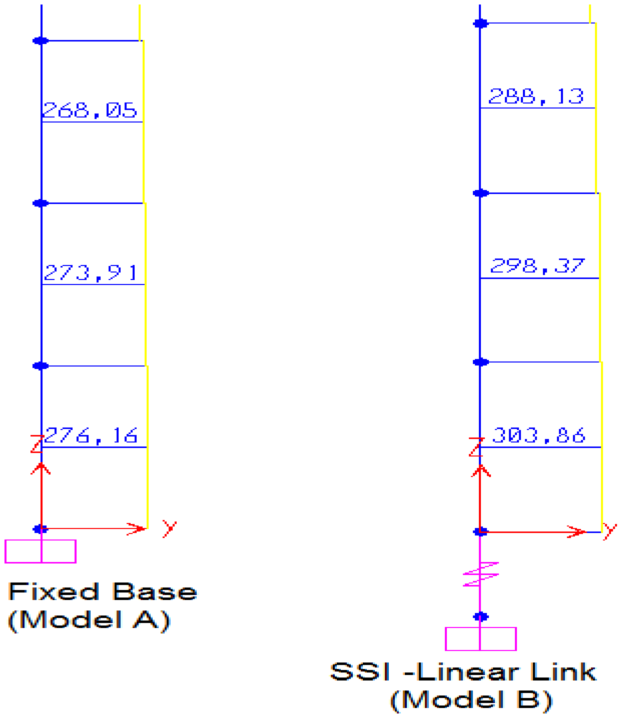


Figure 3.14. Base shear forces.

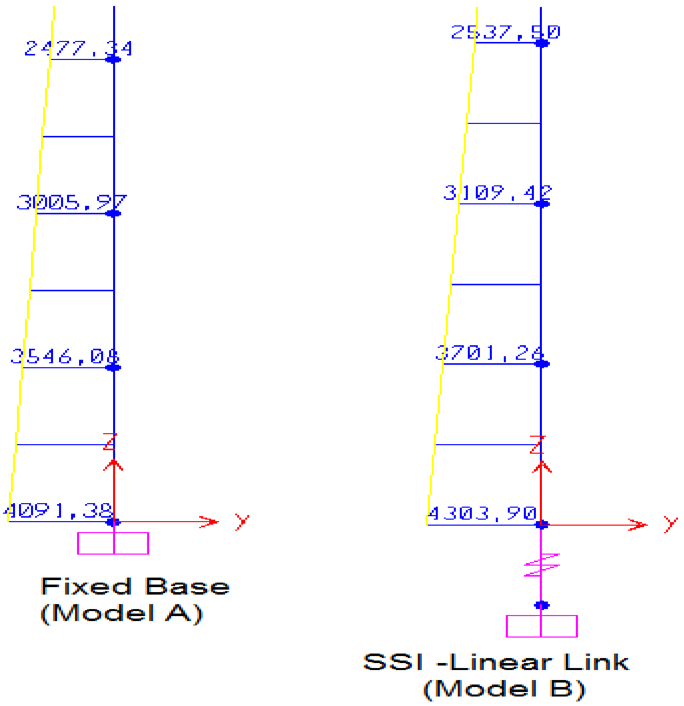


Figure 3.15. Base moment.

3.3.3. Uplift Check:

In order to calculate the overturning moment acting on the foundation, the critical axis about which the foundation is rotating should be selected. In our study it is the global x-axis. It is assumed that the rotation point will be the corner-point of the foundation and as the soil becomes softer, this point will get closer to the centre. If the overturning moment is greater than the resisting moment, foundation uplift would occur.

<u>The resisting moment of the tank-foundation system as per their dead weight;</u>
(4 x 100 x 10) x 2.5 =10,000 kN.m (4 tanks)
(17 x 5 x 1 x 2.4 x 10) x 2.5=5,000 kN.m (foundation)
<u>Total: 15,000 kN.m</u> (resisting moment)
<u>Overturning moment for one tank: 4.300 kN.m (from analysis)</u>
*4,300 kN.m ≤ 15,000 kNm **-OK-***
*8,600 kNm (for 4 tanks combined with SRSS) ≤ 15,000 kN.m **-OK-***
Foundation uplift will not occur for these cases.

However for the extreme case of all 4 the tanks vibrating in the same phase;
*Total Overturning Moment: 4 x 4,300 **= 17,200** kN.m > 15,000 kN.m **-UPLIFT-***

For the case of all the 4 tanks vibrating in the same phase, foundation uplift would occur. Foundation uplift is represented by a non-linear elastic link member discussed in further chapters. Non-linear properties of the member represent the loss of stiffness, due to the interaction surface becoming smaller caused by the foundation uplift.

3.3.4. Evaluation of Response Spectrum Analyses

As a result of SSI, period of the structure and equivalent damping increases. Therefore it is expected that base shear forces be reduced for the system where SSI is considered relative to the conventional system supported by a fixed support. Generally this is the real case when the vibration period of the system supported by a fixed base corresponds to the horizontal plateau of the response spectrum. So with the increase in period when SSI is considered, the period of the system will either still be in the plateau or in the decreasing part of the response spectrum. With the combined effect of the increase in the damping and the period, the force response of the system is expected to decrease. Therefore SSI is assumed to be beneficial for the structure and it is disregarded in order to stay on the safer side as far as design is concerned.

However for the case considered in this study, the structure is very rigid. And the fixed base vibration period of the structure corresponds to the first part of the response spectrum (linearly increasing part). Therefore when the SSI is considered; spectral acceleration corresponding to the increased period of the system corresponds to the horizontal plateau of the spectrum. This results an increase in the base shear response of the system due to SSI. On the other hand, with the increase in damping the base shear response will be decreased. Here it should be remembered that damping ratio of the system is taken 2% for rigid base and 5% for SSI interaction models.

To sum it up, in this study SSI effect on the response of the structure is negative due to increase in period but positive due to increase in damping. The total effect of SSI is the combination of both effects. As it can be seen visually at Figures 3.12 and 3.13 the shear force and bending moment response of the system is increased about 10% and 5% respectively, relative to fixed base system when SSI effects are considered. Displacement demands are also increased compared to fixed base system.

Two main structural safety checks are conducted.

First the displacement control at pipe connection points is conducted. Although the displacements are increased with the effect of SSI, the displacements are observed to be smaller than the safe limits. Therefore it is concluded that the HPV system will not undergo any damage due to relative displacements.

Secondly overturning check is conducted. Overturning moment calculated via dynamic response spectrum analysis is compared with the resisting moment caused by the dead weight of the structure. Results are evaluated and it is concluded that for the extreme case of all the 4 tanks vibrating in the same phase, foundation uplift would occur.

3.3.5. Practical Solution Offers

It is suggested that two separate mat foundations examined in this study can be designed at one big mat foundation. On the other hand the catwalks on the top should as weak as possible in order for them to prevent working together structurally with the tanks.

Similar pressure vessel units exist in the important oil pipe line units in our country. Considering the active seismicity of our country, the results of soil structure interaction may be severely devastating. Therefore behaviour and response of different superstructure, foundation and soil combinations should further be researched under the effects of different earthquake excitations.

3.4. Time History Analyses

In this section, four different earthquake records are used and soil-structure interaction model is developed further by defining non-linear plastic link member. It is modelled in two degrees of freedom (rotational and translational) as force-displacement and moment rotation and according to these models; various non-linear time-history analyses are conducted and the results are evaluated.

3.4.1. Earthquake Records

As earthquake records; unmodified horizontal components of;

1. El-Centro (with only increased amplitudes)
2. Yarımca (near-field earthquake record)

And horizontal components of;

3. Düzce (to be spectrum matched)
4. Westmorland (to be spectrum matched)

The latter two earthquakes are modified to be compatible with the spectrum used in the previous response spectrum analysis cases.

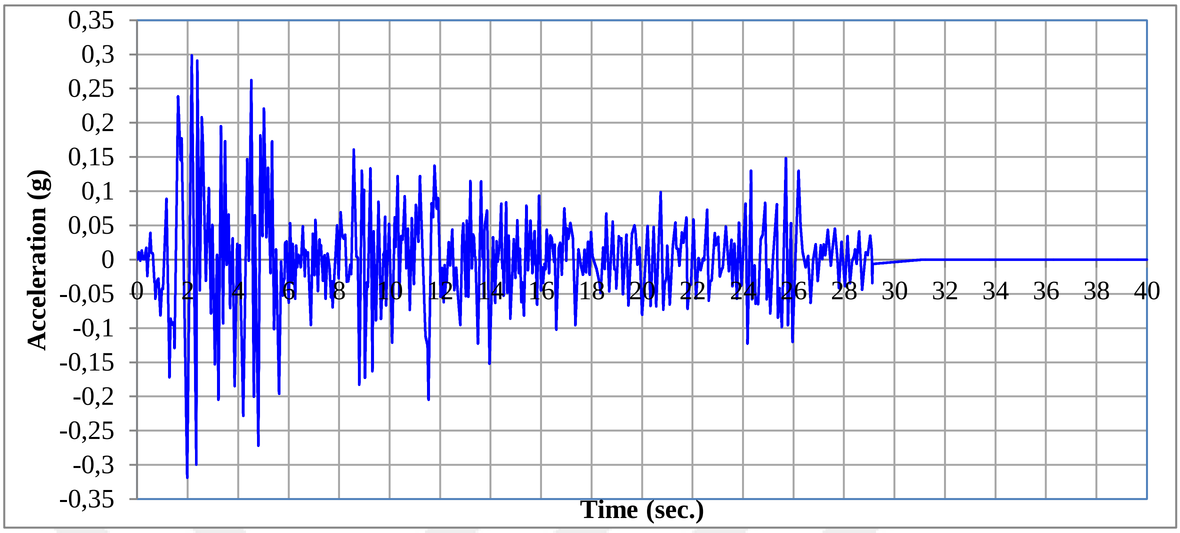


Figure 3.16. El-Centro earthquake (horizontal, scaled).

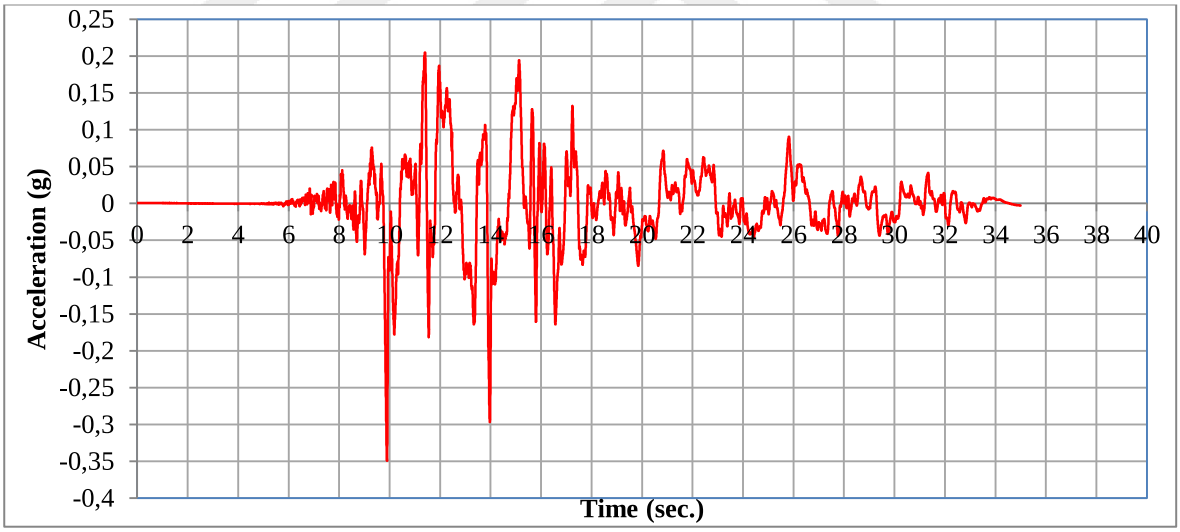


Figure 3.17. Yarımca earthquake (horizontal).

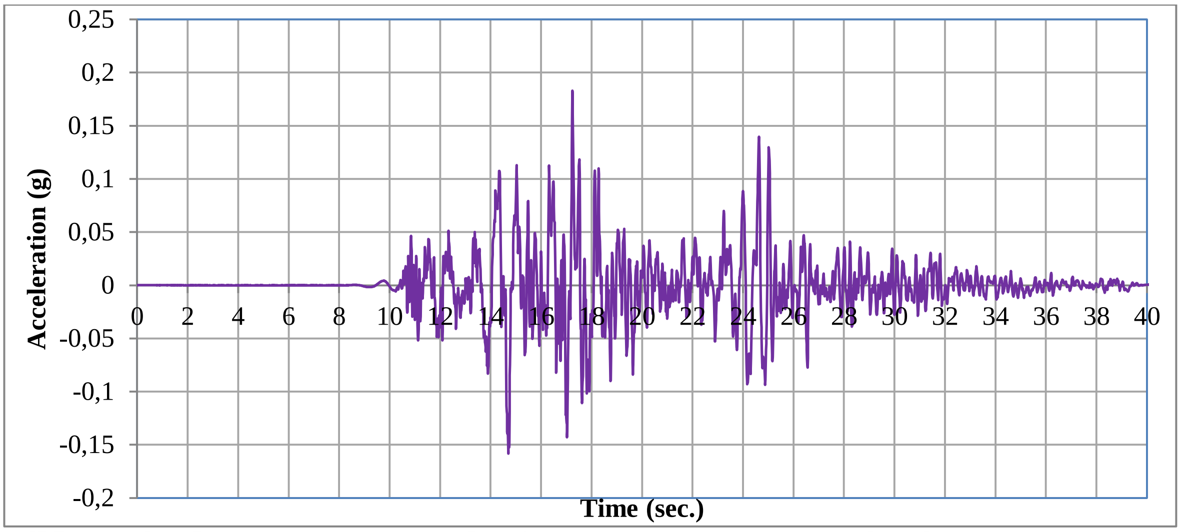


Figure 3.18. Düzce earthquake (horizontal, spectrum matched).

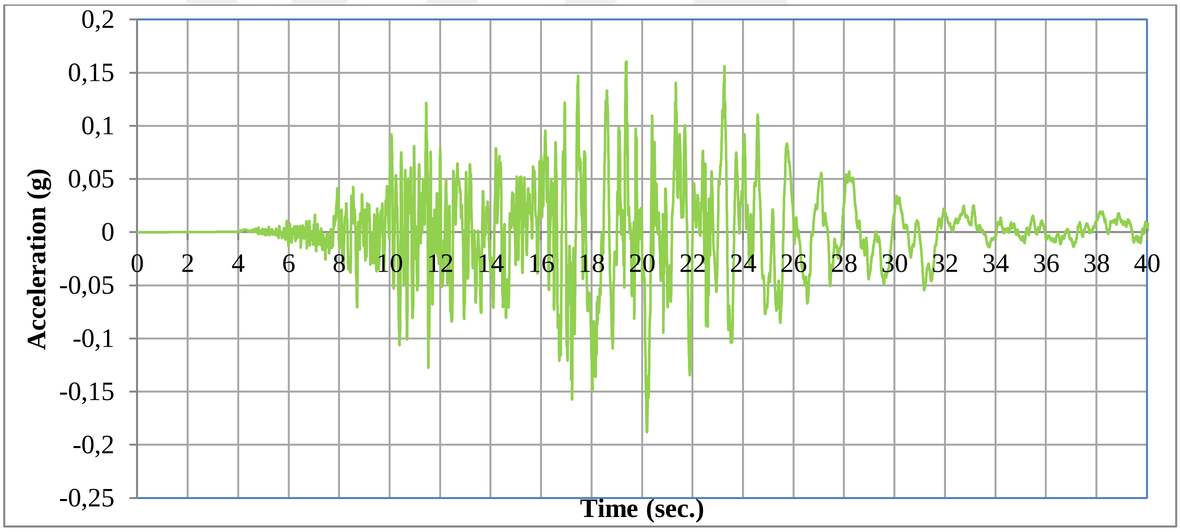


Figure 3.19. Westmorland earthquake (horizontal, spectrum matched).

3.4.2. Earthquake Response Spectra

As mentioned in the previous section, acceleration time histories of Düzce and Westmorland earthquakes are modified so that their response spectra can match the response spectrum used in the response spectral analysis case. The software “SeismoMatch” is used to modify these two earthquake time histories. Response spectra of all earthquakes and the comparison of response spectra of the matched records and IBC response spectrum are plotted.

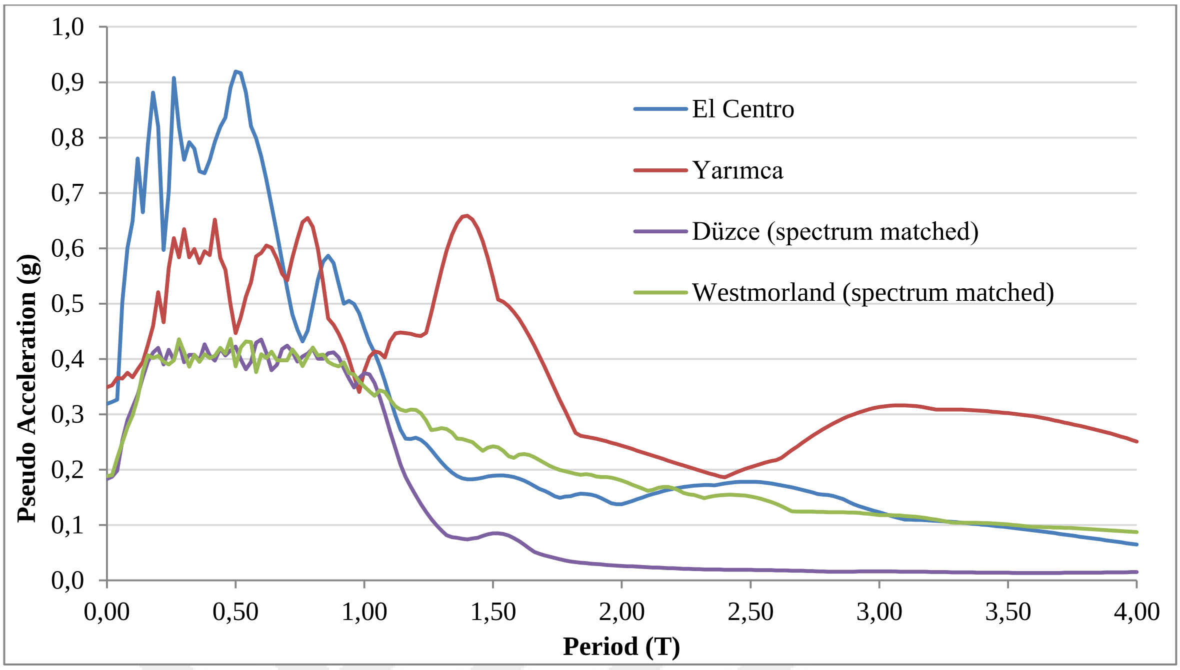


Figure 3.20. Response spectra of the earthquakes.

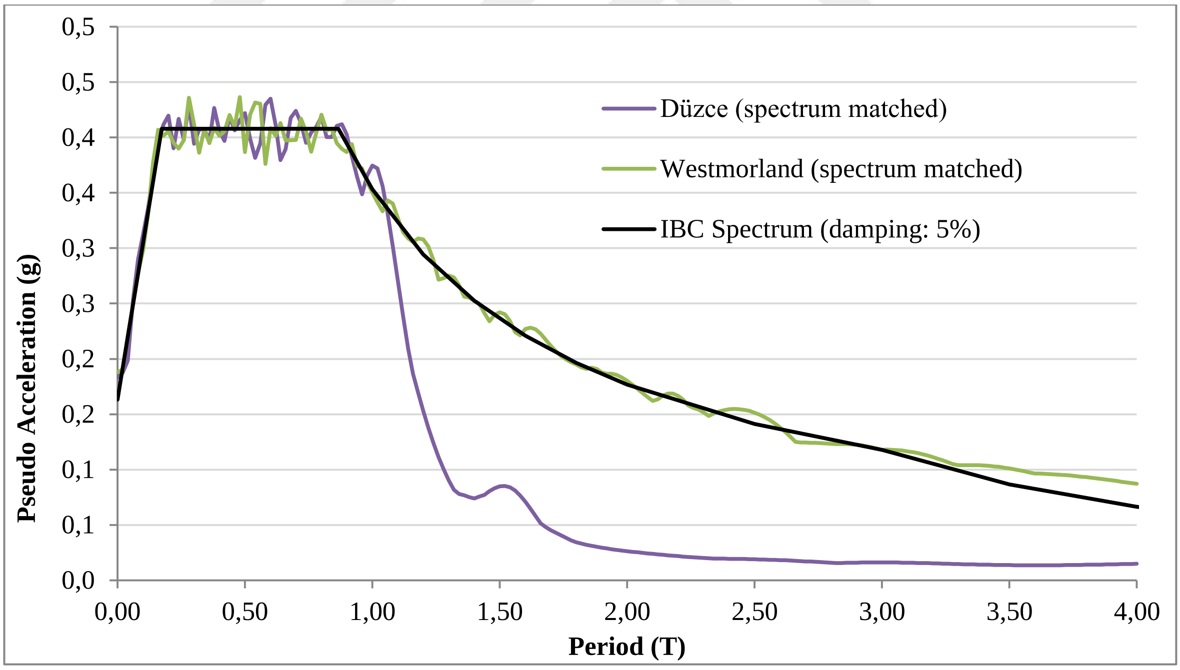


Figure 3.21. Response spectra of the spectrum matched earthquakes.

3.4.3. Developing of "Model C" (NL-Plastic Link)

Linear-elastic link members (L-Link) which is used in spectral analysis (model B) is modified as non-linear and idealised as bilinear plastic link members (NL-Link). Stiffness values of link member are assigned such that they are compatible with the values used in the linear model. Slope of the moment-rotation model for NL-Link corresponds to stiffness values for linear link. After a certain yield point, a second arbitrary slope (stiffness) is assigned. Thus the bilinear NL-Link model is defined.

Aside from the previously discussed degrees of freedom, other degrees of freedom are fixed. Changing the yield point but keeping the slopes unchanged which belong to the rotational degree of freedom (i.e. yield moment, yield rotation); six different analyses cases are created. Threshold yield force for force-displacement model of translational stiffness of NL-Link is kept extremely high such that non-linearity only occurs due to rotation.

The responses of the soil structure interaction surface and the superstructure are observed and evaluated. Translational degrees of freedom for these six cases are kept unchanged for the sake of simplicity and ease in comparison.

Bilinear Moment-Rotation models of the 6 cases belonging to the NL-Link mentioned in the previous paragraph are shown in Figure 3.20 below:

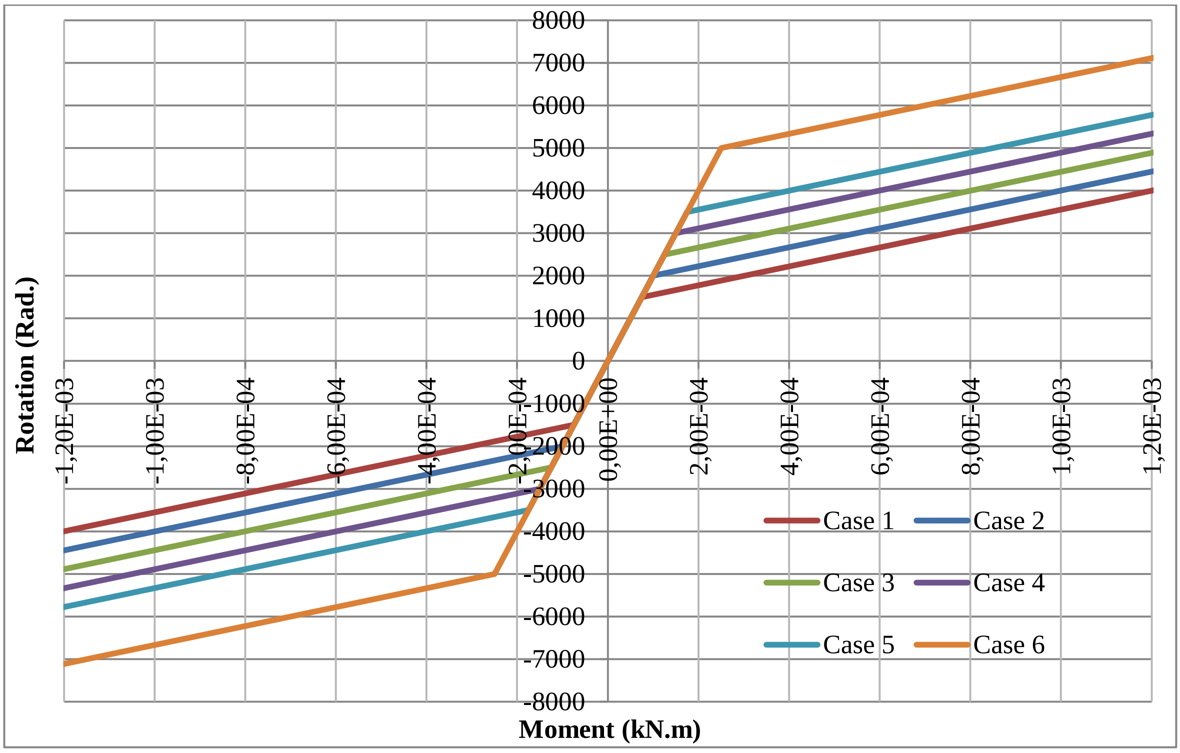


Figure 3.22. Moment-rotation model (stiffness) for SSI for different yield points.

3.4.4. Moment-Rotation Hysteresis Curves

For the six cases shown graphically above, non-linear time history analyses are conducted. For all these 6 different yield levels and 4 different earthquakes; Moment-Rotation responses at base (non-linear link member) are plotted as shown below:

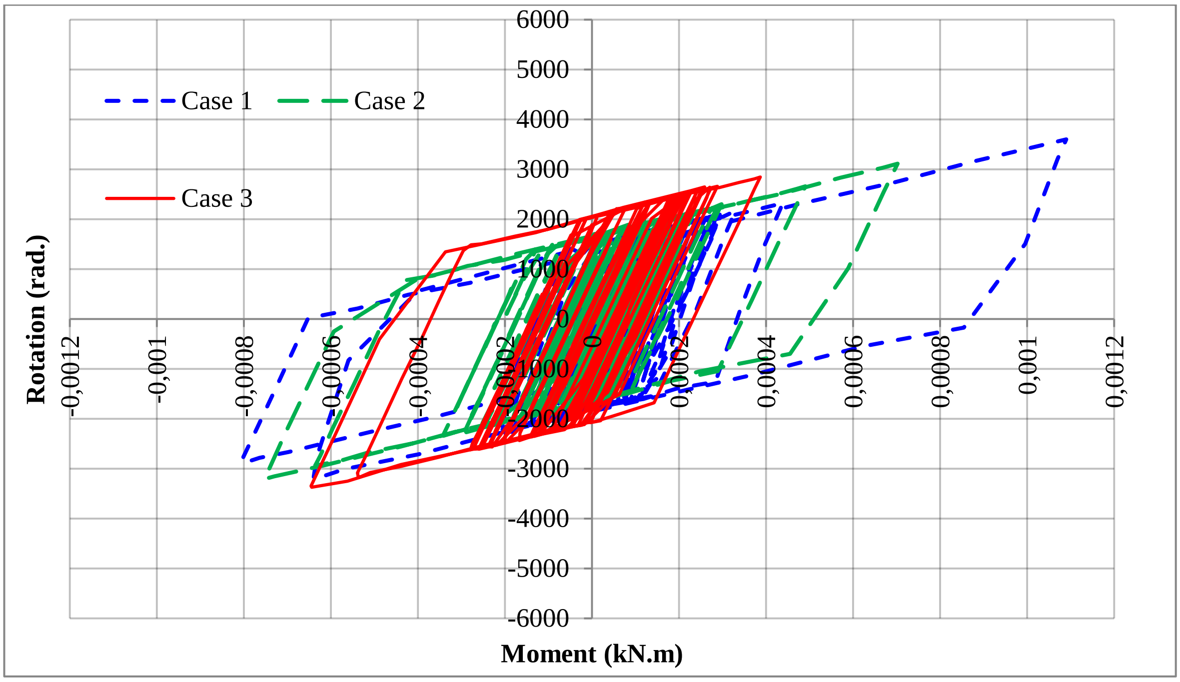


Figure 3.23. Moment-rotation hysteresis response at base, El Centro (case 1-3).

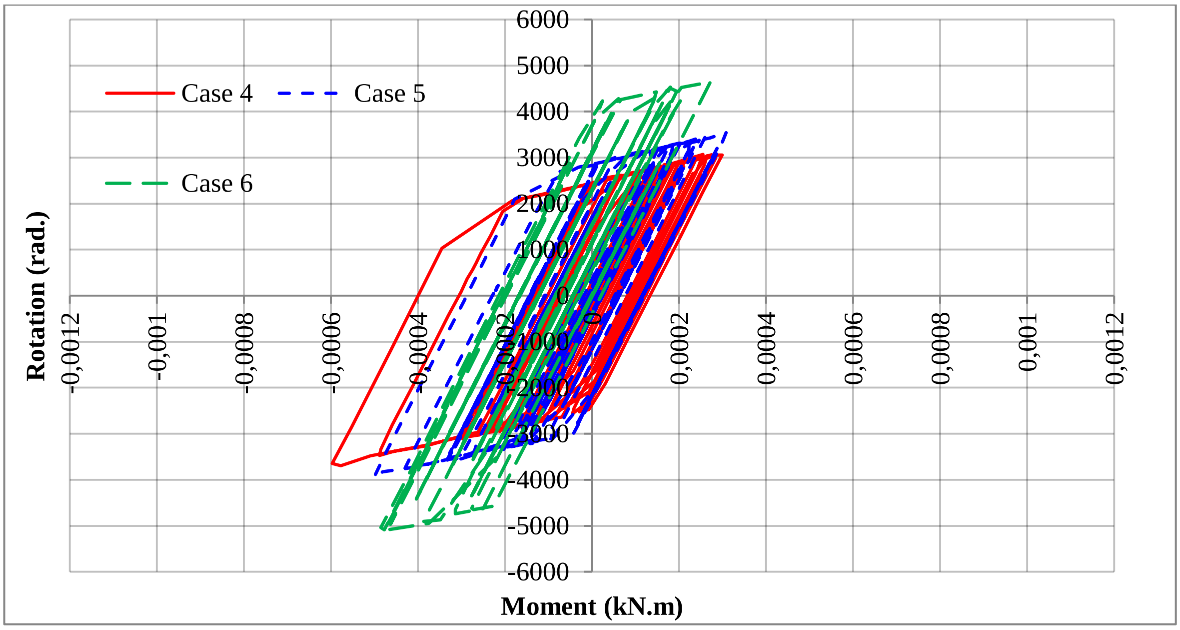


Figure 3.24. Moment-rotation hysteresis response at base, El Centro (case 4-6).

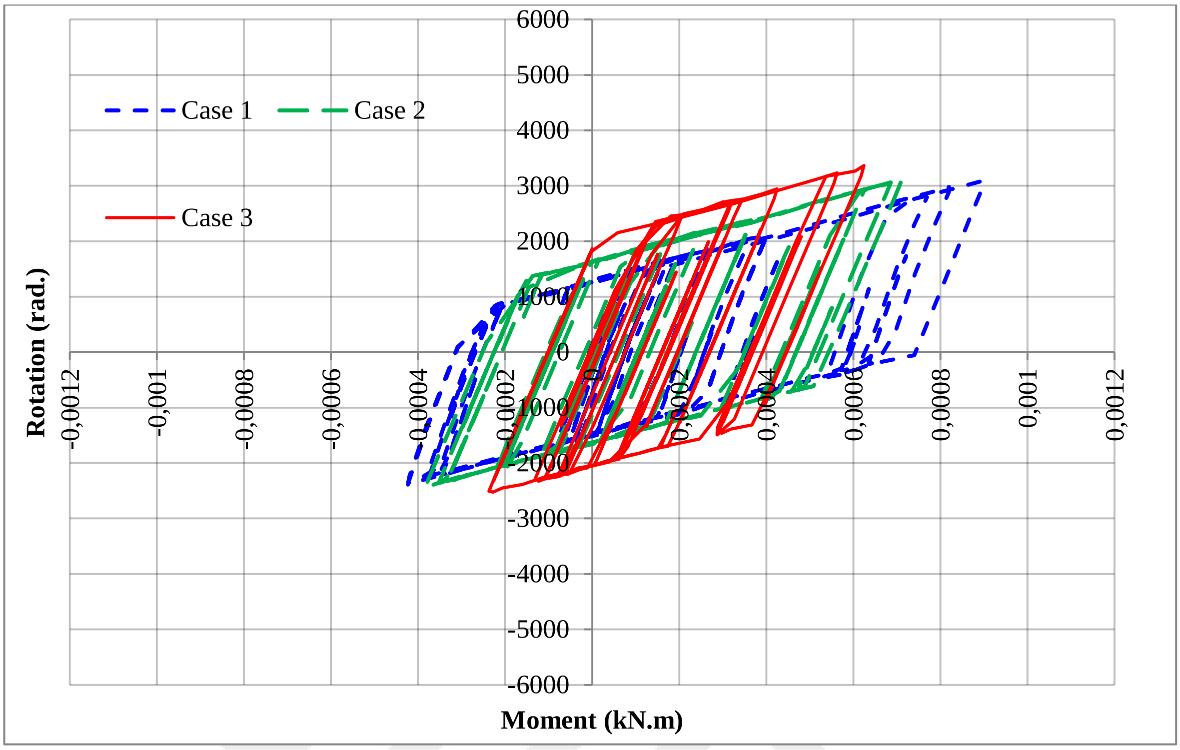


Figure 3.25. Moment-rotation hysteresis response at base, Yarımca (case 1-3).

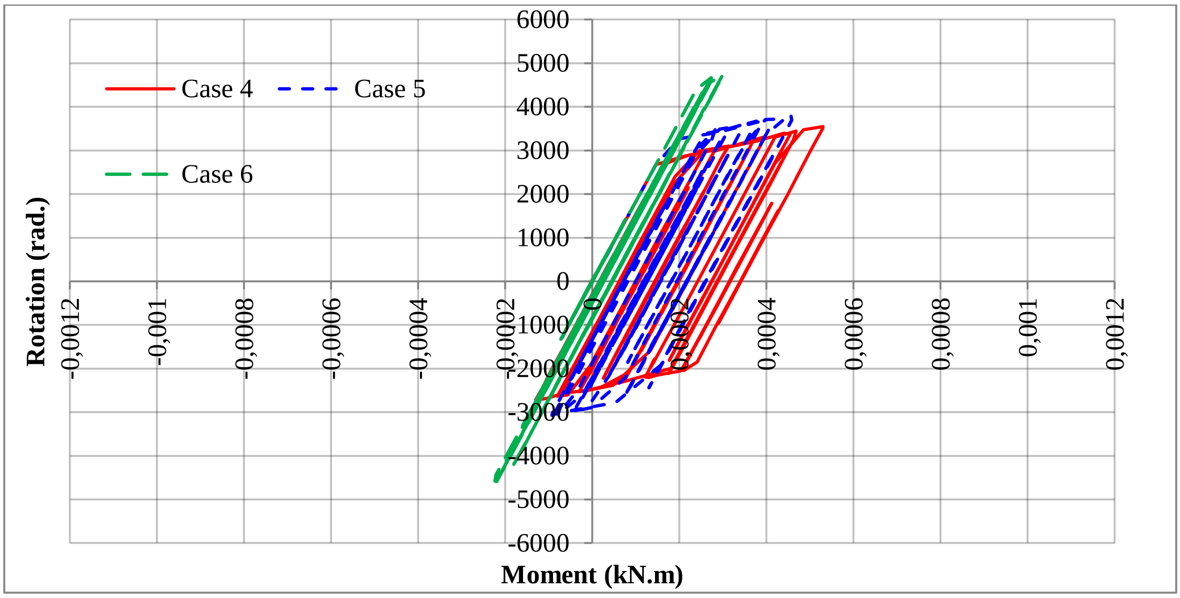


Figure 3.26. Moment-rotation hysteresis response at base, Yarımca (case 4-6).

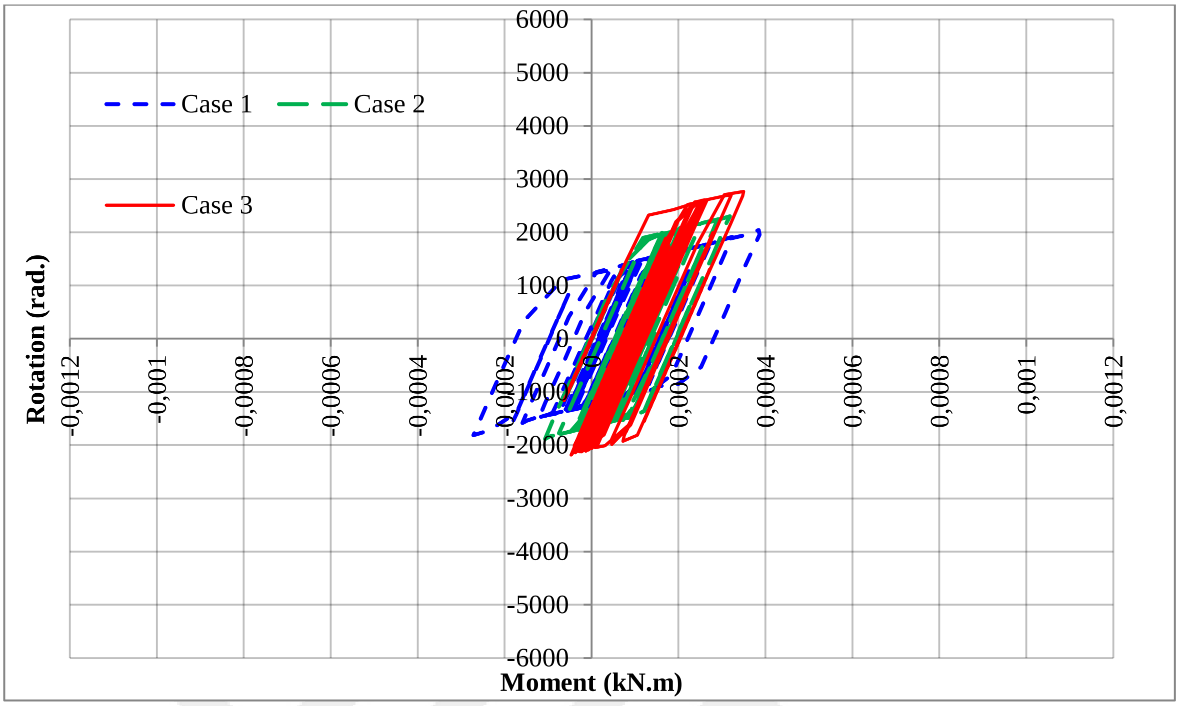


Figure 3.27. Moment-rotation hysteresis response at base, Düzce (case 1-3).

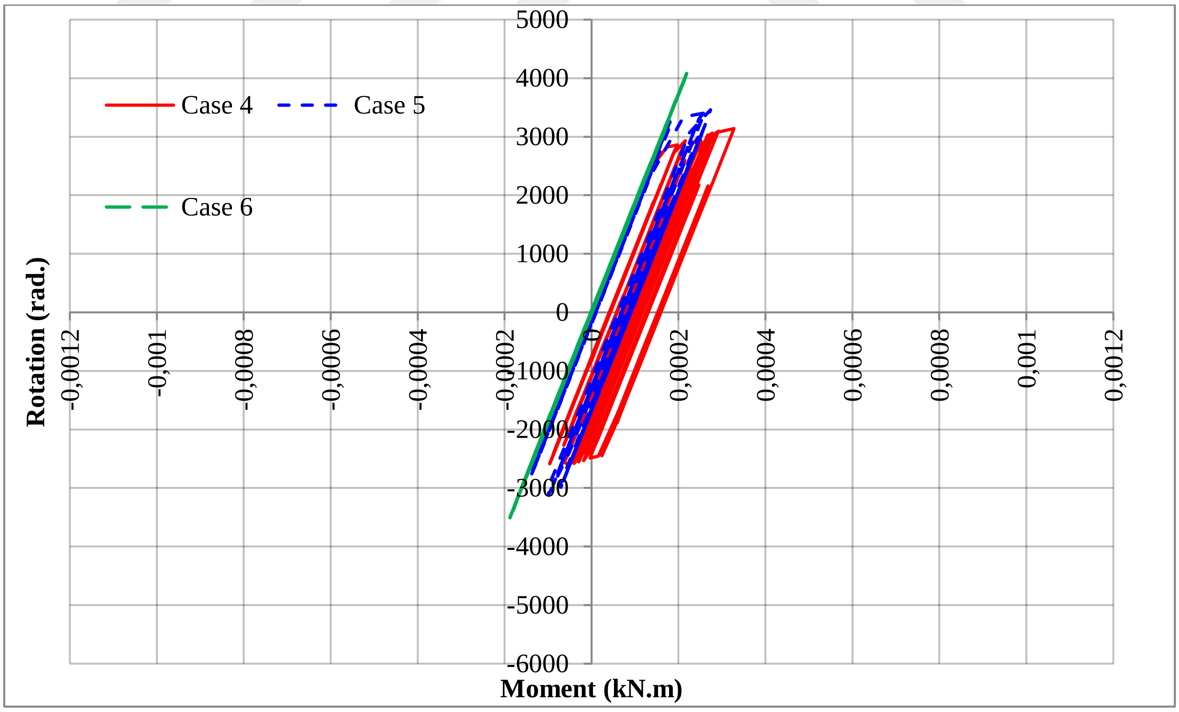


Figure 3.28. Moment-rotation hysteresis response at base, Düzce (case 4-6).

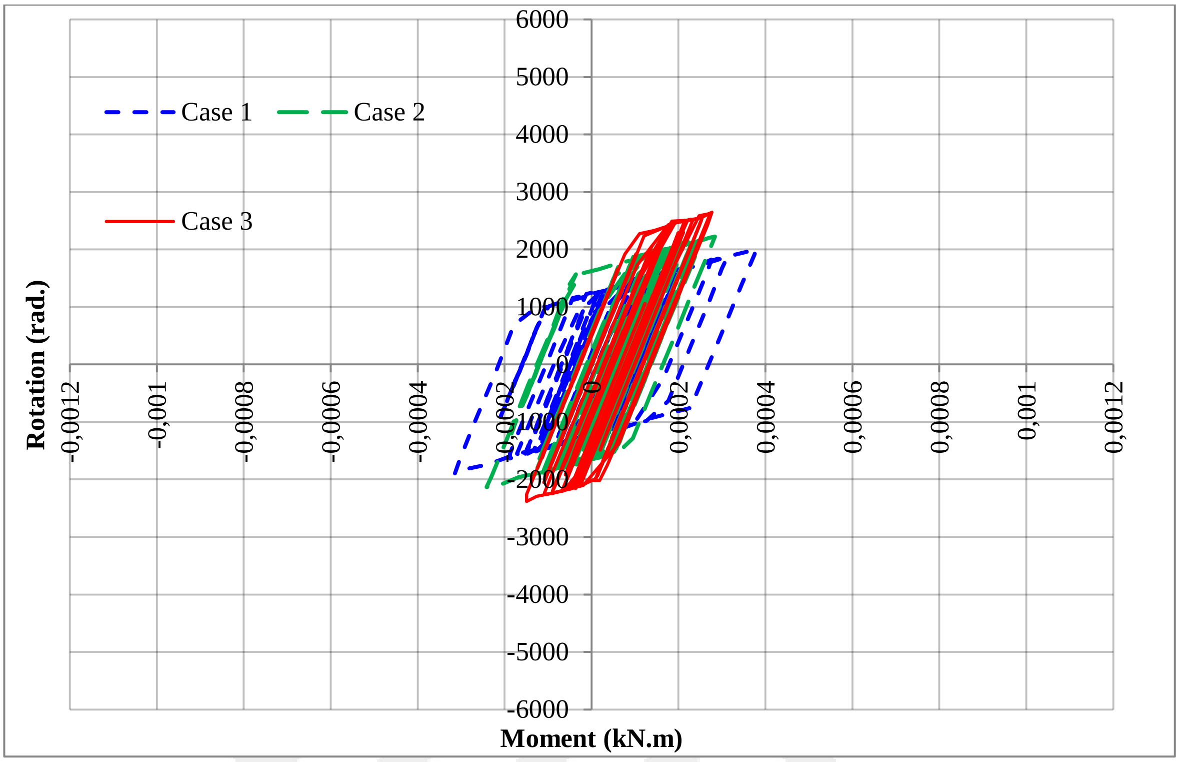


Figure 3.29. Moment-rotation hysteresis response at base, Westmorland (case 1-3).

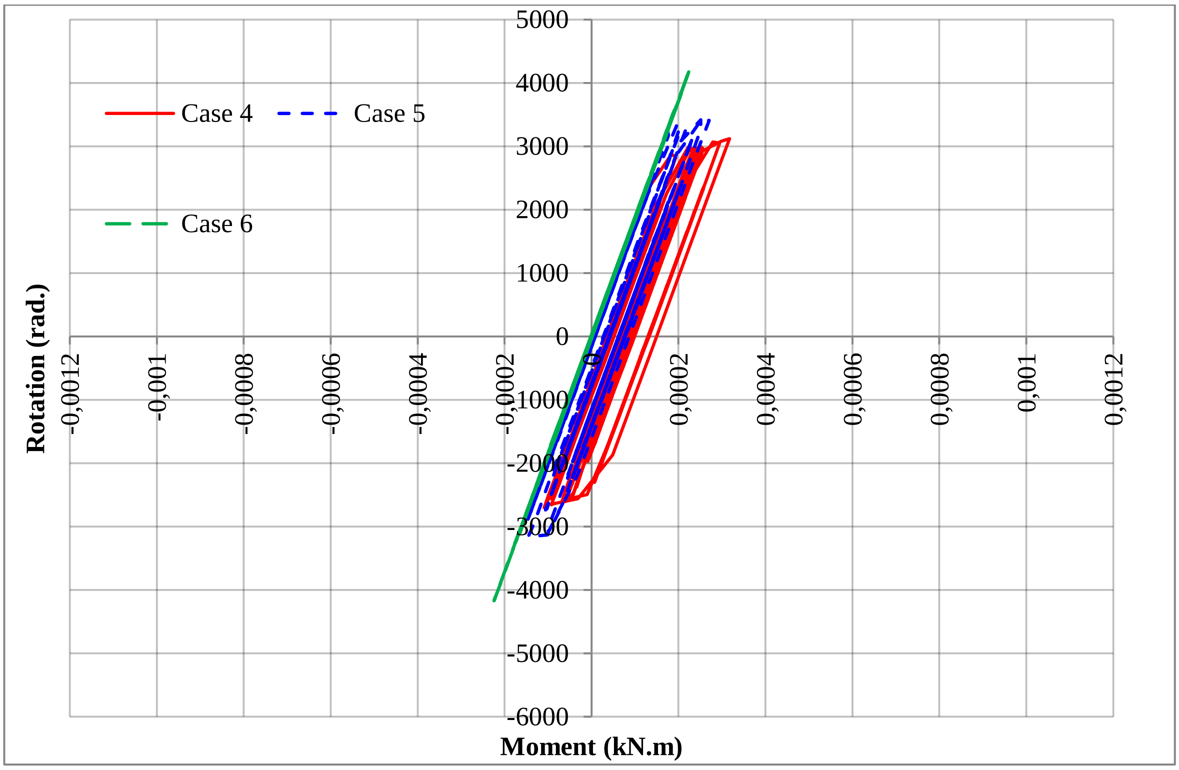


Figure 3.30. Moment-rotation hysteresis response at base, Westmorland (case 4-6).

When the responses presented above are observed as the yield level is changed the results also changes even though the stiffness values (slope of Moment-Rotation link model) are not changed. As the yield level is decreased the response of link member changes from linear to non-linear and the deformations also increases respectively. Similarly as the yield level is increased the behaviour of the member gets closer to a linear elastic behaviour and the deformations also decreases.

It is important to note that different yield levels represent different levels of plasticity of the soil. So while low yield levels (Cases 1 to 3) represents high levels of plasticity of a relatively softer soil, while higher yield levels (Cases 4 to 6) represents relatively harder soils.

In the graphs below for high level, medium level and low level of plasticity (i.e. Cases 1, 4 and 6 respectively) hysteresis moment-rotation response of the structure at base to each earthquake is plotted and evaluated.

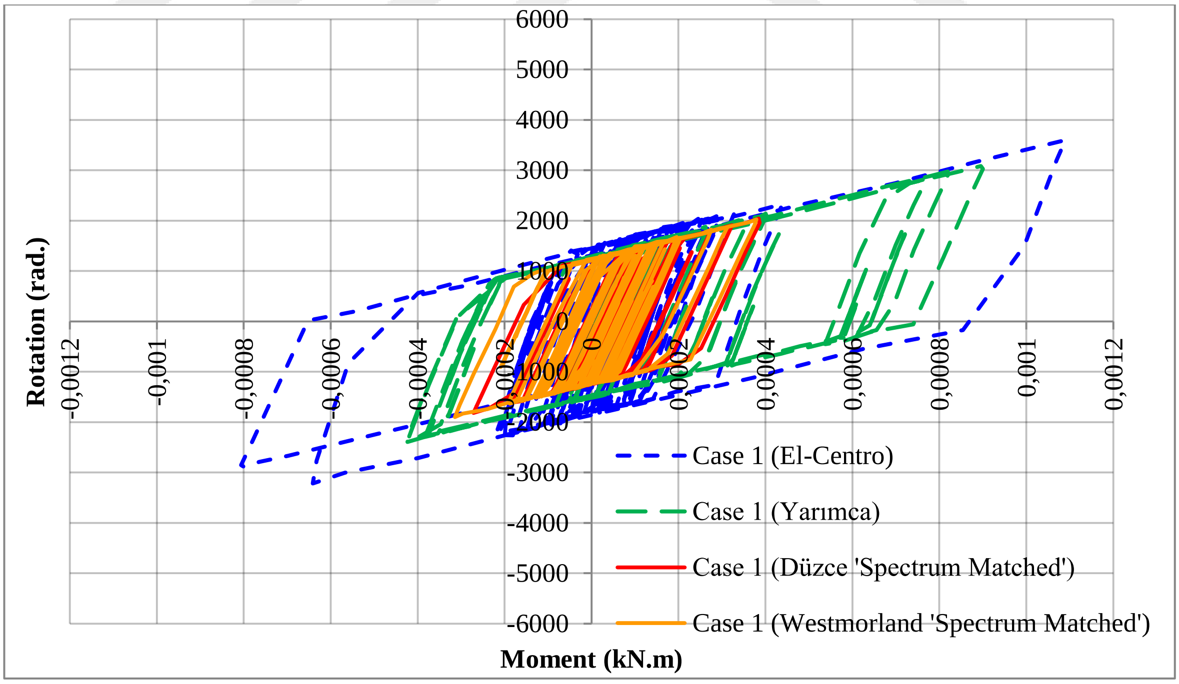


Figure 3.31. Moment-rotation hysteresis response at base: highest level of plasticity (case 1) (for all earthquakes).

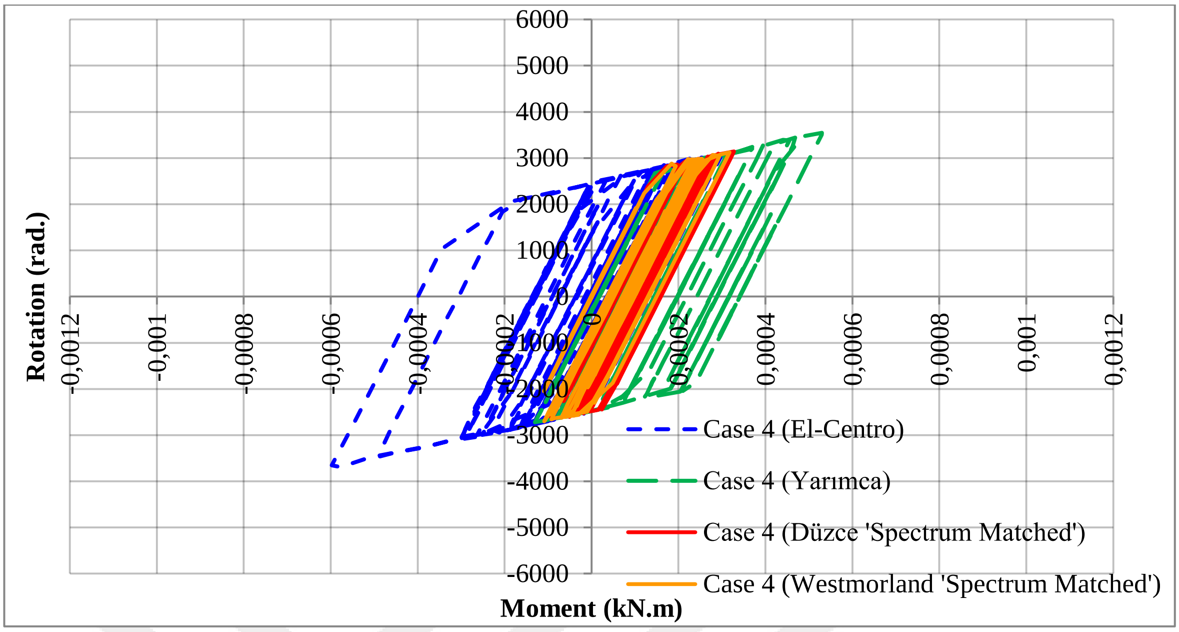


Figure 3.32. Moment-rotation hysteresis response at base: medium level of plasticity (case 4) (for all earthquakes).

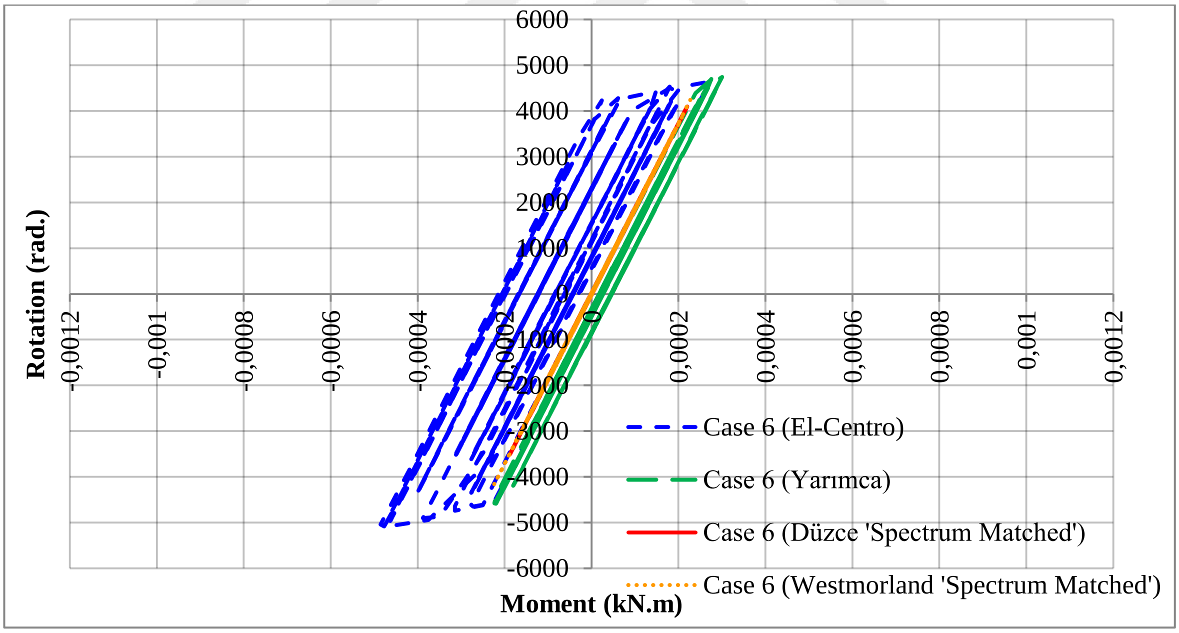


Figure 3.33. Moment-rotation hysteresis response at base: lowest level of plasticity (case 6) (for all earthquakes).

When the responses shown above are examined, it is observed that the responses of spectrum matched earthquake records (Westmorland and Düzce) are quite similar to each other.

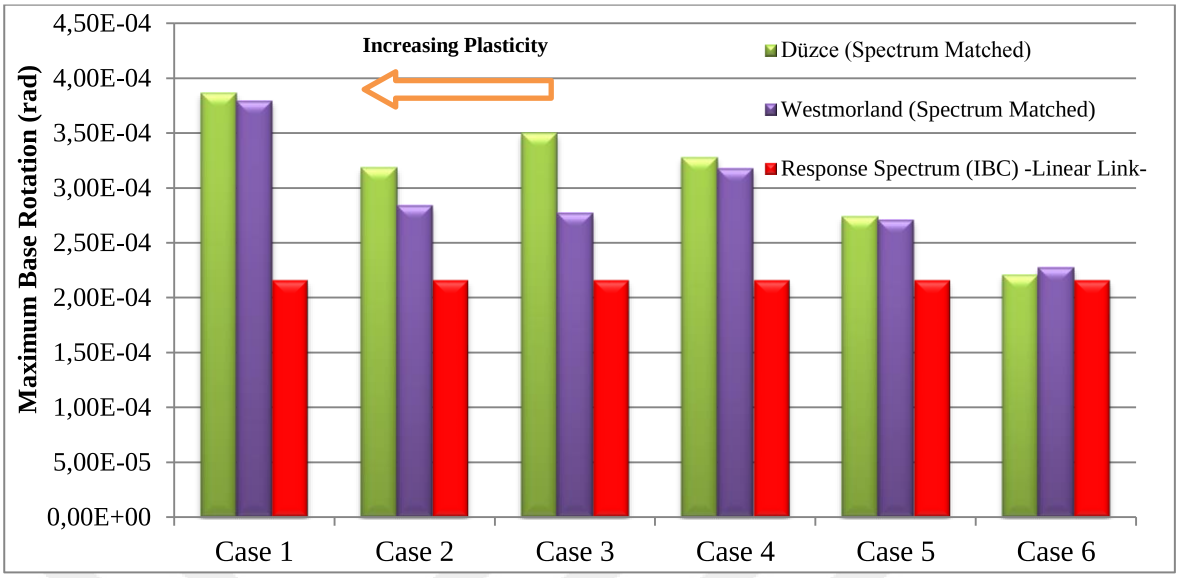


Figure 3.34. Maximum base rotations for all levels of plasticity.
(Spectrum matched earthquakes and response spectrum analyses)

The analysis results of non-linear time history cases (using spectrum matched earthquakes) and response spectrum analysis cases where the same spring stiffness values are assigned for linear link member are compared. When the results are evaluated it is observed that the responses of high plasticity systems are greater than the response of response spectrum analysis case. However it is important to note that the results of the least plastic cases are similar to the results of the response spectrum analysis case.

3.4.5. Total Acceleration Response at Top

For each of the 4 considered earthquakes and all 6 different levels of plasticity cases total acceleration responses at the top of the structure are calculated and tabulated in the graph shown below.

It should be noted that although the response of all earthquakes are presented, numerical comparison is only sensible between the responses of the modified spectrum matched earthquakes. However all the earthquake responses are important as far as observing the change in response of the structure for different levels of soil plasticity is concerned.

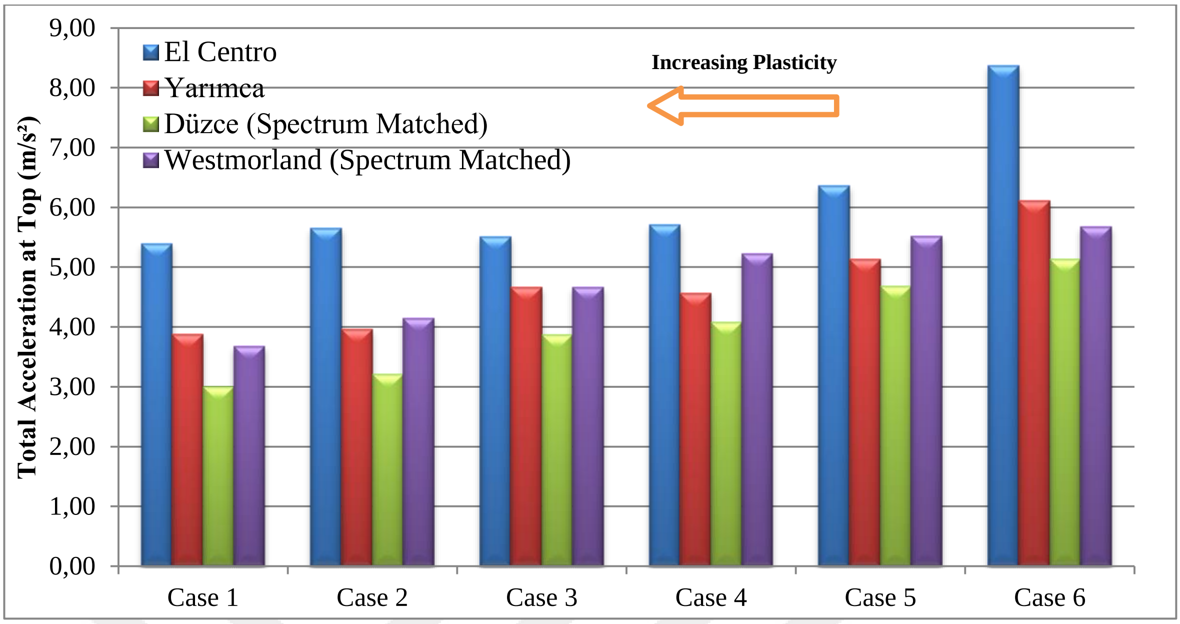


Figure 3.35. Total acceleration response at top.

It can be observed that as the level of plasticity of base is decreased (in other words as the yield level of the link member is increased) the acceleration response of the system increases for all cases.

3.4.6. Relative Displacement at Top

For the non-linear time history analyses the maximum relative displacement values at top of the structure are calculated and tabulated in the graph shown below:

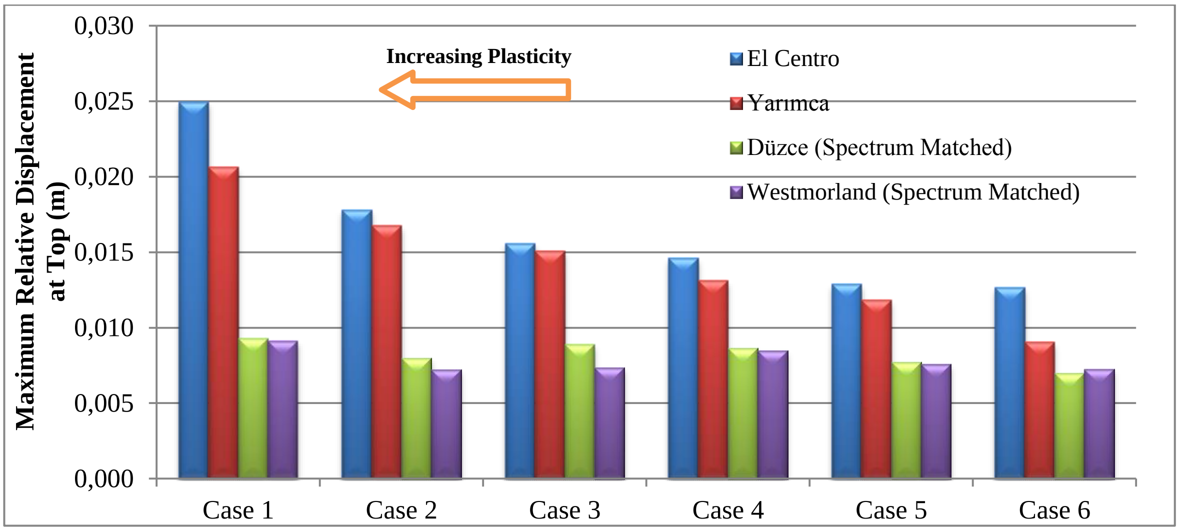


Figure 3.36. Maximum relative displacement at top.

For the cases where unmodified El Centro and Yarımca earthquakes are used, it can be observed that the displacement response of the structure increases as the plasticity level of the link member is increased. However for the cases where modified "spectrum matched" Düzce and Westmorland earthquake cases are used, the results are observed as similar for all levels of plasticity and no increasing or decreasing trend is observed.

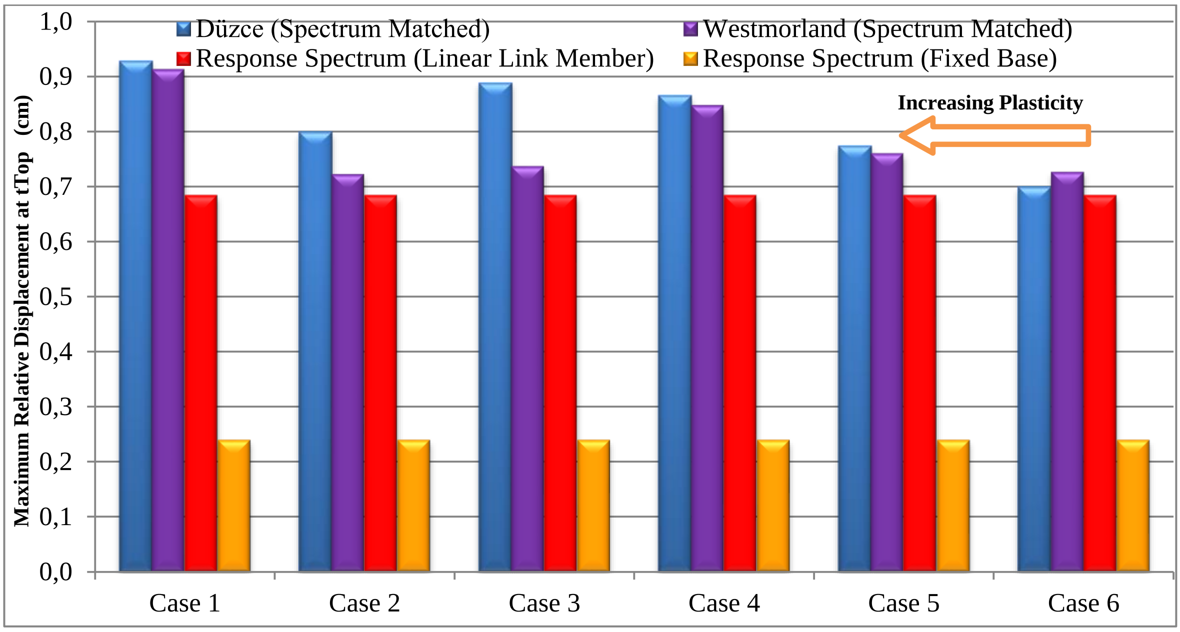


Figure 3.37. Maximum relative displacement at top.

3.4.7. Base Shear Response

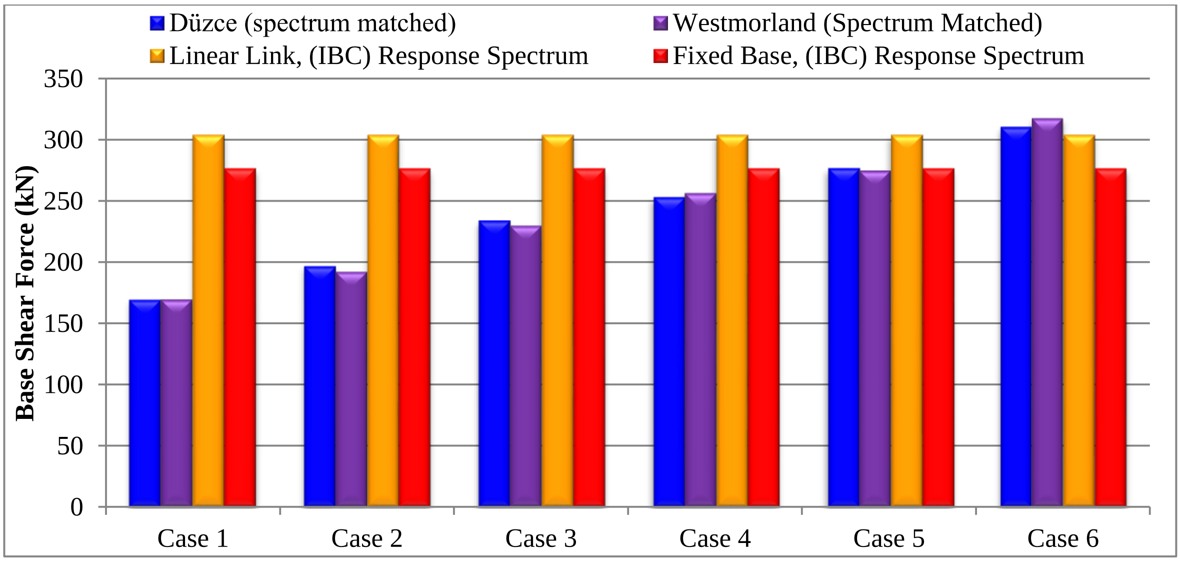


Figure 3.38. Base shear force response.

When the analysis results tabulated above are evaluated it is observed that as the level of plasticity decreases (when yield level of the link member is increased) base shear force responses also increases. For the cases with higher levels of plasticity, the results are significantly lower than the response spectrum analysis case. As the plasticity is decreased the system behaviour tends to become closer to linear and as a result, the maximum base shear response of the structure is observed similar to the response of response spectrum analysis case.

3.4.8. Base Moment Response

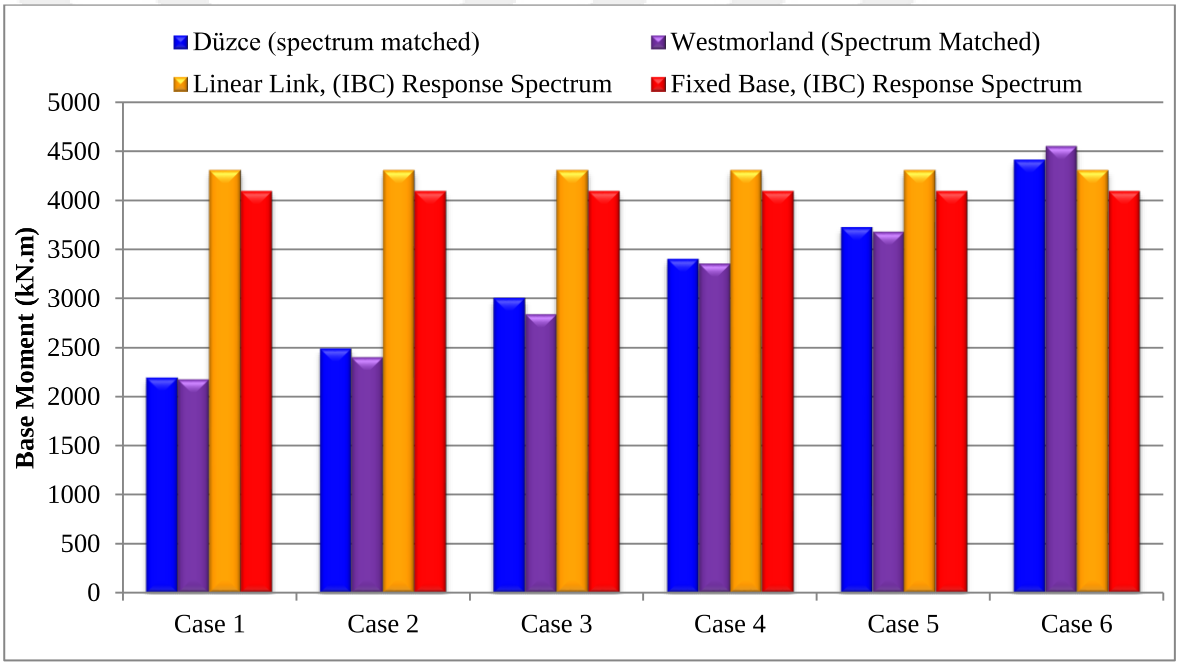


Figure 3.39. Base moment response.

As it can be observed from the graph above the base moment response follows a similar trend to base shear force response. As the level of plasticity is decreased, the base moment response increases.

It is also really important to note that with high levels of plasticity (corresponding to softer soils) with the increased plastic behaviour of soil base responses are decreased significantly compared to harder soils. Although this may seem beneficial for the structure, very high levels of plasticity may lead to excessive deformations and uplift on the structure which will be examined in the next sections.

3.4.9. Effects of Foundation Uplift

If foundation uplift occurs as the effective contact area of foundation and soil decreases overall stiffness due to SSI will also decrease. As a result of this soil may exhibit plastic behaviour. And this situation may result in excessive deformations in the connection (tank-pipe) points of non-building -tank type- of structures.

In this last part of the study in addition to already defined non-linear plastic link member, the model is further developed by defining an additional non-linear elastic link member in order to be able to take uplift effects into account. This link member is connected in series to already modelled plastic link member. As our primary focus being examining the near field effects, another elastic link member is not defined. Eventually; only the effects of plasticity and uplift are considered. Symbolic representation of the model can be related to the macro element model proposed by "Cramer at al. 2002":

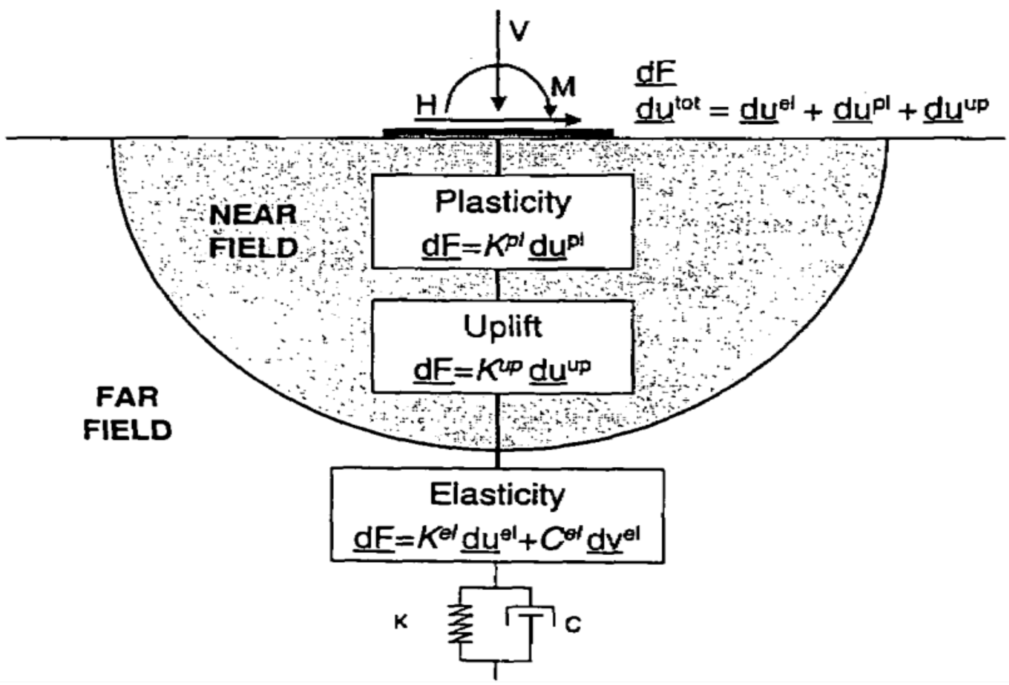


Figure 3.40. Macro-element model (Cremer at. al. 2002).

Since the springs (link members) are connected in series, equivalent spring stiffness is decreased and therefore the period of the structure is increased. This coupled behaviour is represented by two springs connected in series.

First spring represents the non-linear plastic behaviour. This is a representation of the reduction of stiffness as the soil exhibits plastic behaviour. Second spring represents the uplift of the foundation (non-linear elastic behaviour). Under the effects of a dynamic loading such as earthquake excitations, it is assumed the first member will exhibit plastic (hysteretic) behaviour and second member will exhibit elastic behaviour.

Below is the representation of the proposed macro-element by Cremer at. al. 2002.

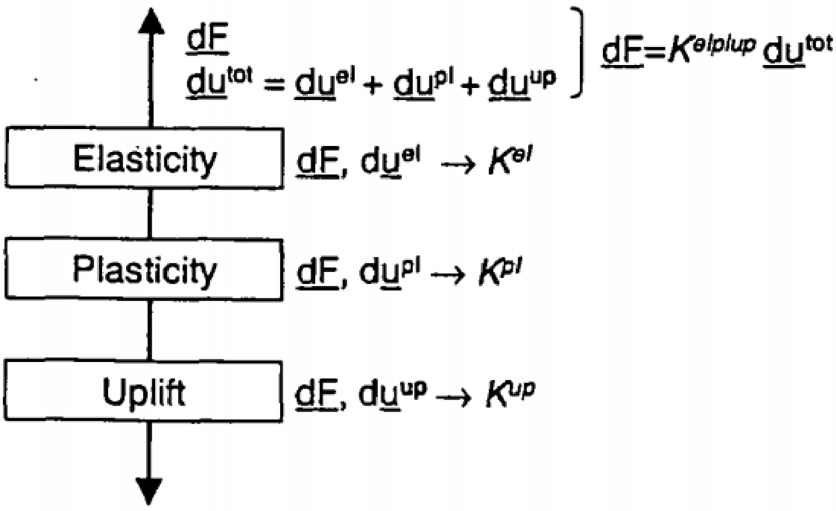


Figure 3.41. Structure of the global model (Cremer at. al. 2002).

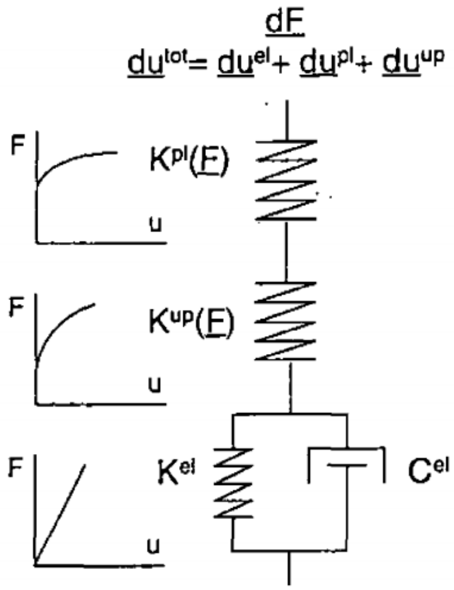


Figure 3.42. Analogical system (Cremer at. al. 2002).

3.4.10. Developing of "Model D" (NL-Plastic + NL-Elastic Link)

The link models defined for the analyses are shown graphically as below (non-linear plastic and non-linear elastic models). An arbitrary threshold moment value is defined for uplift (non-linear elastic link) and a yield moment is defined for plastic behaviour of soil (non-linear plastic link). To define realistic values for these parameters is very difficult. Therefore generic values are used with the motivation and focus on the changes in the response of the structure and evaluating the different results. However it can be interpreted that higher moment thresholds to represent wide foundations resting on stiff soil, where lower threshold moments representing soils of clay with high water content.

Different yield levels of non-linear plastic member are assumed to represent different soil plasticity indexes. As the yield moment decreases plastic displacement demand increases.

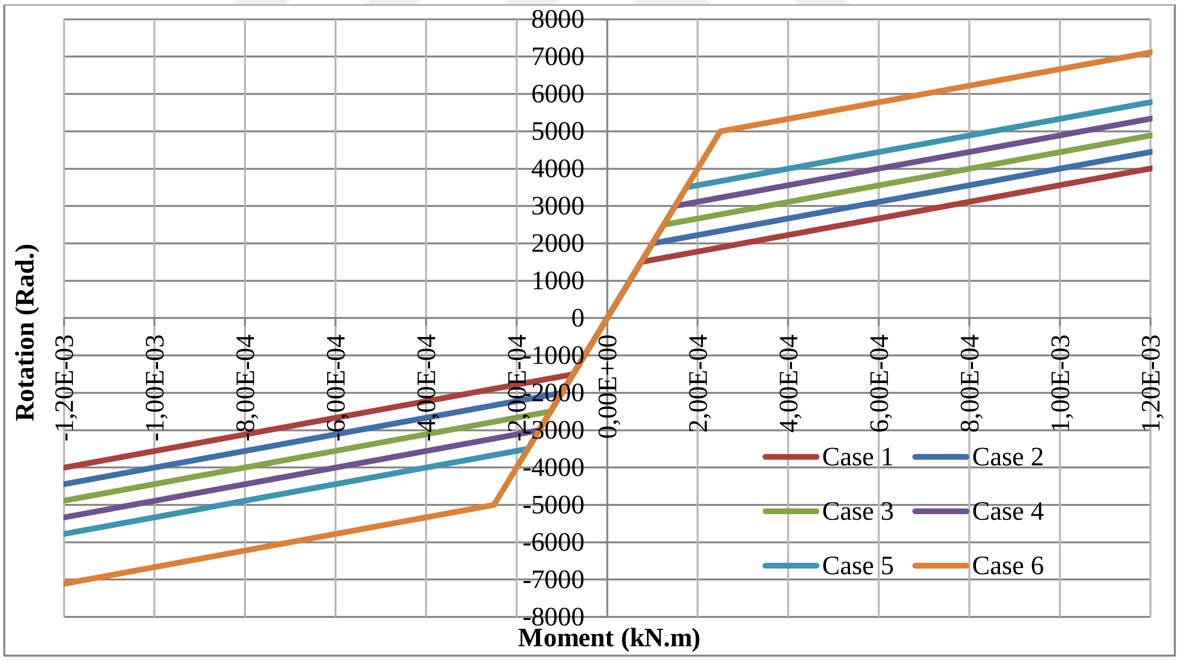


Figure 3.43. Non-linear plastic link member for different yield levels.

The properties for the plastic member are kept unchanged for a moderate level plasticity (level 4 for plasticity levels of plastic link ranging from 1 to 6 for the analysis cases in previous sections) for all the analyses conducted considering uplift effects.

Analyses are conducted by changing the yield levels of non-linear elastic link member. Four different moment threshold levels for elastic link are defined. And the analysis is repeated for each level. This link member is shown graphically below:

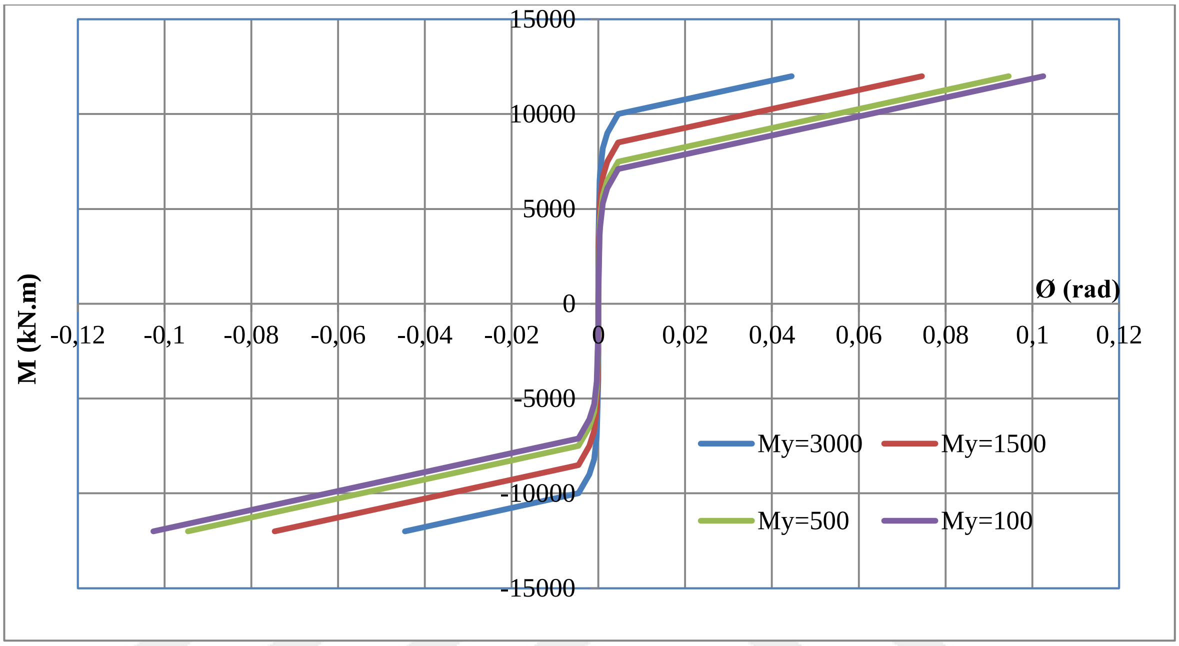


Figure 3.44. Nonlinear elastic link member for foundation uplift.

The link members graphically shown in Figures 3.39 and 3.40 are connected in series and a simplified macro element model is developed. (Model D) The coupled behaviour for these link members depends on the dynamic parameters selected.

Different threshold moments are defined for foundation uplift. It is assumed and interpreted that higher moment threshold values correspond to the situations where rotation will occur about a location closer to corner of the foundation (hard-stiff sandy soil) whereas cases with lower threshold values correspond to soft clayey soil. For plastic link member on the other hand; higher yield level represents the SSI system's lack of plastic deformation capacity and lower yield level represents clayey soils with high levels of plasticity.

A simple representation of the last model (Model D) is presented graphically below:

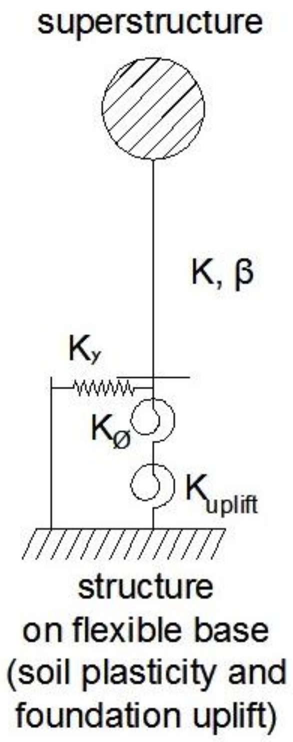


Figure 3.45. Visual representation of model D.

3.4.11. Input Earthquake Ground Motion

For this non-linear analysis cases Yarımca earthquake is used. The PGA value of this earthquake is 0.35 g.

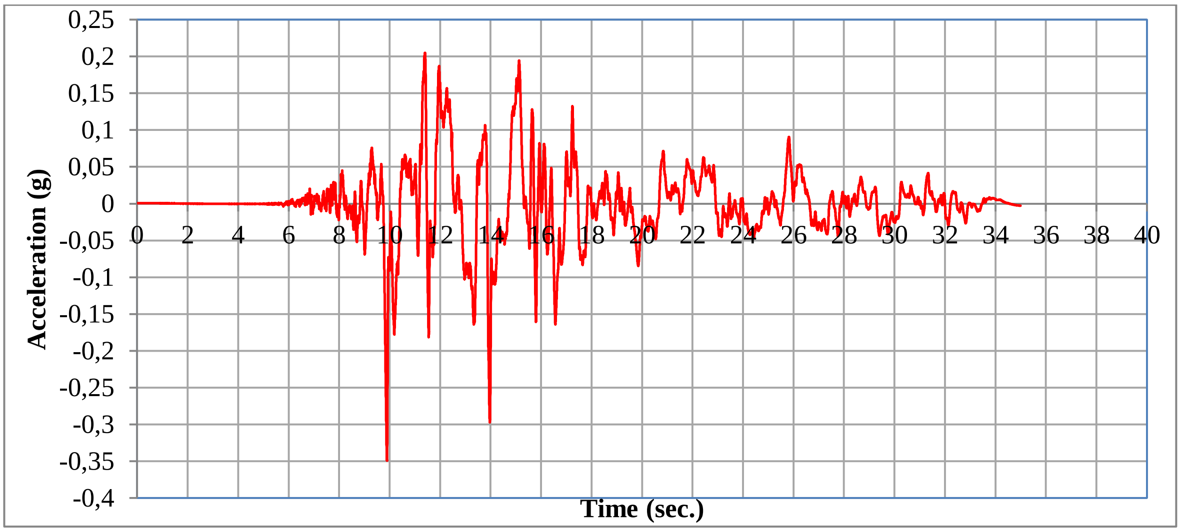


Figure 3.46. Yarımca earthquake (horizontal).

3.4.12.Analyses with "Model D"

For the analysis cases including both non-linear elastic and non-linear plastic links, the response of SSI model under the effect of Yarımca earthquake is calculated and shown graphically below.

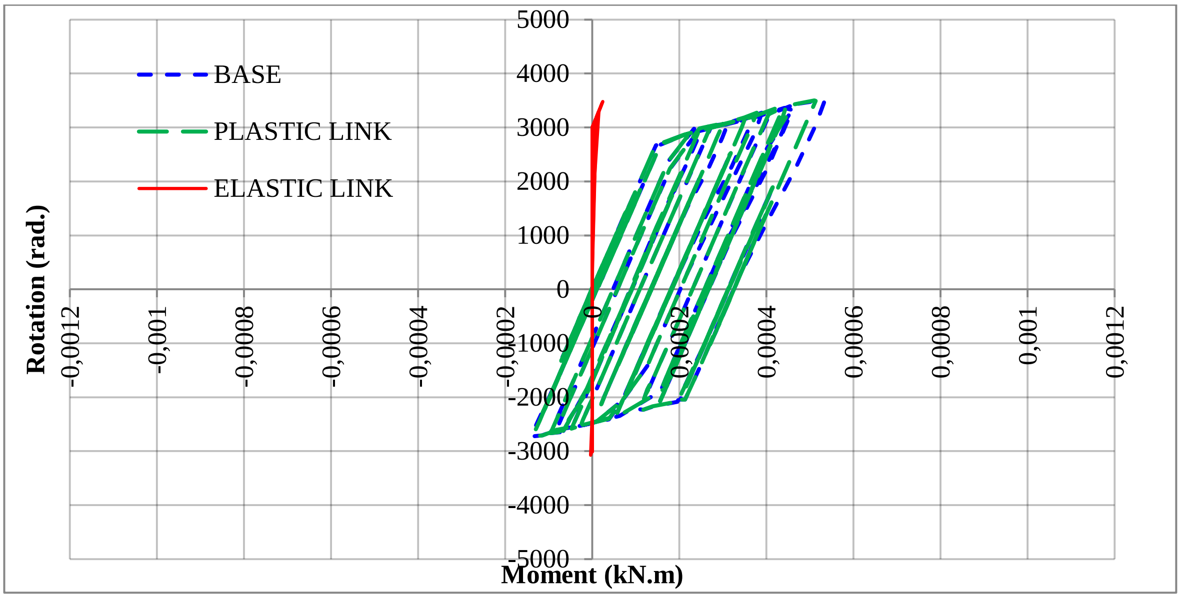


Figure 3.47. Moment rotation curves for high threshold moment level.

(My.el:3000 kN.m)

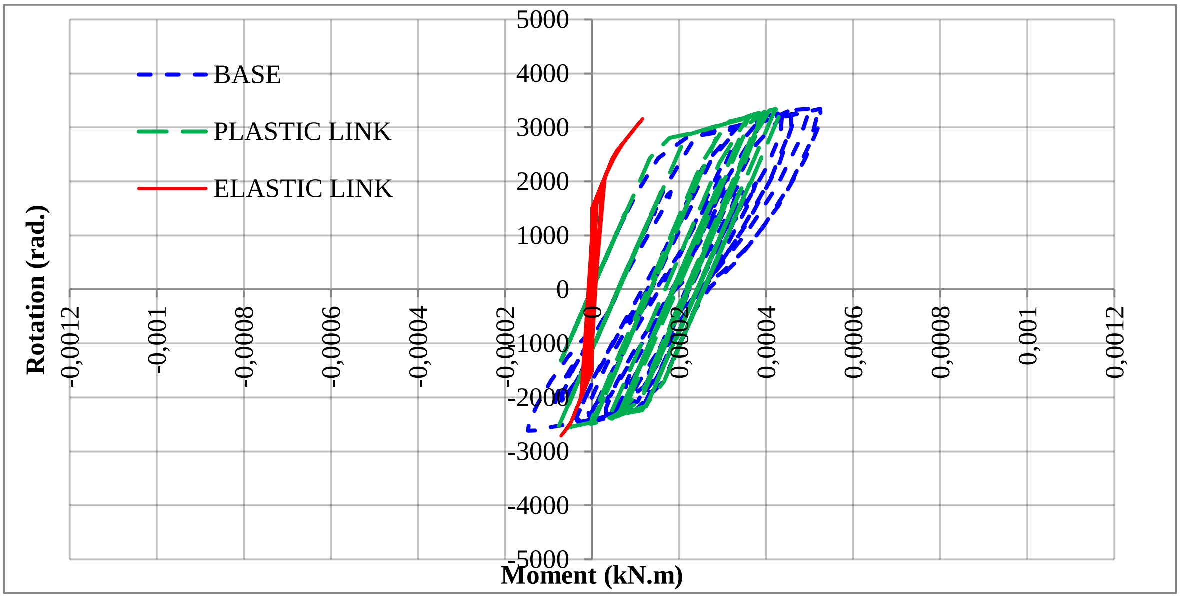


Figure 3.48. Moment rotation curves for medium-high threshold moment level.

(My.el:1500 kN.m)

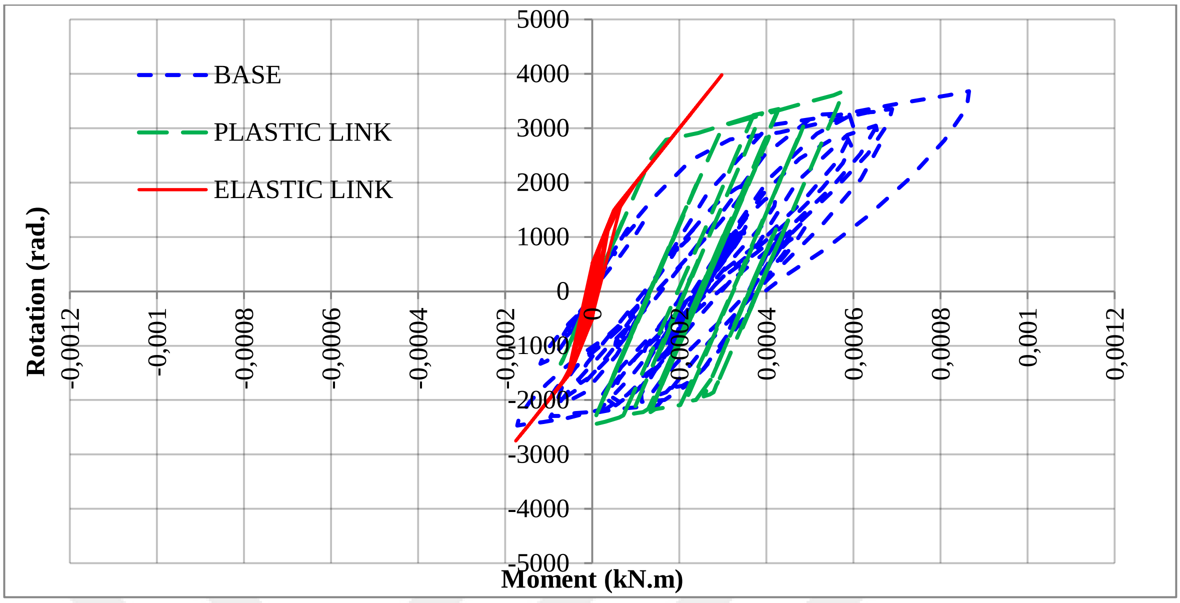


Figure 3.49. Moment rotation curves for medium-low threshold moment level. (My.el:500 kN.m)

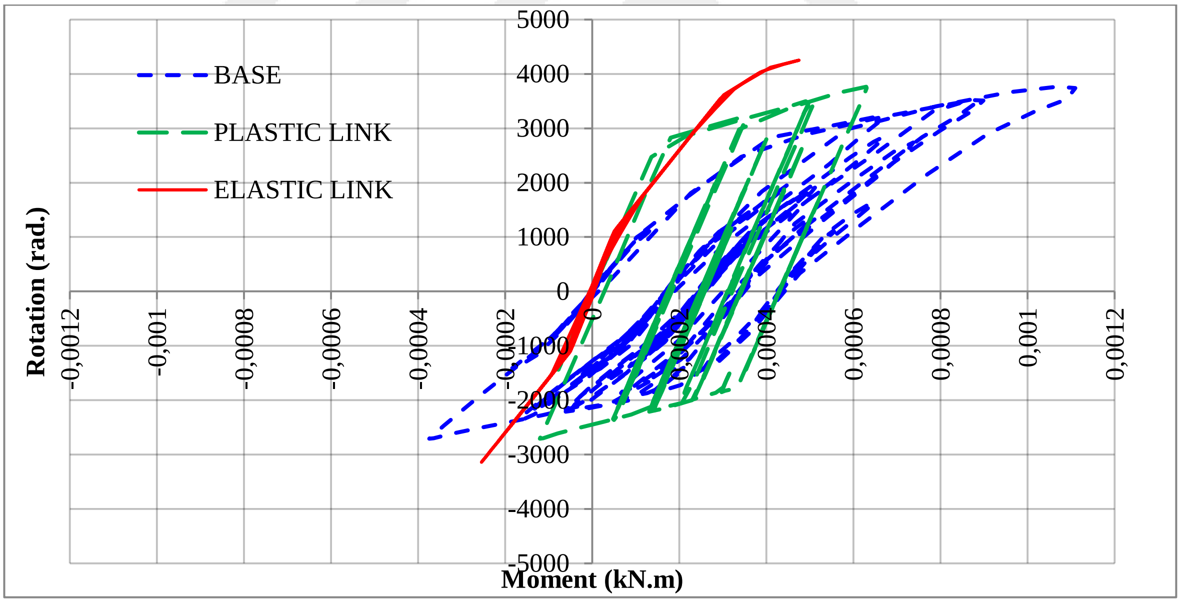


Figure 3.50. Moment rotation curves for very low threshold moment level. (My.el:100 kN.m)

Analyses are conducted for four cases defined at Section 3.4.10. As mentioned before, the parameters for nonlinear analysis cases kept unchanged in order to be able to observe the change in effects of uplift to the overall behaviour of the structure. The moment-rotation behaviour of elastic link, plastic link and combined effect of both links at base are observed and shown graphically.

In the figures (Figure 3.43 to Figure 3.46) uplift response is shown in red (uplift only, non-linear elastic link). Plastic behaviour is shown in green. And combined behaviour (uplift + plastic rotation) is shown in blue. As can be observed in Figure 3.43, since there is no uplift (due to very high level of moment threshold) total behaviour is the same as the response of non-linear plastic member. However as the threshold moment of non-linear elastic link (uplift) it can be observed that since the non-linear elastic member starts activating blue line which represents the total deformation starts to differ from the green line. Also excessive deformations are observed as the level of threshold moment gets lower. This situation especially points out the importance of modelling coupled non-linear members as far as seismic design of tanks resting on shallow foundations are considered.

In summary, when link members connected in series are used, base rotations are significantly greater than the cases when only non-linear plastic links are considered. As noted before the threshold moment and yield level values of nonlinear plastic and nonlinear elastic links are generic. Therefore it is important to mention that the numeric values do not represent a physical actual case but it is a simplified approach on how the behaviour and the response of similar systems are changing when considering the SSI effects.

3.4.13.Verification of “Model D”

The proposed model is actually a simplified representation of a quite complex SSI interface.

When the literature is reviewed it is observed that in the study carried out by “Cremer at. al. 2002” two different methods are used to model the SSI interface:

- A complex FE model of SSI
- A simplified macro-element model of SSI

The representations of these two models are visually presented as below:

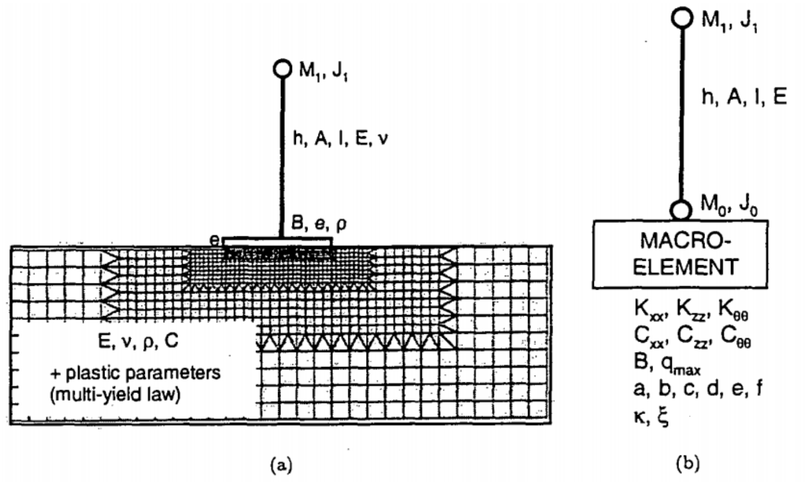


Figure 3.51. Mesh of the soil and the structure (a) Dynaflow mesh; (b) FEAP mash (Cremer at. al. 2002).

When the results of these two different models are compared it is observed that they are quite similar to each other. The finite element modelling is quite complex and the analysis with these model is quite time consuming. On the other hand the simplified macro-element model is proven to be quite accurate and can easily be adapted for practical purposes due to its relative simplicity and it requires much less time to analyse.

Below is the comparison of results carried out by these two models presented by "Cremer at. al. 2002"

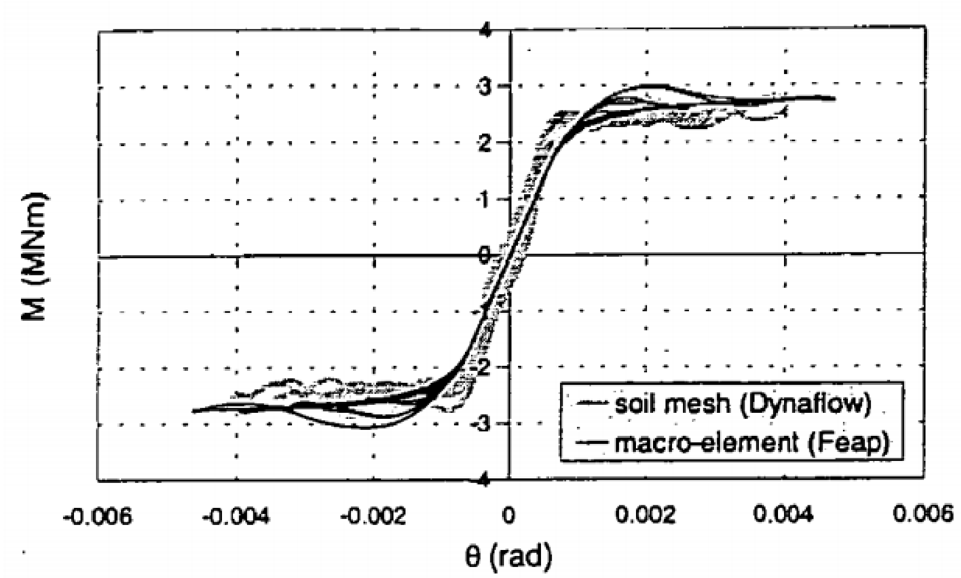


Figure 3.52. Elastic soil with uplift (Cremer at. al. 2002).

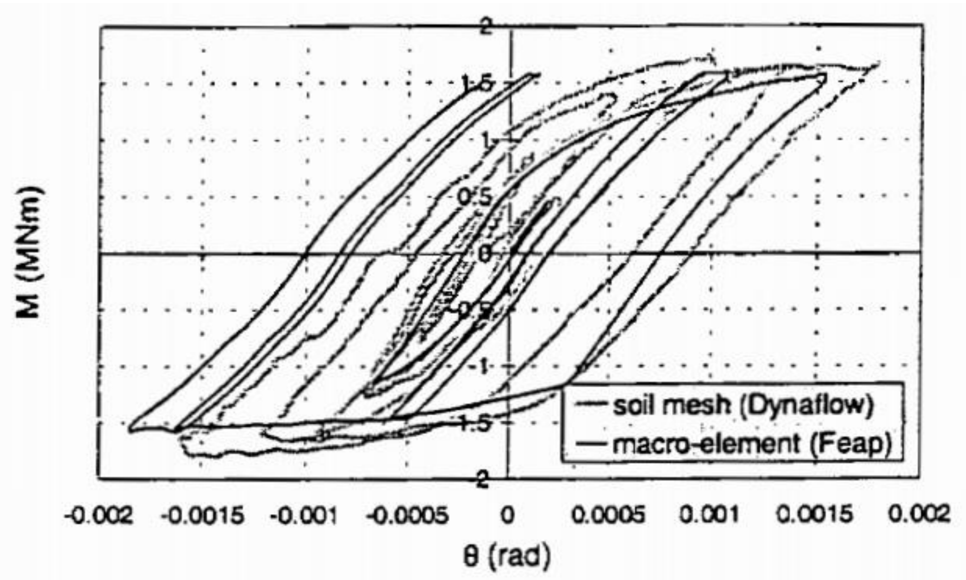


Figure 3.53. Elastoplastic soil with uplift (Cremer at. al. 2002).

In the graphs below the analysis results are drawn using the macro elements presented by "Cremer at. al. 2002". Influence of uplift nonlinearities and the coupled uplift and plastic nonlinearities are presented.

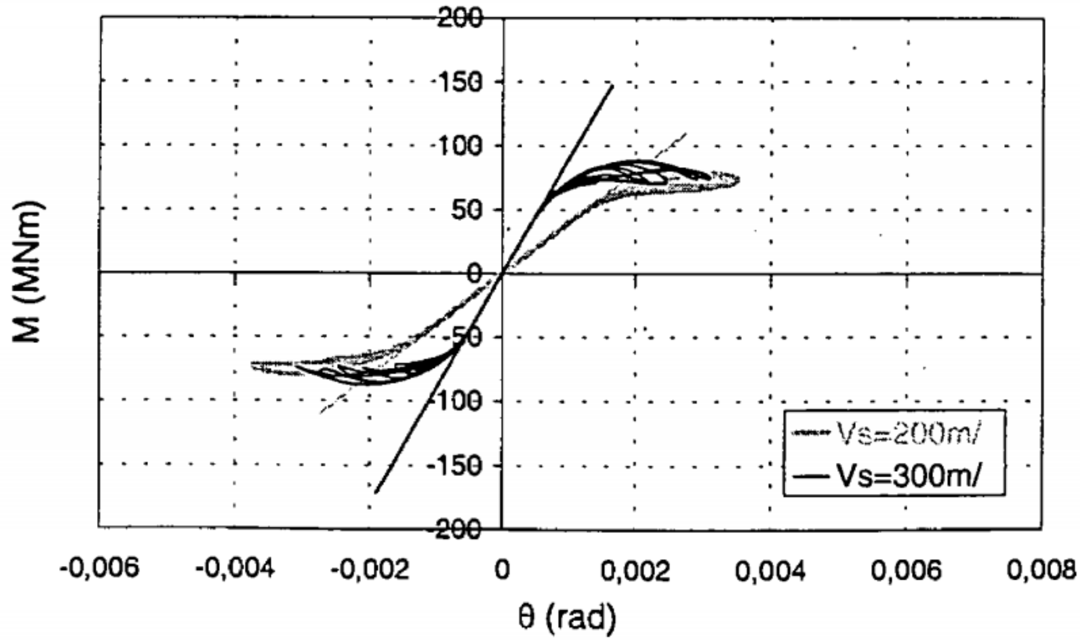


Figure 3.54. Influence of uplift nonlinearities; elastic soils (Cremer at. al. 2002).

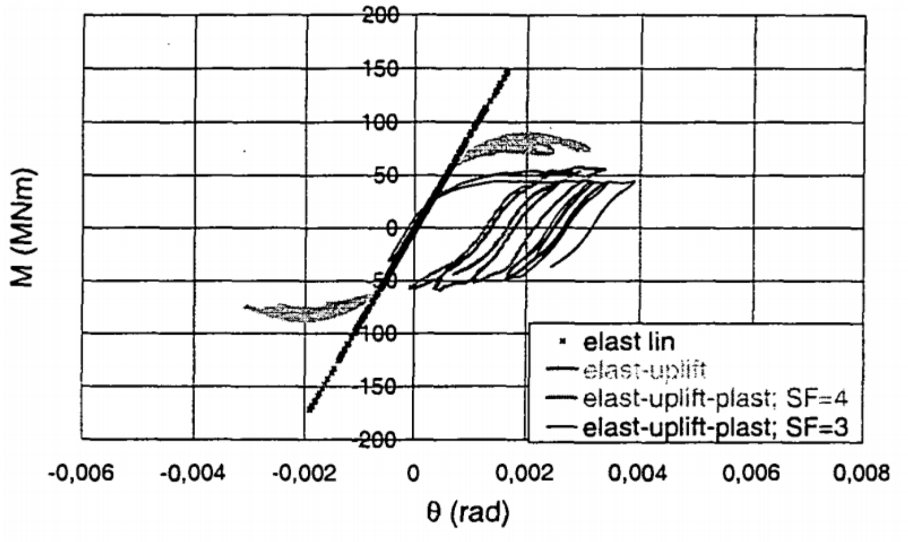


Figure 3.55. Influence of uplift and plastic nonlinearities (Cremer at. al. 2002).

When the trends of the graphs are compared; it can be observed that the SSI model used in this study is consistent and compatible with the macro element model presented by "Cremer at. al. 2002". So in a sense this can be interpreted as a verification of modelling the SSI interface with macro elements.

4. CONCLUSIONS

In this study, earthquake response of an HPV unit under the effect of SSI is examined. As the main engineering demand parameter base deformation (rotation angle) is considered. Nonlinear effects on SSI surface are taken into account and non-linear plastic and non-linear elastic members (which are connected in series) are modelled and analysed.

It is assumed that equivalent damping of the system is due to; plastic damping due to the plastic deformations on SSI surface, and viscous (radiation) damping due to radiating of energy waves in the infinite elastic soil medium and proportional damping of the superstructure proportional to mass and stiffness. All these effects are taken into consideration and analysis results are evaluated.

The effects of soil structure interaction (SSI) for structures resting on shallow foundations:

- Linear elastic behaviour representing soil and foundation working as a whole (soil being in full contact with the foundation)
- Non-linear elastic behaviour representing the dissolution of foundation and the soil
- Non-linear plastic hysteresis behaviour representing the permanent deformations in the soil

It is assumed that overall behaviour will be combination of these three behaviours. This situation is defined as “macro element model” presented by Cremer at al. and adapted to this study.

It is assumed the total damping in SSI interface is governed by hysteretic damping due to plastic behaviour of the soil and radiation geometric (viscous) damping caused by radiating of vibration energy from the foundation into the infinite soil medium. Hysteretic damping is the area inside the moment rotation hysteretic behaviour of the soil-foundation model (non-linear plastic link) and it represents the loss of energy due to plastic deformation. The damping of the superstructure is proportional (Rayleigh) damping proportional to stiffness and mass of the superstructure.

The results and conclusions of this study are discussed below:

The structure resting on a fixed base (when no SSI is involved) has 2% damping and a fundamental vibration period of the structure is 0.12 seconds. Due to SSI the vibration period of the tank increases to 0.22 seconds and reaches the plateau of the response spectrum. However equivalent of the damping is also increased and calculated around 5%. So, when SSI effects are considered with the combined effects of these two changes, the shear force and moment response of the system at base increases about 10% and 5% respectively. Although it is important to note that SSI effects may have an increase in the structural response; for the case studied when the numerical analysis results are compared with limits provided in codes and the literature it is observed to stay within allowable limits.

In industrial structures critical situations are mostly due to deformations. Modelling the SSI interface considering foundation uplift and plastic behaviour of the soil, it is observed that deformations are greater than elastic SSI models and even greater than the models considering no SSI effects.

When foundation uplift and plastic deformations are considered together, for the cases where threshold moments for uplift link are high, it is observed that overall behaviour is similar that of the behaviour of the case where only plastic behaviour (without uplift) is considered. However as the threshold moment for uplift is decreased, non-linear elastic link begins to active and total base rotation response is significantly increased. This situation can be interpreted as the necessity to consider coupled non-linear members for SSI as far as seismic design of structures resting on shallow foundations is concerned.

For the soils with medium yield levels (non-linear plastic link) considering both non-linear plastic and elastic links (soil plasticity and uplift) which are connected in series, it is observed that the base rotation is almost twice bigger than the case where only non-linear plastic link is considered and it is much greater than the response of the linear elastic solutions. The plastic deformations increase even further when soils with high plasticity (with low yield levels) are concerned.

It is observed that threshold yield moment levels used in the simplified models influence the outcome (the response) significantly.

It should be noted that P-Delta effects are neglected in this study. When tall structures are analysed; taking into consideration of these effects may even further increase the force and overturning moment response of these structures. So the study may further be developed by taking into consideration of these effects and investigating their impact on the overall response of the structure.

The developed mathematical model can be adapted for different types of structures by modifying the parameters belonging to superstructure. The proposed model may also further be developed by modelling the foundation-soil medium with finite elements. However it is verified that a simplified modelling approach is quite reasonable when it is compared to a complex FE modelling. It is also suggested that conventional programs like SAP2000 can be used to take into consideration of SSI effects as far as practical design purposes of structures is concerned.

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