

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**FINITE ELEMENT ANALYSES OF QUAY
WALLS ON LIQUEFIABLE SOILS**



by
Çiğda YARAN

February, 2020
İZMİR

FINITE ELEMENT ANALYSES OF QUAY WALLS ON LIQUEFIABLE SOILS

**A Thesis Submitted to the
Graduate School of Natural and Applied Sciences of Dokuz Eylül University
In Partial Fulfillment of the Requirements for the Degree of Master of Science
in Civil Engineering, Geotechnical Engineering Program**

**by
Çiğda YARAN**

February, 2020

İZMİR

M.Sc THESIS EXAMINATION RESULT FORM

We have read the thesis entitled “FINITE ELEMENT ANALYSES OF QUAY WALLS ON LIQUEFIABLE SOILS” completed by ÇİĞDA YARAN under supervision of PROF. DR. GÜRKAN ÖZDEN and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Master of Science.

Prof.Dr. Gürkan ÖZDEN

Supervisor

Yeliz YÜKSELEN AKSOY

(Jury Member)

A. Hakan ÖREN

(Jury Member)

Ender BAŞARI

(Jury Member)

Yalçın ARISOY

(Jury Member)

Prof.Dr. Kadriye ERTEKİN

Director

Graduate School of Natural and Applied Sciences

ACKNOWLEDGEMENTS

I would like to thank my advisor, Prof. Dr. Gürkan Özden for his kind, patience and attention throughout during my thesis study. I am also grateful to the other members of Geotechnical Division of Dokuz Eylül University Civil Engineering Department.

For their endless support and motivation, I would like to give my special thanks to my family- Sadiye Arkayın, Sevil Yaran, İlter Yaran, İlgiz Yaran Ögel and Serdar Ögel. I was able to finish my thesis with their encouragement, understanding and patient.



Çiğda YARAN

FINITE ELEMENT ANALYSES OF QUAY WALLS ON LIQUEFIABLE SOILS

ABSTRACT

In this thesis, back analyses of No.6 Gravity Quay Wall of Derince Port which is one of the several damaged coastal structures in August 1999 Marmara Earthquake, was made. The site reports after the earthquake revealed that the quay wall faced damages in terms of lateral and vertical displacements due to liquefaction of the backfill. Dynamic finite element method was utilized during the analyses in which response of the liquefiable soil was taken into consideration by means of UBC3D-PLM material model. Earthquake motion was defined at the bedrock level using three earthquake records, two of which occurred on the North Anatolian Fault. Analyses results showed that response of the liquefiable backfill should be discussed considering soil-structure interaction and frequency content of the earthquake records. The simple excess pore pressure dissipation technique provided on wall blocks could considerably enhance earthquake response of the quay wall.

Keywords: Gravity quay wall, liquefaction, dynamic finite element method, back analysis, soil-structure interaction

SIVILAŞAN ZEMİNLERE OTURAN RIHTIM DUVARLARININ SONLU ELEMANLAR ANALİZLERİ

ÖZ

Bu tezde, 17 Ağustos 1999 Marmara Depremi sırasında hasar gören kıyı liman yapılarından birisi olan Derince Limanı No.6 Bloklı Rıhtım Duvarı'nın geriye dönük analizleri ele alınmıştır. Geri dolgusundaki sıvılaşma nedeniyle hasarlandığı bilinen bu bloklı rıhtımın deprem davranışı dinamik sonlu elemanlar yöntemi kullanılarak incelenmiş, sıvılaşabilen geri dolgu için UBC3D-PLM bünye malzeme modeli kullanılmıştır. Analizlerde deprem hareketi taban kayasında tanımlanmıştır. İkisi Kuzey Anadolu Fayı'ndan kaynaklanan üç deprem hareketi altında yapılan hesaplar rıhtım duvarı arkasındaki sıvılaşabilen geri dolgu davranışının zemin-yapı etkileşimi ve deprem kaydının frekans içeriği ile birlikte yorumlanması gerektiğini göstermiştir. Rıhtım duvarı blokları üzerinde tesis edilen bir aşırı boşluk suyu basıncı sönümleme sisteminin duvarın deprem davranışını önemli ölçüde iyileştirebileceği belirlenmiştir.

Anahtar Kelimeler: Bloklı rıhtım duvarı, sıvılaşma, dinamik sonlu elemanlar yöntemi, geriye dönük analiz, zemin-yapı etkileşimi

CONTENTS

	Page
M.Sc THESIS RESULT EXAMINATION FORM.....	ii
ACKNOWLEDGEMENTS.....	iii
ABSTRACT.....	iv
ÖZ.....	v
LIST OF FIGURES.....	ix
LIST OF TABLES.....	xii
 CHAPTER ONE – INTRODUCTION.....	 1
1.1 Introduction.....	1
1.2 Scope of The Thesis.....	3
 CHAPTER TWO – REVIEW OF THE LITERATURE ABOUT CASE HISTORIES.....	 4
2.1 An Introduction to Ports and Quay Wall Performance in Earthquakes.....	4
2.2 Performance of Ports during Earthquakes.....	6
2.2.1 Taichung Port, Taiwan.....	7
2.2.2 San Antonia Port, Chile.....	8
2.2.3 Kalamata Port, Greece.....	9
2.2.4 Port of Algiers, Algeria.....	10
2.2.5 Port of Kushiro, Japan.....	11
2.2.6 Port of Kobe, Japan.....	14
2.2.7 Derince Port, Turkey.....	16
2.2.8 Port of Hitachinaka, Japan.....	17
2.2.9 Akita Port, Japan.....	17
2.2.10 Hakodate Port, Japan.....	19
2.2.11 Commercial Port, Guam, USA.....	21
2.2.12 Hakata Port, Japan.....	22
2.2.13 Port of Oakland, USA.....	24

2.2.14 Port of San Fernando, Philippines.....	25
2.2.15 Port de Port au Prince, The North Wharf, Haiti.....	26
2.2.16 Los Angeles Port, USA.....	27
CHAPTER THREE – DESIGN METHODOLOGY.....	28
3.1 Introduction	28
3.2 Definition of Performance Grades.....	29
3.2.1 Design Earthquake Motion.....	29
3.2.1.1 (L1) Earthquake Level.....	29
3.2.1.2 (L2) Earthquake Level.....	29
3.2.1.3 (L3) Earthquake Level.....	30
3.2.2 Classifications in Terms of Earthquake Performance.....	30
3.2.2.1 Private Structures.....	30
3.2.2.2 Normal Structures.....	30
3.2.2.3 Basic Structures.....	30
3.2.2.4 Unimportant Structures.....	31
3.2.3 Damage Performance Level.....	31
3.2.3.1 Minimum Damage Performance Level (MD).....	31
3.2.3.2 Controlled Damage Performance Level (CD).....	31
3.2.3.3 Advanced Damage Performance Level (AD).....	32
3.2.3.4 Collapse Failure Damage Case (CFD).....	32
3.3 Analysis Method.....	32
3.3.1 Simplified Analysis.....	33
3.3.1.1 The Design of Gravity Type Quay Walls under Earthquake Effect...33	
3.3.1.2 Static Horizontal Forces.....	33
3.3.1.2.1 Active Earth Force.....	33
3.3.1.2.2 Surcharge.....	34
3.3.1.2.3 Hydrostatic Forces.....	34
3.3.1.3 Dynamic Horizontal Forces.....	35
3.3.1.3.1 Dynamic Active Earth Pressure Coefficient.....	35
3.3.1.3.2 Dynamic Surcharge Pressures and Dynamic Earth Forces.....	37

3.3.1.3.3 Hydrodynamic Forces.....	37
3.3.1.3.4 Inertia Forces.....	37
3.3.1.4 Stability Analysis in Gravity Type Quay Walls.....	38
3.3.2 Dynamic Analysis.....	40
3.3.2.1 UBC3D-Post Liquefaction Model (PLM).....	42
3.3.2.1.1 Yield Function.....	43
3.3.2.1.2 Elastic Behavior.....	44
3.3.2.1.3 Flow Rule.....	45
3.3.2.1.4 Hardening Rule.....	46
3.3.2.1.5 Load State During Liquefaction.....	46
3.3.2.1.6 Post Liquefaction Rule and Cyclic Mobility.....	48
3.3.2.1.7 Dynamic Boundary Condition.....	49
3.3.2.1.8 Parameters of the UBC3D-PLM Model.....	49
3.3.2.1.9 State Parameters in the UBC3D-PLM Model.....	53
3.3.2.1.10 UBC3D-PLM Model in Dynamic Calculations.....	53
CHAPTER FOUR – A CASE STUDY: NO.6 GRAVITY QUAY WALL OF DERİNCE PORT.....	56
4.1 Introduction	56
4.2 The Soil Properties of No.6 Gravity Quay Wall.....	61
4.3 Liquefaction Potential of Site Soils According to NCEER Method.....	65
4.4 Backward Numerical Analyses of Quay Wall No.6.....	67
4.4.1 Finite Element Model.....	67
4.4.2 FEM Analysis Stages.....	72
4.4.3 Discussion of Analysis Results.....	76
4.5 Studied Method to Decrease Excess Pore Water Pressure Generation.....	93
CHAPTER FIVE – CONCLUSIONS AND RECOMMENDATIONS.....	103
5.1 Conclusions.....	103
5.2 Recommendations.....	105

REFERENCES.....	106
------------------------	------------

APPENDICES.....	110
------------------------	------------



LIST OF FIGURES

	Page
Figure 2.1 Cross section of Taichung Port, Taiwan	7
Figure 2.2 Cross section of San Antonio Port, Chile.....	8
Figure 2.3 Cross section of Kalamata Port, Greece.....	9
Figure 2.4 Cross section of Port of Algiers, Algeria.....	10
Figure 2.5 Plan of Kushiro Port.....	11
Figure 2.6 Caisson wall at the Port Kushiro, Japan.....	12
Figure 2.7 East quay wall at the Port Kushiro, Japan	13
Figure 2.8 Sheet pile wall at the Port Kushiro, Japan	13
Figure 2.9 Ground improvement at the Port Kushiro, Japan.....	14
Figure 2.10 Port of Kobe, Japan	15
Figure 2.11 Derince Port, Turkey	16
Figure 2.12 Cross-section of quay walls in the northern wharf area.....	18
Figure 2.13 Ohama 2 at the Akita Port, Japan.....	18
Figure 2.14 Ohama 1 at the Akita Port, Japan	19
Figure 2.15 Hakodate Port, Japan	20
Figure 2.16 The sand boiling due to liquefaction at Hakodate Port, Japan.....	21
Figure 2.17 Commercial Port, USA	22
Figure 2.18 The location of No.1 and No.2 quay, Japan	22
Figure 2.19 The cross section of the No.1, Japan	23
Figure 2.20 The sand boiling due to liquefaction (a) at front apron; (b) at back apron.....	23
Figure 2.21 Liquefaction at behind quay wall.....	24
Figure 2.22 Port of Oakland, USA.....	25
Figure 2.23 Port of San Fernando, Philippines.....	25
Figure 2.24 Port de Port au Prince, Haiti.....	26
Figure 2.25 The B126(New) APL terminal in Los Angeles Port, USA.....	27
Figure 3.1 Yield surfaces.....	44
Figure 3.2 The process of cyclic mobility for dense sand	49
Figure 3.3 The input parameters of the UBC-3D-PLM model.....	53
Figure 3.4 The input parameters of the Hardening soil model.....	55

Figure 4.1 The major marine structures and ports in Izmit Bay.....	56
Figure 4.2 Gravity type quay wall inside the Derince Port.....	58
Figure 4.3 Layout of the Derince Port along with the wharfs with gravity type quay walls (i.e. wharfs 6~8)	59
Figure 4.4 Dimension of block type quay walls (unit: m)	60
Figure 4.5 The damages due to 1999 Marmara Earthquake in gravity quay wall in Derince, (a) The liquefaction traces in backfill between transit warehouse and quay wall; (b) Seaward movement of the quay wall; (c) The lateral displacement patten after earthquake.....	61
Figure 4.6 The borehole location plan.....	62
Figure 4.7 Soil profile along Section A-A Along Wharf #6.....	63
Figure 4.8 Idealized geotechnical model based on average SPT blow count data.....	64
Figure 4.9 The second idealized geotechnical model based on variable SPT blow count data.....	65
Figure 4.10 The third idealized geotechnical model based on variable SPT blow count data.....	65
Figure 4.11 The relation between cyclic resistance ratio and corrected SPT.....	66
Figure 4.12 Liquefaction analyses with respect to NCEER Method for Kocaeli Station Record.....	67
Figure 4.13 The model geometry of dynamic finite element.....	70
Figure 4.14 The model geometry for stratified soil profile.....	71
Figure 4.15 The model geometry for stratified soil profile with non-liquefiable backfill.....	71
Figure 4.16 The acceleration time series related to analyses and general properties related to series.....	74
Figure 4.17 The elastic acceleration spectrum for %5 damped ratio at the bedrock level.....	75
Figure 4.18 The change of excess pore water pressure ratio in time for stratified soil profile in undrained model (a) Düzce Earthquake scaled according to Kocaeli Earthquake; (b) Kobe Earthquake scaled according to Kocaeli Earthquake; (c) Kocaeli Earthquake.....	77

Figure 4.19 The change of excess pore water pressure ratio in time for stratified soil profile with non-liquefiable backfill in undrained model (a) Düzce Earthquake scaled according to Kocaeli Earthquake; (b) Kobe Earthquake scaled according to Kocaeli Earthquake; (c) Kocaeli Earthquake.....	78
Figure 4.20 The comparison of excess pore water pressure ratio in time between stratified soil profile and stratified soil profile with non-liquefiable backfill in undrained model for Kocaeli Earthquake.....	80
Figure 4.21 The calculated acceleration-time series behind the quay wall in Kocaeli Earthquake analysis and the comparison with the earthquake record defined at bedrock level.....	80
Figure 4.22 The distribution of excess pore water pressure ratio in dynamic finite element model in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	81
Figure 4.23 The cumulative displacement and quay wall displacement in terms of “deformed mesh” form in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	82
Figure 4.24 The vertical displacement distribution in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	83
Figure 4.25 The lateral displacement distribution in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	84
Figure 4.26 The distribution of excess pore water pressure ratio in dynamic finite element in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	85
Figure 4.27 The cumulative displacement and quay wall displacement in terms of “deformed mesh” form in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	86
Figure 4.28 The vertical displacement distribution in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	87

Figure 4.29	The lateral displacement distribution in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	88
Figure 4.30	The distribution of excess pore water pressure ratio in dynamic finite element in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	89
Figure 4.31	The cumulative displacement and quay wall displacement in terms of “deformed mesh” form in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	90
Figure 4.32	The vertical displacement distribution in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	91
Figure 4.33	The vertical displacement distribution in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill.....	92
Figure 4.34	The new model geometry.....	93
Figure 4.35	The change of excess pore water pressure ratio in stratified soil profile with non-liquefiable backfill in drained model (a) Düzce Earthquake scaled according to Kocaeli Earthquake; (b) Kobe Earthquake scaled according to Kocaeli Earthquake; (c) Kocaeli Earthquake.....	95
Figure 4.36	The cross-sections for reviewing liquefaction behavior.....	97
Figure 4.37	Comparison of excess pore water ratio between the original wall and new model for Kocaeli Earthquake.....	98
Figure 4.38	The excess pore water pressure ratio is compared between stratified soil profile with non-liquefiable backfill and stratified soil profile	102

LIST OF TABLES

	Page
Table 2.1 Quay walls around the World under earthquake effects.....	7
Table 3.1 Performance levels projected on several earthquake levels.....	32
Table 3.2 The short period soil coefficient (F_a).....	36
Table 3.3 The coefficients of bottom frictions in gravity type quay walls.....	38
Table 3.4 Factor of safety against sliding and overturning according to performance level.....	39
Table 4.1 The earthquake data.....	57
Table 4.2 Proposed damage criteria for gravity quay walls.....	68
Table 4.3 Hardening soil material model (HS) parameters.....	69
Table 4.4 UBC3D-PLM model parameters.....	70
Table 4.5 The construction staged.....	73
Table 4.6 The assigned material parameter to open flow channels.....	82
Table 4.7 The coarseness factor for polygon/cluster.....	82

CHAPTER ONE

INTRODUCTION

1.1 Introduction

While economies of the countries are mostly based on agriculture in the past, nowadays, they are based on industry and tourism. With the developments in international industrialization as regard to the increase in globalization, import and export has gained importance for the countries in global competition and so they are considered as main building blocks of economies.

When national and international visitors for a country are taken into account, their financial value for that country cannot be ignored. In order to attract the attention of foreign visitors, countries are increasingly in tendency of making huge investments. In consequence of these, construction of ports become very important in transportation of foreign visitors. Thus, ports become a significant factor in the economic improvement of coastal communities.

Besides the importance of ports in transportation of foreign visitors within the context of tourism, ports are also important for the transportation of final goods and raw materials. For both national and international transportation networks, ports are considered as very important nodes. When their importance in transportation is taken into account, it can be considered that ports play a significant role in trading of the countries because, through the ports, functions such as shipping and distribution, and other functions as regard to transportation of cargoes via water way can be carried out. When it is compared with the other modes of transportation such as land and air, trade through water is dominant mode for many countries, and so the demand for the ports is increasing today (Na et al., 2008).

Moreover, in terms of regional and local transportation lifeline systems, ports are often playing a role of regional economic centers. Since, ports are very important for the economic development of countries and opening door to other countries for

economic cooperation, some natural disasters such as an earthquake can distract economy of the countries (Na et al., 2008).

Ports can be constructed worldwide where there is sea and trade. Thus, if there is sea and trade potential, ports can be constructed at different countries that have different economies, different cultures and hence different port typologies (Borg, 2007) . Since, they are located in different geographical areas, ports are characterized according to different seismic hazard levels. For instance, many ports are located in areas which are characterized by moderate to high seismic hazards. Earthquakes happened in the past have sometimes damaged the structures in the ports. However, these earthquakes did not only cause physical damage, but also generated large economic and social losses on both national, local and individual scales. Thus, in terms of their importance in trade, economy, transportation, tourism and individuals, it is required to well-design ports against earthquake loads (Borg, 2007) in addition to wave, berthing and similar marine actions.

A port consists of several sub-sections and structures. Quay walls are port structures for berthing and mooring of the ships. Quay wall design is crucial since it links a ship with the ground. Their stability and serviceability, as well as their safety should be considered in due course. Especially during earthquakes, quay walls must not suffer in order to continue their service and maintain their stability.

In consequence of earthquake damages, the seismic design for ports play significant role. Thus, the aim of this study is to back analyze a quay wall which is placed in liquefiable soils using finite element method, and compare the results of the analysis model with field performance of the actual wall (Derince Port No.6 Quay Wall damaged in 1999 Marmara Earthquake) while suggesting a new liquefaction remediation technique for gravity type quay walls.

1.2 Scope of the Thesis

In the direction of the aim of the study, the rest of this thesis is organized as follows. In Chapter Two, general information related to quay walls is presented. Quay walls around the world that damaged during earthquake are reviewed. Earthquake effects on the quay wall are discussed.

In Chapter Three, current design methodology for gravity type quay wall is given. The design of gravity type quay walls under earthquake effect is addressed in this chapter. Firstly, the simplified analysis is examined by using pseudo-static method and then, dynamic finite element analysis is covered using UBC3D-PLM model.

In Chapter Four, back analyses are performed for Derince Port No.6 Gravity Quay Wall. In this context, dynamic analysis is done using a commercial finite element software. The model geometry was defined, and the material parameters are determined for the UBC3D-PLM and Hardening Small Strain (HS-small) material models. The dynamic action was defined at the bedrock model for three different earthquake motions. Based on achieved results, a new FEM model was established in order to check effectiveness of a newly proposed liquefaction remediation technique for gravity type quay walls. The results of two different models under three earthquake motions were compared with each other and discussions were made.

In Chapter Five, conclusions and recommendations were made from the soil-structure interaction point of view. It was concluded that performance-based design shall be the contemporary design approach since it was shown that neither the soil nor the structure (gravity type quay wall in this case) were not the sole governing factors on the response of the wall to earthquake action. The frequency content of the earthquake motion and gravity quay wall-soil interaction were found to be effective especially on liquefaction potential of the cohesionless backfill of the wall. Based on the results, recommendations are made pertaining to the improvement of the current design methodology and future academic studies.

CHAPTER TWO

REVIEW OF THE LITERATURE ABOUT CASE HISTORIES

2.1 An Introduction to Ports and Quay Wall Performance in Earthquakes

Moving over the water is one of the oldest transportation ways since almost the beginning of the humanity. With the aim of exploring the world and conquering new territories as well as gaining economic power, people have needed to use water for their transportation. Thus, both conquerors with the aim of gaining new parts in the world and traders for their economic concerns, were sailing over the oceans.

The people who have travelled over the sea also needed places to berth their ships and this led to construction of harbors later on. Thanks to these developments in construction of harbor in the past, knowledge about new technologies in harbor construction has accelerated as well as increase in the growth of prosperity of countries which have borders to sea.

The world's oldest well-known port nearby Lothal (India), is more than 4000 years old serving as a trade major node in Asia. As well as Asia, harbors were also constructed for trade in the Mediterranean. For instance, Alexandria, which was established at the last three centuries before Christ, is known as the main center for trade in this region. To increase the effectiveness and efficiency of trade, construction of port has increased in these centuries. With the increase of the knowledge in constructing harbor, new techniques have emerged. For instance, the Romans are accepted as the first in using a kind of concrete for the construction of quay walls (Karakuş, 2007).

As the need of people has changed over the ages, they also needed to build ports for their different purposes like as commercial ports, cruise terminal ports, ferry ports, fishing ports, yacht marinas, dock yard ports, military ports, and coast guard ports. Actually, one port is enough to carry out more than one of these functions, and so the

port can be divided into different sections for performing these different functions (Borg, 2007).

The most common types of port are commercial ports which are constructed for the aim of providing the flow of goods and any trade materials between countries. Thanks to the increase in globalization, countries are becoming closer than they became in the past, and thanks to the increase in trade, their economies become stronger than in the past. Since the traders want to transport their goods and raw materials to far parts of the world, logistics is going to be more complicated. Thus, the means of transport has started to be considered as strategically important decisions that must be given by logistic firms to increase their efficiency to gain competitive advantage in the world (Borg, 2007).

Since the importance of the ports in export and import of countries is increasing day by day, to meet functional and economic needs, the design and construction of the ports must be strengthened to increase their resistance especially in the case of natural disasters which lead to physical damages. In the direction of this aim, especially for the highly seismic and moderately seismic regions, the design and analysis of a wharf structure typology must be strengthened to meet performance requirements in the case of a large earthquake.

One of the main structures of ports that suffered from damage during the large earthquakes are quay walls. Quay wall which is a major structure of a port is constructed to berth the ships as earth-retaining structures. To both mooring the ships and preventing possible negative effects of the vessels to the ports, bollards are mostly used in the construction of quay walls. Quay walls are designed and built not only for one purpose, they also retain the backfill soil, provide bearing capacity for crane loads, goods and storage (Karakuş, 2007).

Gravity walls are the most common type of the quay wall which is used all around the world. Gravity type quay walls, which are composed of caisson, block type and other gravity retaining structures, are constructed on the seabed. The stability of

gravity quay walls against earth pressure from the backfill soil behind the wall is ensured with the friction at the bottom of the wall. In gravity quay walls that are constructed on firm foundation seaward displacement and tilting have been observed during earthquake. In addition, seaward displacement, settlement and tilting on the quay wall may arise from loose sand behind the wall or natural loose sand beneath the foundation (PIANC, 2001).

Since the transportation by water is increasing, quay walls are going to play a significant role in future design of ports. Designing and constructing the quay is very important for the port because stability and resistance of the wall is required to prevent damages during the earthquakes. Thus, the required stability must be provided by the foundations.

In this sense, it is important to check the structure of quay walls in liquefiable soils to reduce the extent of damage during the earthquakes. Throughout the study, to comprehend and describe different quay wall typologies, elements that constitute quay walls, and the damages associated with these walls, a series of case studies about the quay wall structures in the literature which were damaged because of earthquakes are discussed. Quay walls around the world which were damaged during earthquakes are presented briefly in Table 2.1.

2.2 Performance of Ports during Earthquakes

In this part, earthquakes around the world and effects on ports are scrutinized in an order as it is shown in Table 2.1. It is observed that the quay wall suffered from backfill liquefaction during earthquake. In many cases, liquefaction takes place in loose sand layers and reclaimed lands and causes serious damage during earthquake such as seaward displacement, settlement and tilting. In this sense, determination of the design method and parameters play crucial role in the design of quay wall. In the following part, the case studies related to the impact of earthquakes on quay wall are presented.

Table 2.1 Quay walls around the World under earthquake effects

No	Port Name	Port Structure	Quay Wall Type	Earthquake
1	Taichung Harbour	gravity quay wall	caisson	Chi-Chi
3	Kalamata Port		block type	Greece
4	Port of Algiers		block type	Chenoua
5	Port of Kushiro		caisson	Kushiro-Oki
6	Port of Kobe		caisson wall	Great Hanshin
7	Derince Port		block type	Izmit
8	Hitachinaka Port		caisson wall	Great East Japan
9	Akita Port	sheet pile quay wall	sheet pile	Nihonakai-Chubu
10	Hakodate Port		sheet pile	Hokkaido-Nansei-Oki
11	Commercial Port		sheet pile	Guam
12	Hakata Port		sheet pile	Fukuoka
13	Port of Oakland	pile supported wharf	reinforcement concrete piles	Loma Prieta
14	Port of San Fernando		reinforcement concrete piles	Luzon
15	Port de Port au Prince, The North Wharf		reinforcement concrete piles	Haiti
16	Los Angeles Port		prestressed octagonal concrete pile	Northridge

2.2.1 Taichung Port, Taiwan

Taichung Port is an international port which is located in Taiwan as the second-largest port. Figure 2.1 shows that the total height and water depth of its quay wall is 19.2 m. and 13 m., respectively. The type of this quay wall is block caisson which can also be considered as gravity type.

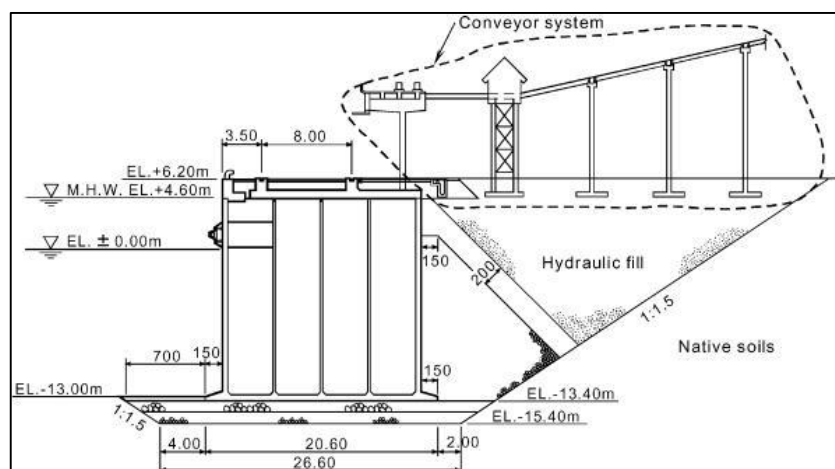


Figure 2.1 Cross section of Taichung Port, Taiwan (PIANC, 2001)

The caisson No.1 Quay Wall at the Taichung port suffered damages during the Chi-Chi earthquake on 21st September 1999. The earthquake had a magnitude of M_s 7.7. The evaluated peak ground surface acceleration based on strong motion earthquake records was measured as 0.16g. Design seismic coefficient (k_h) of caisson wall was 0.15 (PIANC, 2001).

The port was constructed on a firm foundation. The horizontal permanent displacement and vertical displacements were 1.5 m and 0.1 m, respectively. The loosely filled sands behind the caissons were liquefied due to strong shaking. Due to liquefaction, seaward displacement was observed as 1.0 m in average and the settlement of the backfill was about 70 cm (Chen et. al., 2000).

2.2.2 San Antonio Port, Chile

San Antonio Port is in Chile. The total height and water depth are 14.5 m and 9.6 m, respectively. The type of this quay wall is block type quay wall as shown in Figure 2.2.

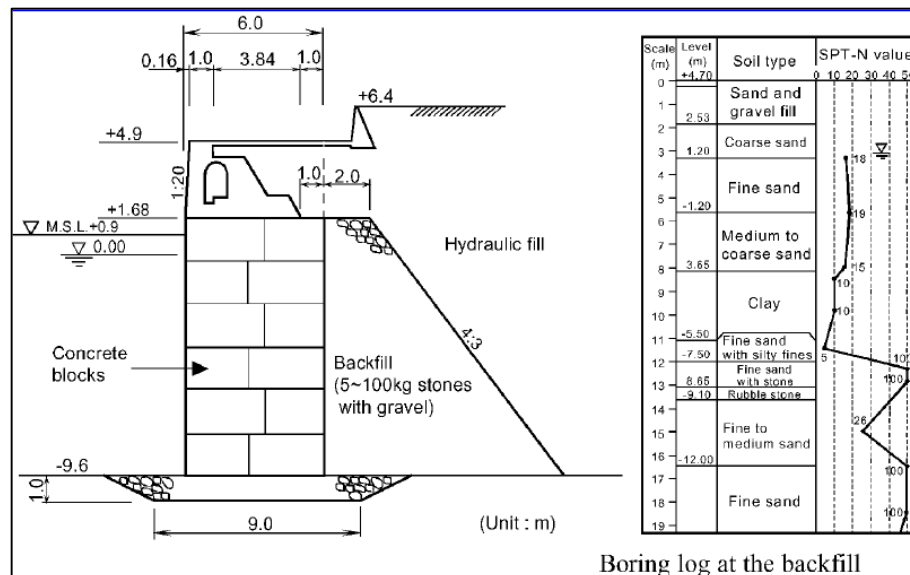


Figure 2.2 Cross section of San Antonio Port, Chile (PIANC, 2001)

The wall suffered severe damages during the Chile earthquake on 3rd March 1985. The earthquake magnitude was $M_s=7.8$. The evaluated peak ground surface acceleration based on strong motion earthquake records was measured as 0.67g.

Breakwater and wharf facilities were extensively damaged due to extension and surface subsidence as well as road and railroad lines along the waterfront. Tilting and toppling of the wharf cranes were observed because of differential subsidence of as much as 90 cm of the fill relative to the breakwaters on the west wharf (Algermissen et al., 1985). The wall collapsed due to backfill liquefaction and the strong earthquake motion (Borg, 2007).

2.2.3 Kalamata Port, Greece

Kalamata Port is a small coastal harbor which is situated in the second most populous city of the Peloponnese Peninsula in Southern Greece. The wall type is a gravity quay wall comprising precast blocks. The quay wall was placed on an excavated platform with fill of sandy gravel materials forming the foundation layer prior to backfilling with stone debris and likely hydraulic fills (Taylor and Sarma, 2008). The total height of wall and water depth is 11.6 m and 9.5 m, respectively (Figure 2.3).

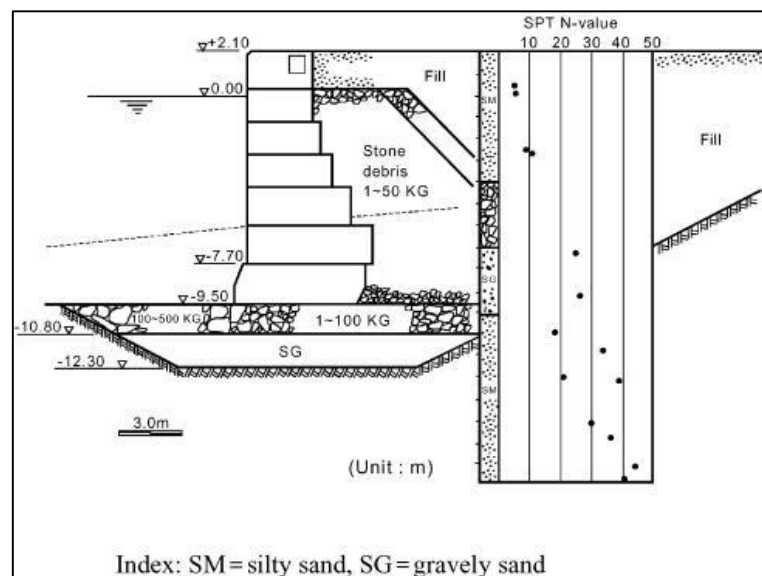


Figure 2.3 Cross section of Kalamata Port, Greece (PIANC, 2001)

The gravity type quay walls at the Port of Kalamata suffered damages during the 13th September 1986 earthquake. The earthquake had a magnitude of $M_s=6.2$. The evaluated peak ground surface acceleration (PGA) was measured at the surface between 0.2g and 0.3g (PIANC, 2001).

The wall was constructed on firm foundation. The horizontal seaward displacement, however, varied between 0.2 m and 0.1 m. The tilt of block type quay wall was in between 4 and 5 degrees. The backfill settled 20 cm, but no settlement took place at a distance of 30 to 40 m away from the wall. The quay wall remained serviceable (Pitilakis and Moutsakis, 1989).

2.2.4 Port of Algiers, Algeria

The port of Algiers which is located in the north-western part of the Bay of Algiers is open to the Mediterranean Sea. This quay wall is gravity type as presented in Figure 2.4. The total height of the wall and water depth are 12.2 m and 10 m, respectively.

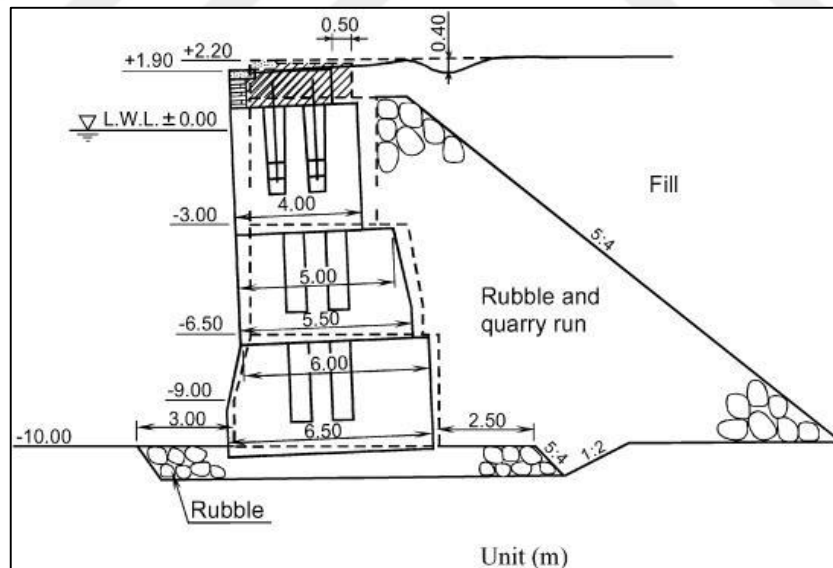


Figure 2.4 Cross section of Port of Algiers, Algeria (PIANC, 2001)

The wall at the Port of Algiers suffered damages during Chenoa earthquake on 29th October 1989. The earthquake had a magnitude of $M_s=6.0$. The epicentre was 60 km from the west of Algiers.

The horizontal and vertical displacements were measured as 0.5 m and 0.3 m, respectively. The rockfalls and landslide induced to close coastal roads (PIANC, 2001). No additional detailed information is available about the consequences of the earthquake (PIANC, 2001).

2.2.5 Port of Kushiro, Japan

The Kushiro Port is located in eastern part of Hokkaido Island and mouth of the Kushiro River. Kushiro Port, which is the largest one in eastern Hokkaido, includes sheet pile quay and gravity type quay walls as shown in Figure 2.5 (Kokusho, 2009).

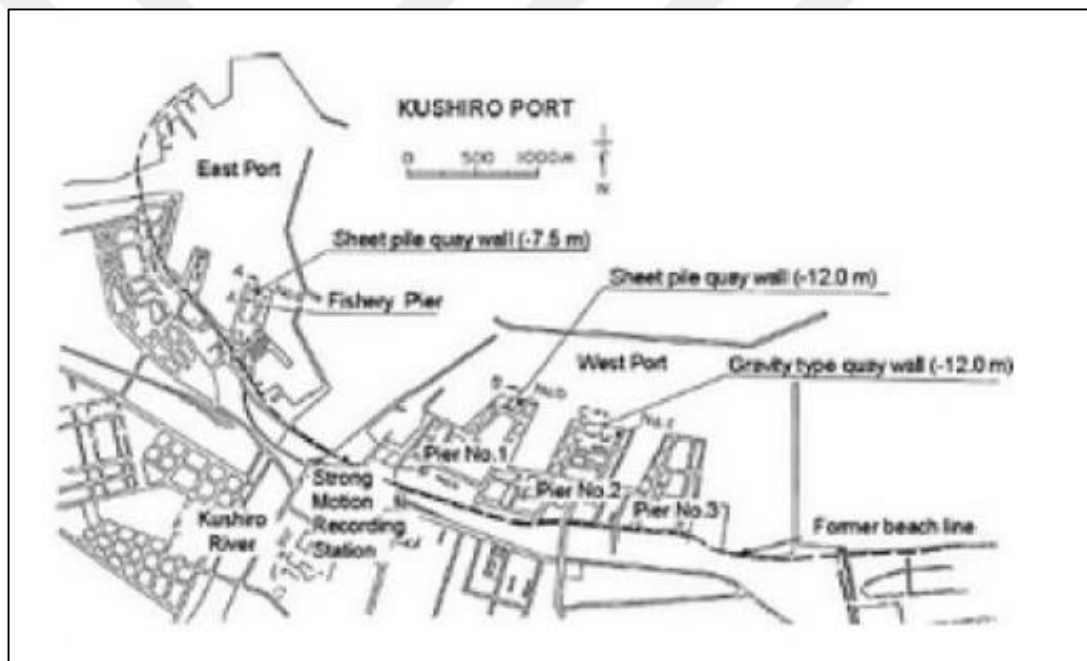


Figure 2.5 Plan of Kushiro Port (Kokusho, 2009)

The caisson quay wall is located at the west port district in Kushiro Port. The total height of the wall and water depth are 12 m and 9 m, respectively as it is shown in Figure 2.6. The quay walls suffered damages during Kushiro-Oki earthquake on 15th January 1993. The earthquake had a magnitude of $M_s=7.8$. The surface peak ground surface acceleration (PGA) was measured as 0.47g. Design seismic coefficient (k_h) of caisson quay wall was set as 0.2.

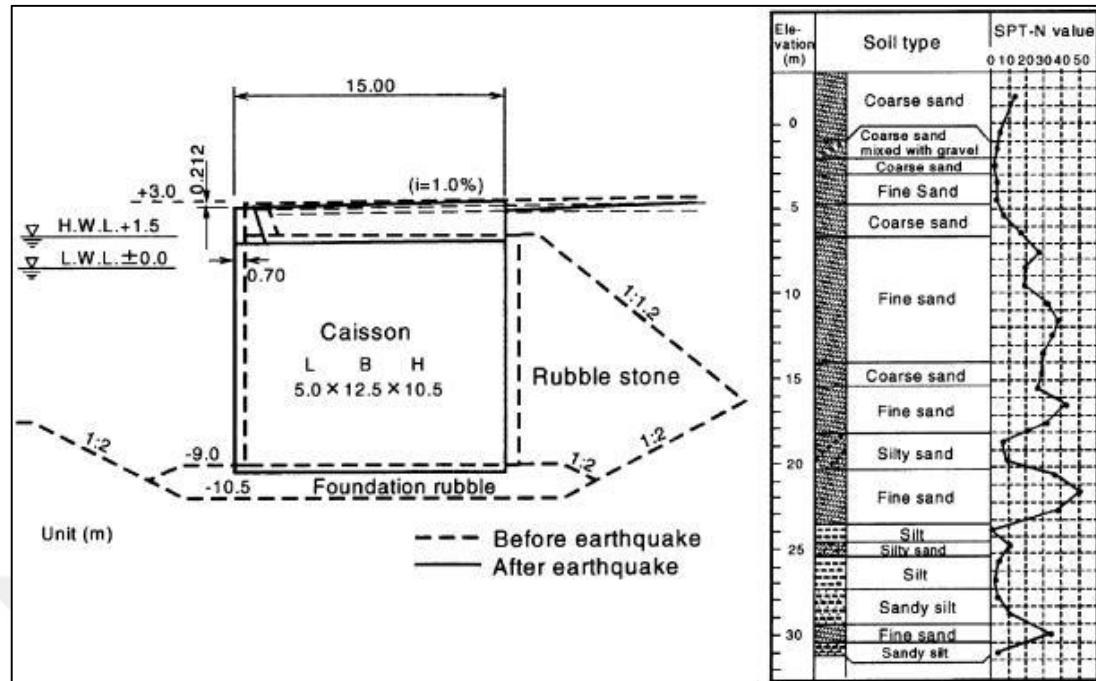


Figure 2.6 Caisson quay wall at the Port Kushiro, Japan (PIANC, 2001)

The wall was constructed on a firm foundation. The horizontal and vertical displacements were determined as 0.75 m and 0.2 m. The tilt of the wall happened to be 2% following the earthquake. Seaward displacement and liquefaction of the backfill were observed. The caisson moved towards the sea about 26 cm and settled about 40 cm (Kokusho, 2009).

During the same earthquake, the eastern quay wall of Kita wharf at the East Port District suffered damage. Quay walls were constructed on relatively dense seabed deposits from the sand and backfilled with dredged sand (Kokusho, 2009). The total height of the wall and water depth are 11 m and 8.5 m, respectively as given in Figure 2.7. Design seismic coefficient (k_h) of caisson quay wall was 0.15.

Seaward displacement of the caisson quay wall was 1.9 m. The vertical displacement varied between 0.2 m and 0.5 m. Liquefaction of the backfill was observed. The displacement in the wall was relatively larger when compared to other similar damaged caisson walls because the width to height ratio is small (PIANC, 2001).

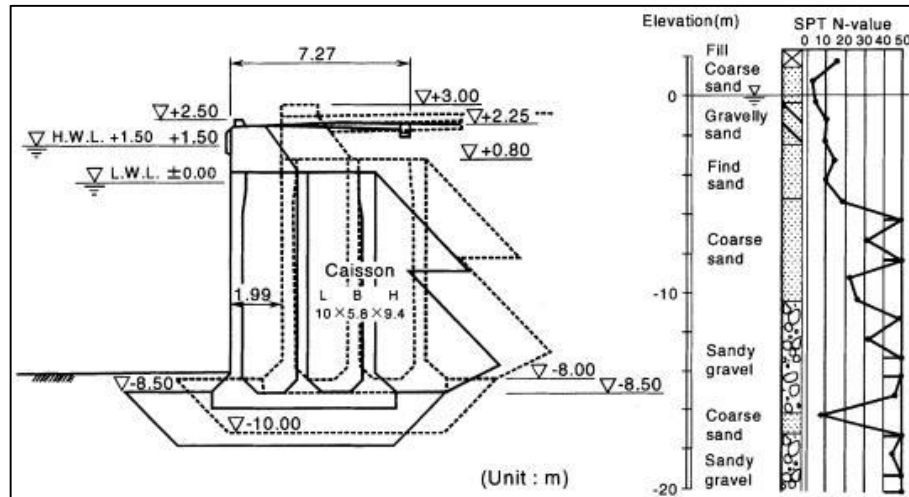


Figure 2.7 East quay wall at the Port Kushiro, Japan (PIANC, 2001)

The southern sheet pile in the Fishery part of East Port District suffered damage at the same earthquake. The steel sheet pile was connected with battered steel piles using anchor rods (Kokusho, 2009). The height of the sheet pile above the seabed and water depth are 10.2 m and 7.5 m, respectively (see Figure 2.8). Design seismic coefficient (k_h) of sheet pile quay wall was 0.2.

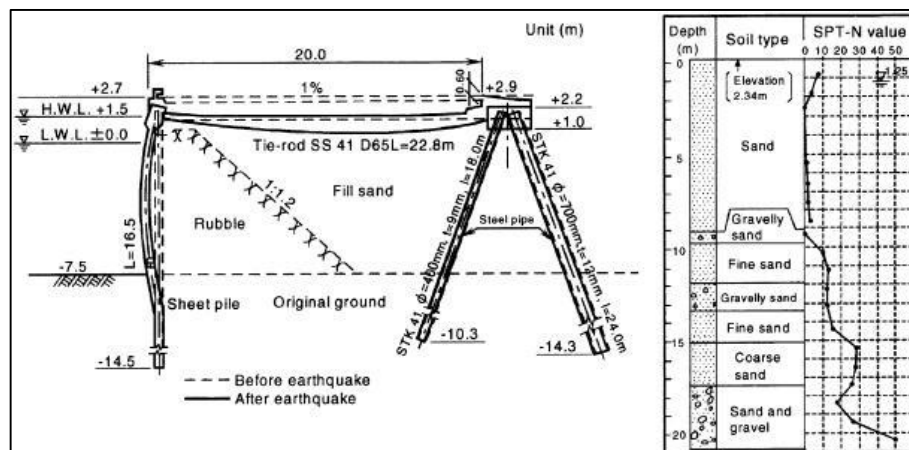


Figure 2.8 Sheet pile wall at the Port Kushiro, Japan (PIANC, 2001)

The backfill soil behind the sheet pile was loosely deposited sand layer. Due to liquefaction during the earthquake, deformation of the sheet pile is shown in Figure 2.8. Liquefaction of the backfill resulted in crack openings in the sheet wall at an elevation of 4 m below the water surface (PIANC, 2001). The horizontal permanent

displacement was 0.4 m at the top of the wall. The vertical displacement varied between 0.1 m and 0.3 m.

The southern quay wall in West Port District did not suffer damage during the same earthquake. The height of the sheet pile above the seabed and water depth are 15 m and 12 m, respectively as presented in Figure 2.9.

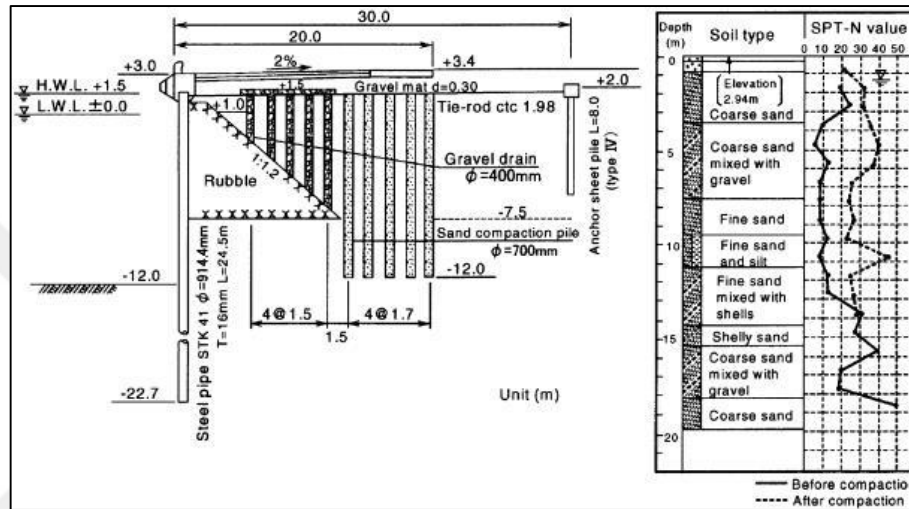


Figure 2.9 Ground improvement at the Port Kushiro, Japan (PIANC, 2001)

The gravel mats, gravel drains and sand compaction piles were used in order to prevent liquefaction in the backfill in this port (Borg, 2007). The sand compaction piles have been provided to densify and to increase shear strength of the backfill. The gravel drain which is a well-known compaction method might have reduced deformation of the structures. The walls with remedial measures against liquefaction did not suffer, so the permanent horizontal displacement, vertical displacement or tilting were not recorded (Kokusho, 2009).

2.2.6 Port of Kobe, Japan

About 90% of the quay walls are caisson type in the Kobe Port. This port suffered damage during the Great-Hanshin Earthquake, also known as the Hyogoken-Nanbu Earthquake or Kobe Earthquake on 17th January 1995.

The cross section of a gravity wall that suffered damages in Kobe Port is illustrated in Figure 2.10. The wall height and depth of water were measured as 18 m and 14 m, respectively. Design seismic coefficient (k_h) for the wall was 0.10. The magnitude of the earthquake was $M_j=7.2$. The peak ground surface accelerations (PGA) at Kobe Port were 0.53g and 0.45g in the horizontal and vertical directions.

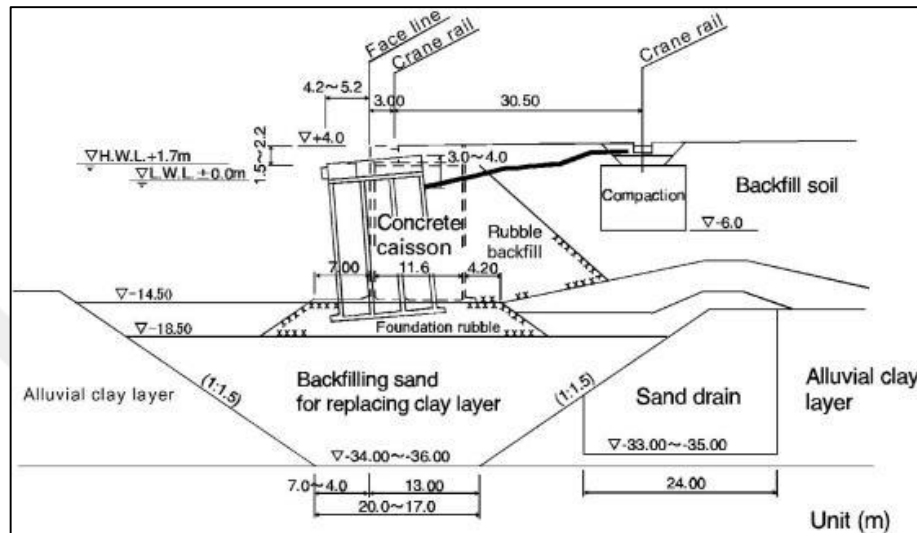


Figure 2.10 Port of Kobe, Japan (PIANC, 2001)

It was stated that when the level of the earthquake motion exceeded the limiting condition considered in the seismic design of the caisson type quay wall, the caisson wall moved toward the sea, causing the settlement in the soil behind the wall (Inagaki et al., 1996). This mechanism was exactly the major cause of damage to gravity quay walls in the Kobe and other earthquakes. The measured horizontal displacement following Kobe Earthquake varied between 4.2 m and 5.2 m. The vertical displacement was in between 1.5 m and 2.2 m. The wall tilted 4 to 5 degrees on the average. During the earthquake, backfill liquefied inserting a considerable influence on the performance of the quay wall. The backfilled sand beneath the wall foundation to replace the former clay layer excessively deformed since it was in loose state (Borg, 2007).

The three quay walls inside the Kobe Port, however, did not suffer damage during the earthquake. These quay walls were specifically built to tolerate the maximum credible earthquakes in this area to ensure a key function for the port immediately after

a large earthquake to unload medicine, food and other emergency supplies to the places which would be destroyed by the earthquake (Inagaki et al., 1996). High seismic resistant quay walls were designed with a seismic coefficient (k_h) as 0.25.

2.2.7 Derince Port, Turkey

Several ports on the Eastern Marmara Sea (Haydarpaşa Port, Tüpraş Oil Refinery Marine, Petrol Office Marine Terminal, Tuzla Port, Darıca Port, Derince Port and Yalova Port) suffered damages during the 1999 Marmara Earthquake ($M=7.4$). Derince Port which is the largest port in the area and the second most important container port in Turkey is located nearby Kocaeli city center.

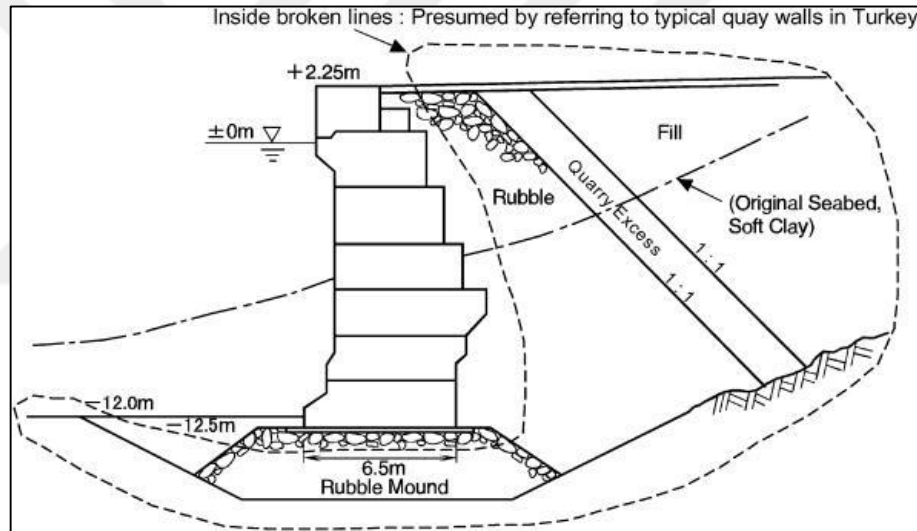


Figure 2.11 Derince Port, Turkey (PIANC, 2001)

Derince Port was the closest port to the earthquake epicenter. Therefore, the earthquake caused serious damage especially on gravity type quay walls. The backfill exhibited large settlements and this caused a crane rail to tilt and the wall displaced in horizontal direction permanently. The measured horizontal displacement was 0.5 m whereas vertical displacement varied between 0.5 m and 0.8 m. The wall moved seaward without any vertical displacement. Liquefaction of the backfill behind the wall was a serious problem. The Gravity Quay Wall No.6 inside the Derince Port is the major case history taken into consideration within the context of this thesis.

2.2.8 Port of Hitachinaka, Japan

The Port of Ibaraki is located 100 km northwest of Tokyo and in the central coastal area of Ibaraki Prefecture. The caisson wall suffered damage during Tohoku earthquake in Japan occurred on 11th March 2011. Tohoku earthquake is also named as the Great East Japan Earthquake. The cross section suffered during earthquake in Hitachinaka Port is shown in Figure 2.12.

The magnitude of the earthquake was $M_j=9$. The peak ground surface accelerations (PGA) in the earthquake reached up to 0.57g. The northern wharf area is Port of Hitachinaka which has two types quay wall for water depths of 12 m and 14 m. The quay wall is concrete caisson wall. The walls were constructed on the surface of a stiff sandy stratum and backfilled with coarse-grained rock waste instead of the sandy and/or silty soil usually used as backfill material because of providing liquefaction resistance and reducing the earth pressure for caissons (Takahashi et al., 2012).

The northern wharf walls were damaged as a result of the strong motion such as subsidence in the backfill ground. The displacement of the caisson part of the walls was seaward by up to 56 cm. However, the quay walls were not completely destroyed because the non-liquefiable soil material prevented liquefaction in the backfill ground (Takahashi et al., 2012). It is understood that the choice of backfill material is very important.

2.2.9 Akita Port, Japan

The port of Akita is located in the Akita City. This seaport is on the northeast part of the Japan Sea coast of Akita Prefecture. The Japanese government classifies this port as a Major Port.

The sheet pile quay wall suffered damage during Nihonkai-Chubu earthquake in Japan on 26th May 1983. The total height of sheet pile above the seabed and water depth is 12 m and 10 m, respectively as it is displayed in Figure 2.13.

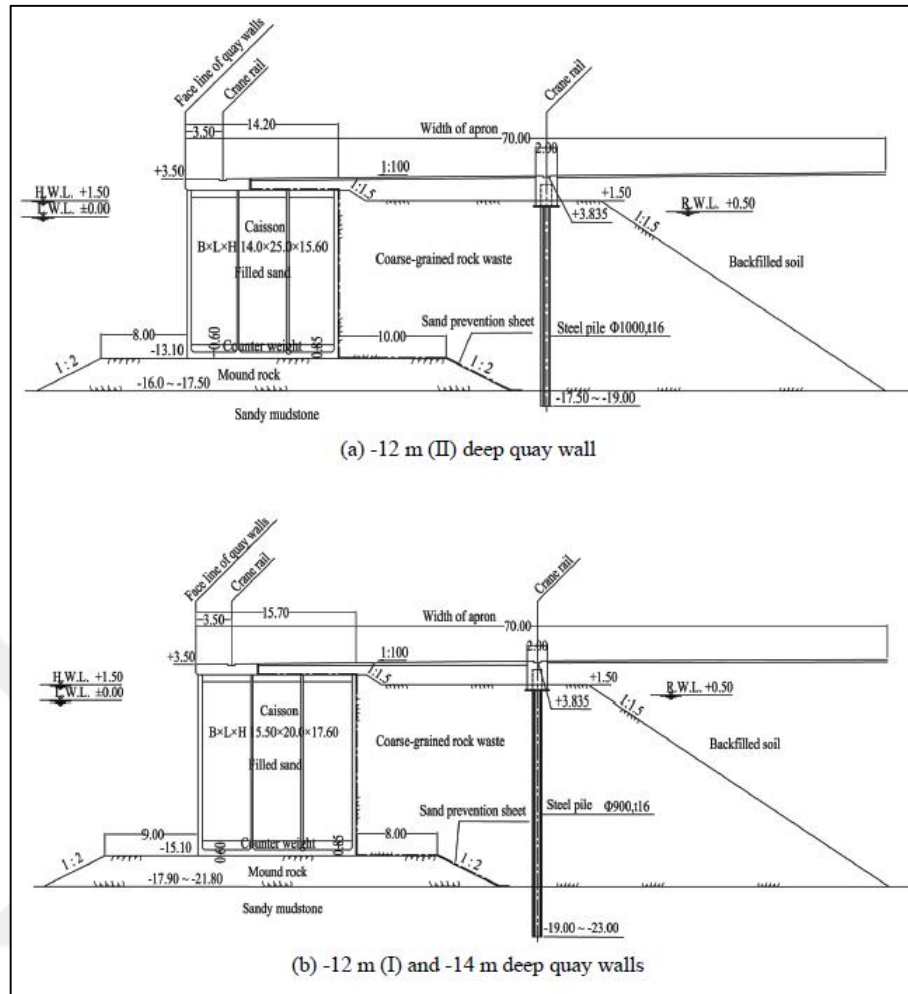


Figure 2.12 Cross-section of quay walls in the northern wharf area (Takahashi et al., 2012)

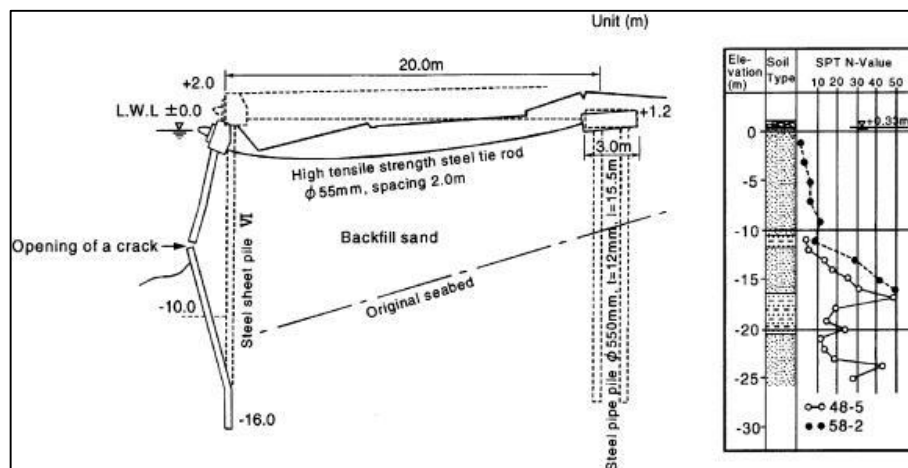


Figure 2.13 Ohama 2 at the Akita Port, Japan (PIANC, 2001)

The earthquake had a magnitude of $M_j=7.7$ (JMA). The evaluated peak ground surface acceleration (PGA) was measured as 0.24g. Design seismic coefficient (k_h) of sheet pile quay wall adopted for the area was 0.1 (PIANC, 2001).

Ohama No. 2 sheet pile under this earthquake suffered significant damage because of liquefaction of backfill. The horizontal displacement is 2 m at the top of the wall. The vertical displacement is ranged between 0.1 m and 0.3 m. Opening of cracks were observed on the sheet pile with tie rod as shown Figure 2.14.

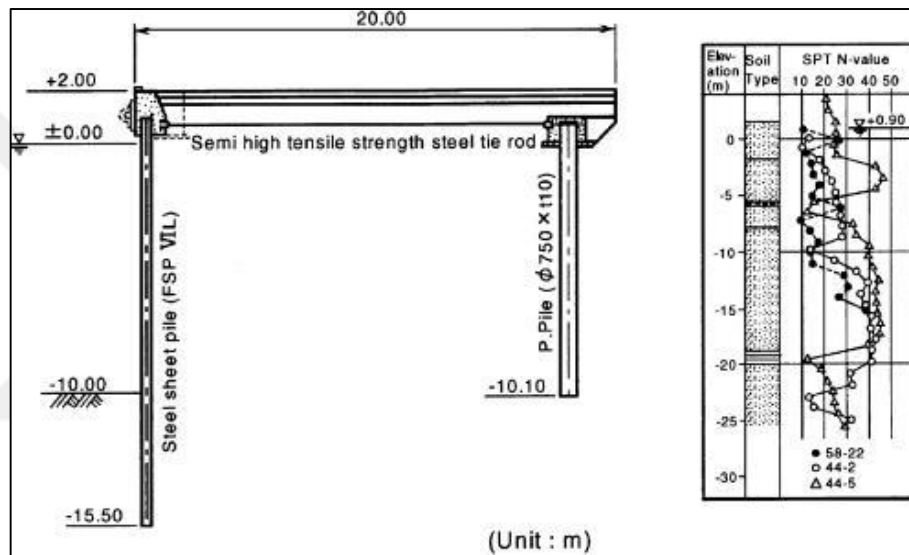


Figure 2.14 Ohama 1 at the Akita Port, Japan (PIANC, 2001)

However, Ohama No. 1 sheet pile, which is similar to that of Ohama No. 2, did not suffer from backfill liquefaction under the same earthquake. Thus, the permanent vertical and horizontal displacement were not observed as seen in Figure 2.13. It seems that occurrence or non-occurrence of backfill liquefaction resulted in the significant difference of seismic performance of sheet pile quay walls (PIANC, 2001).

2.2.10 Hakodate Port, Japan

Hakodate Port which is one of the main ports in northern Japan, is located in Hakodate on the northern island of Hokkaido. The height of sheet pile above the

seabed and water depth are 10.3 m. and 8 m., respectively as it is presented in Figure 2.15.

The sheet pile quay wall suffered damage during Hokkaido-Nansei-Oki earthquake which measured a magnitude of $M_j=7.8$ in Japan on 12th July 1993. Based on strong motion earthquake record, the evaluated surface peak ground surface acceleration (PGA) was measured as only 0.12g. Design seismic coefficient (k_h) of sheet pile quay wall is 0.15.

In spite of the low value of PGA, in order of 0.1g, the wall deformed due to backfill liquefaction and foundation soil liquefaction in the Port of Hakodate in Japan (Figure 2.16). When the liquefaction occurred in the backfill ground, the damages of quay wall were large. Briefly, the degrees of the damage are associated with occurrence of liquefaction in the backfill (Miura et al., 2000). The maximum permanent displacement was 5.2 m. The vertical displacement was 1.6 m. and the resulting tilt degrees were 15 (PIANC, 2001).

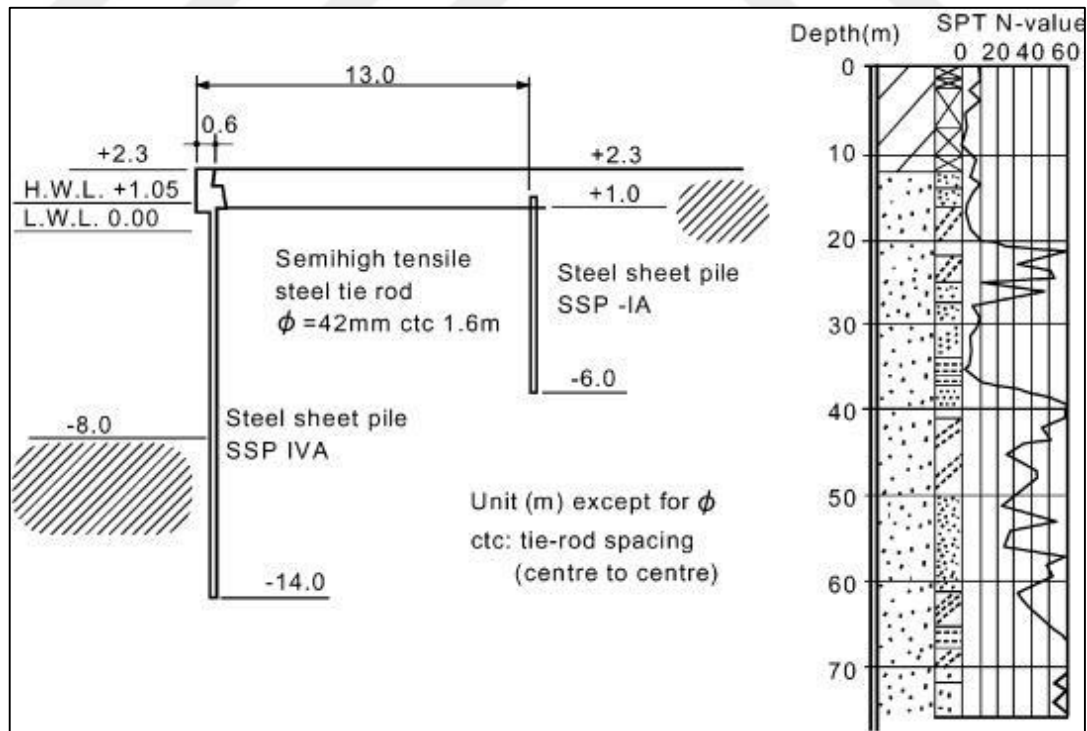


Figure 2.15 Hakodate Port, Japan (PIANC, 2001)



Figure 2.16 The sand boiling due to liquefaction at Hakodate Port, Japan (PIANC, 2001)

2.2.11 Commercial Port, Guam, USA

The commercial berths F3 to F6 of the port at Guam in the USA suffered damage during Guam earthquake on 8th August 1993. The height of the wall from seabed and the water depth were 13.2 m and 10.5m, respectively. The sheet pile quay wall was constructed using a three-step dredging sequence as it is seen in Figure 2.17. Guam earthquake magnitude measured as 8.1. Based on strong motion earthquake record, the evaluated surface peak ground surface acceleration (PGA) varied between 0.15 and 0.25g. The permanent horizontal displacement occurred 0.6 m due to earthquake (PIANC, 2001).

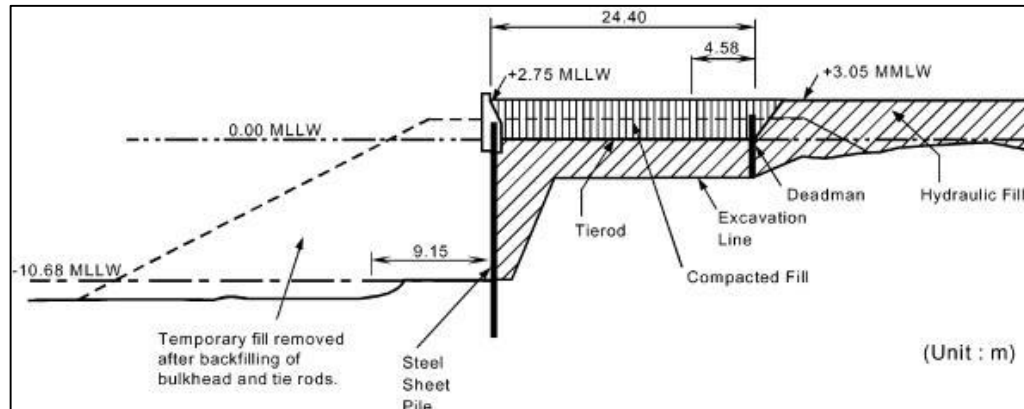


Figure 2.17 Commercial Port, USA (PIANC, 2001)

2.2.12 Hakata Port, Japan

The Fukuoka earthquake in 2005 was damaged harbour and fishing port facilities at the coast side of Hakata Bay in Fukuoka city. Hakata Port is one of them.

The quay wall was a sheet pile type and was designed -7.5 m water depth at Hakozaki-wharf in Hakata Port. Figure 2.18 show the location of No.1 quay (4 Berth, L=520 m) anchored by steel piles and No.2 quay (1 Berth, L=130 m) anchored by steel tubular piles. The cross section of No.1 quay is shown Figure 2.19.

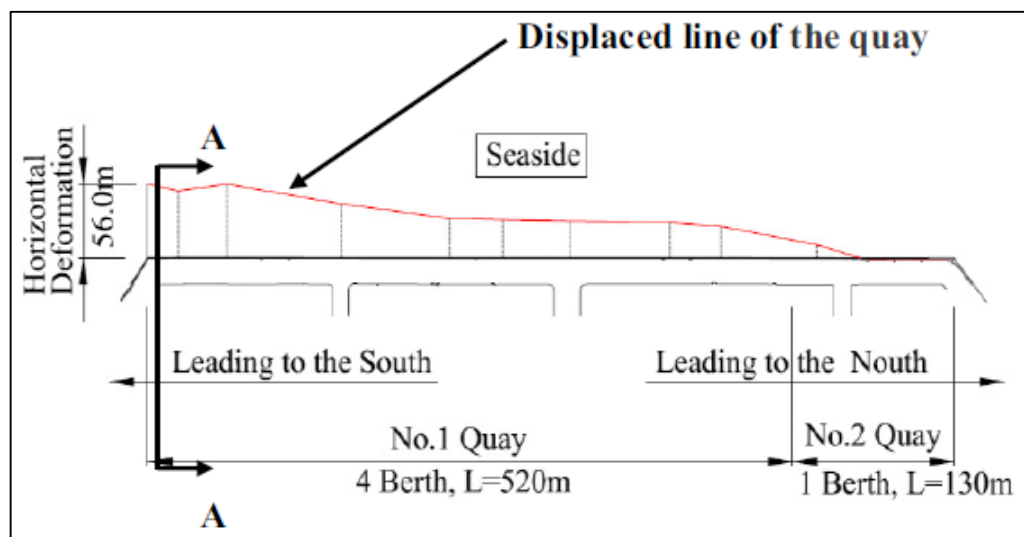


Figure 2.18 The location of No.1 and No.2 quay, Japan (Hamamoto et al., 2016)

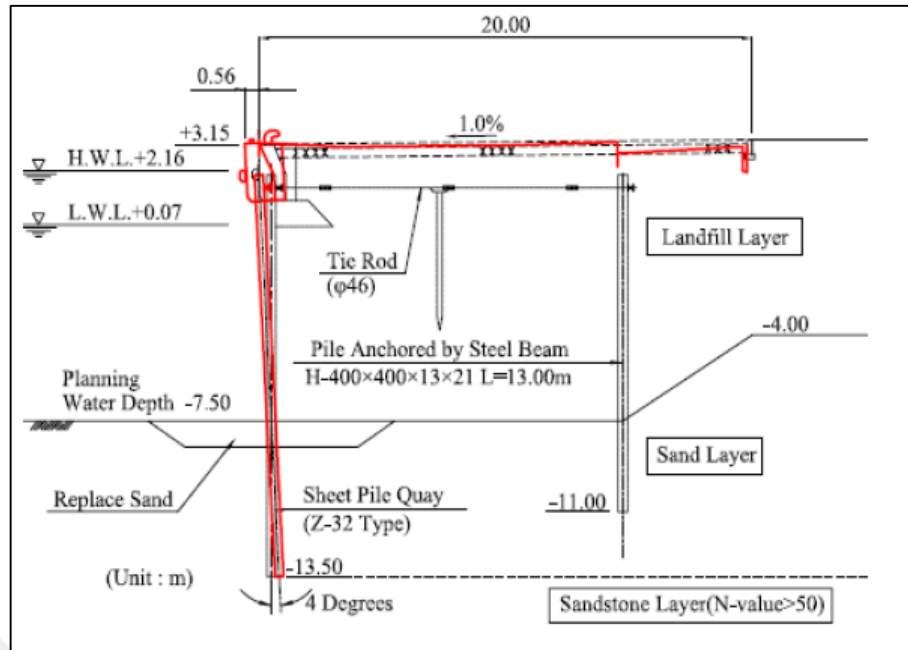


Figure 2.19 The cross section of the No.1, Japan (Hamamoto et al., 2016)

Fukuoka Earthquake's magnitude was measured as 7. After the earthquake, the red line in Figure 2.19 is shown the displaced position to the seaside at the top of the quay wall (Hamamoto et al., 2016). The No.1 quay wall moved the maximum displacement of 56 cm the horizontal direction. The sheet pile wall was inclined at 4 degrees to the inside (Hamamoto et al., 2016). The sand boiling phenomena was occurred, and the front and back apron was damaged due to liquefaction (Figure 2.20 a-b). The sand boiling phenomena just beneath the apron due to liquefaction was noticed. It was recorded that the liquefaction affected the apron at the sheet pile quay wall significantly (Figure 2.21).

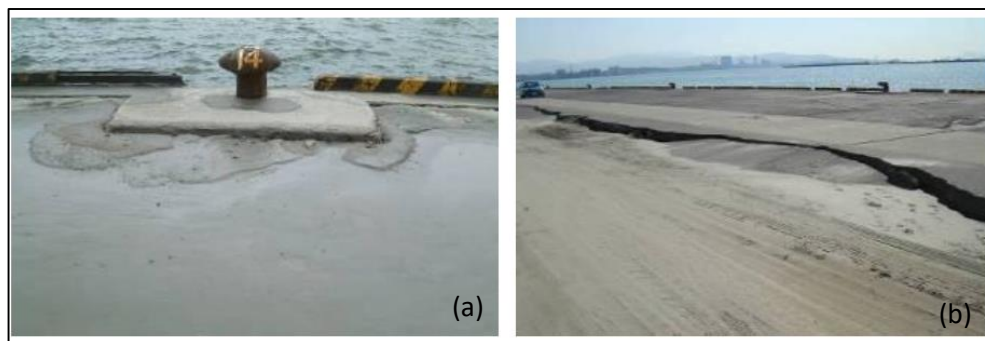


Figure 2.20 The sand boiling due to liquefaction (a) at front apron; (b) at back apron (Hamamoto et al., 2016)



Figure 2.21 Liquefaction behind quay wall (Hamamoto et al., 2016)

2.2.13 Port of Oakland, USA

Port of Oakland which was the first major container ship facilities on the Pacific coast of the USA to construct terminals or container ships is located in Oakland, California on San Francisco Bay.

The cross section of the 7th Street Terminal at the Oakland Port is pile supported wharf as shown in Figure 2.22. Square reinforced concrete piles which cross section was 41x41 cm was used to construct. The height of the wall above the seabed was 15.5 m. and the water depth was 11 m.

This wharf suffered damage during the Loma Prieta earthquake on 17th October 1989. The earthquake magnitude measured a magnitude of $M_s=7.1$. On the evaluation of strong motion records, the peak ground acceleration was in between 0.22 and 0.45g.

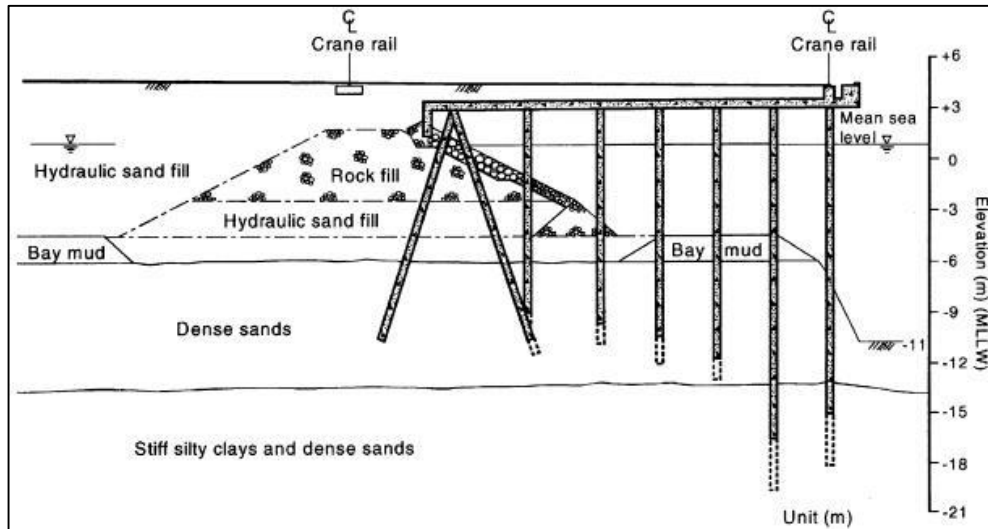


Figure 2.22 Port of Oakland, USA (PIANC, 2001)

At or near the tops of most of the rearmost line of batter piles, tensile failures were observed. In addition, liquefaction of backfill was also observed. The major geotechnical cause of damage is assumed as hydraulic sand fill below the dike (PIANC, 2001).

2.2.14 Port of San Fernando, Philippines

Port of San Fernando which is the commercial port on the north-west coast of Luzon is located in Philippines. This port is open type quay wall that is pile supported wharf pier as presented in Figure 2.23. This port constructed to be used coupled vertical and batter piles. The height of the wall above seabed was 20 m (Borg, 2007).

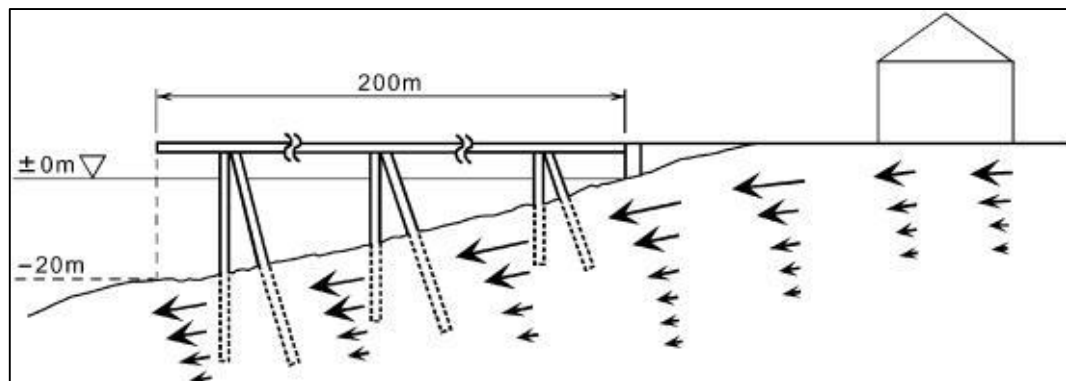


Figure 2.23 Port of San Fernando, Philippines (PIANC, 2001)

This wharf suffered damage during the Luzon earthquake on 16th July 1990. The Government Pier No.1 Port of San Fernando was damaged during earthquake that is magnitude of $M_s=7.1$. Horizontal displacement which is along the longitudinal direction toward of the pier the sea was observed in the port. It was observed many gaps in the longitudinal direction of the concrete deck which is near the entrance to the pier. The largest observation gap was 0.7 m. The sum of all the gaps were approximately 1.5 m at the seaside end of the pier. There were also cracks and chipping which were exposed reinforcement concrete steel bars that were either bent or cut at the pile caps (PIANC, 2001).

2.2.15 Port de Port au Prince, The North Wharf, Haiti

Port de Port au Prince is a main port that consists of two separate facilities designated as North Wharf and South Pier. The type of north wharf is pile supported wharf. The wharf length and width are 450 m and 20 m, respectively. The water depth is 8 to 10 m. Port de Port au Prince was damaged during earthquake which has magnitude of $M_s=7$. The North Wharf collapsed due to liquefaction-induced lateral spreading. The Figure 2.24 shows that liquefaction state is in the vicinity of the North wharf.



Figure 2.24 Port de Port au Prince, Haiti (USGS, 2010)

In Figure 2.24, light coloured areas indicate the sand boils. According to this figure, liquefaction can be traced in the eastern half of the container storage yard and behind and between the two warehouses.

2.2.16 Los Angeles Port, USA

The B126 (New) APL terminal in Los Angeles Port suffered damages during the Northridge earthquake of the 17th January 1994 (Borg, 2007). The water depth is 13.5 m and the total height 18 m as it is displayed in Figure 2.25. The type of wharf was the pile supported wharf which was on prestressed octagonal piles 61 cm in diameter.

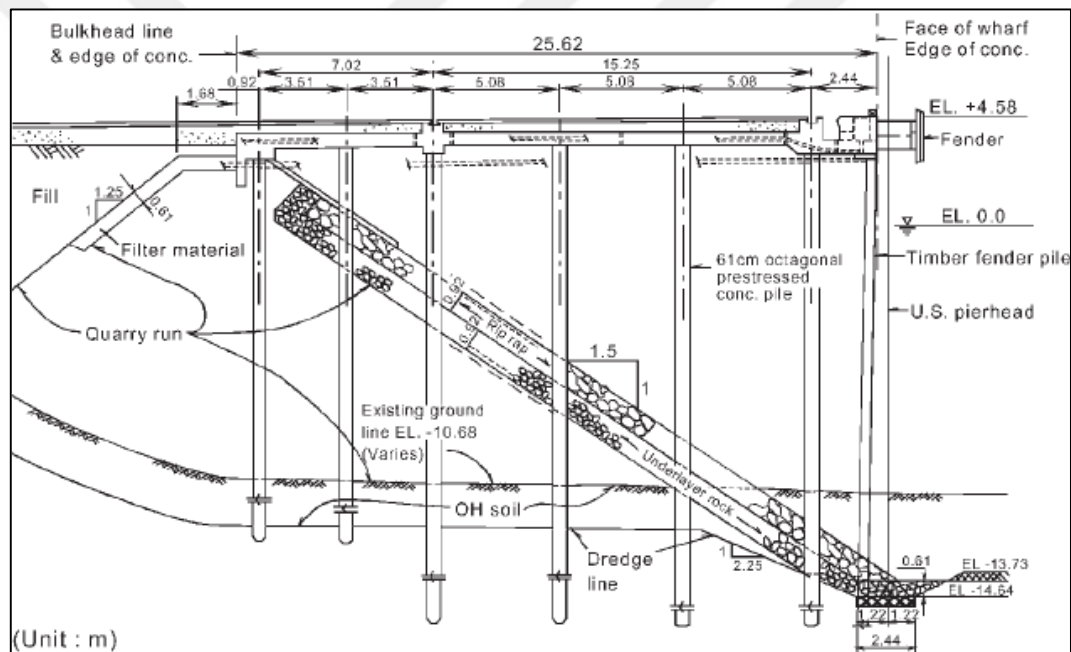


Figure 2.25 The B126 (New) APL terminal in Los Angeles Port (PIANC, 2001)

The earthquake magnitude measured a magnitude of $M_s=6.8$. The peak ground surface acceleration (PGA) which evaluated based on strong motion earthquake record was measured as 0.25g (PIANC, 2001). Design seismic coefficient (k_h) of pile supported wharf was 0.12. This wharf suffered damage because of liquefaction of backfill during earthquake. The horizontal displacement which was 0.8 cm at an expansion joint caused to damage to crane rails. The maximum horizontal displacement was 0.1 m (PIANC, 2001).

CHAPTER THREE

DESIGN METHODOLOGY

3.1 Introduction

In the design of coastal structures, seismic analyses have gained importance and performance-based design method has been emerged. The aim of performance-based design method is to overcome the limitation in the conventional seismic design. Conventional design is solely based on the force balance against a design level seismic force without providing information regarding what happens to the structure if the limiting condition is exceeded (Iai & Ichii, 1998). In the performance-based design approach, on the other hand, structural behavior is assessed considering deformations observed in the ground and the foundation (deformation; tilt; vertical and horizontal displacement and stress states).

As main principle under the effects of earthquakes, performance-based design is taken as a basis. In this design approach, under the specific levels of earthquake ground motions, the damage that potentially will occur in load bearing system elements is predicted numerically, and it is checked that whether this damage is below or not the allowable damage levels for each element. The allowable damage limits are defined as compatible with the performance objectives projected for structure in several earthquake levels. Since the damage of earthquake projected to be calculated at element level occur generally beyond the linear elastic limits in great earthquakes against nonlinear strains, performance-based design approach is more directly related to design concept than nonlinear analysis methods and strains. Mainly, for the performance objectives in which the damage is projected as limited, it is allowed to be used the linear analysis methods within the context of principle of conventional method (TDSC-CRA, 2007).

3.2 Definition of Performance Grades

In the seismic design, the performance of a structure against design earthquake motions must be defined clearly in terms of allowable displacements and stress levels (Iai & Ichii, 1998). In performance-based design, a structure is designed according to defined performance level. The design earthquake and performance levels must be defined before a structure is designed.

3.2.1 Design Earthquake Motion

The levels of design earthquake motions that are going to be taken as a basis in performance-based design of port structures are described in the three level approaches, as follows.

3.2.1.1 (L1) Earthquake Level

This earthquake level states the earthquake ground motions which have the highest probability to occur throughout the service life of the structures. They are relatively frequent, but they do not have severe magnitude. The probability of exceedance of earthquake in the level of (L1) in 50 years is 50%, and the return period corresponding to it is 72 years (TSDC-CRA, 2007).

3.2.1.2 (L2) Earthquake Level

This earthquake level states the earthquake ground motions which have not more probability to occur throughout the service life of structures. In this level, earthquake ground motions are rare but they have severe magnitude. The probability of exceedance of earthquake in (L2) in 50 years is 10%, and return period corresponding to it is 475 years (TSDC-CRA, 2007).

3.2.1.3 (L3) Earthquake Level

This earthquake level states the most severe earthquake ground motions that the structures will be exposed. The probability of exceedance of this very rare earthquake in level (L3) in 50 years is 2%, and return period corresponding to it is 2475 years (TSDC-CRA, 2007).

3.2.2 Classifications in Terms of Earthquake Performance

Coastal and port structures are classified as given below according to projected earthquake performance, purpose of use and importance it has.

3.2.2.1 Private Structures

The coastal and port structures included in private class are grouped as given below:

- Structures that have to be used immediately after earthquake for the purpose of immediate aid and emergency rescue
- Structures that are related to materials which have toxic, inflammable and explosive characteristics.

3.2.2.2 Normal Structures

The coastal and port structures included in normal class are grouped as given below:

- Structures that have to prevent the loss of life and property
- Structures that are important in terms of economy and social aspects
- Structures that are difficultly repaired and reinforced and will cause to loss of time after earthquake.

3.2.2.3 Basic Structures

The coastal and port structures included in basic structure are grouped as given below:

- Less important structures that are out of the private class and normal class
- Structures that are out of unimportant class

3.2.2.4 Unimportant Structures

The coastal and port structures included in unimportant class are grouped as given below:

- Structures than can be easily reconstructed,
- Structures that do not endanger the life safety even if it is damaged to the highest degree.
- Temporary structures

3.2.3 Damage Performance Level

The performance level of coastal and port structure are defined below depending on the expected damages occurred under earthquake effect. For these performance levels, allowable damage limits will be identified for each type of structure or element separately and numerically. The damage performance levels are defined as follows.

3.2.3.1 Minimum Damage Performance Level (MD)

Minimum damage performance level is the performance level which describes the case of no damage occurrence with earthquake effect or very limited structural damage that will occur in coastal and port structures and the elements constituted them. In this case, the operation of port is continuing uninterruptedly or disruptions, that will occur, stay in the level in which they can be fixed in a few days (TSDC-CRA, 2007).

3.2.3.2 Controlled Damage Performance Level (CD)

Controlled damage performance level is defined as performance level in which it is given permission to occurrence of damage that is not too much serious and repairable under the earthquake effect in the coastal and port structures and the elements

constituted them. In this case, the occurrence of short-time disruptions (few weeks or months) in the port operation of the related structure or element is normal (TSDC-CRA, 2007).

3.2.3.3 Advanced Damage Performance Level (AD)

Advanced damage performance level (AD) represents the advanced extensive damage which occurred before collapse failure under the earthquake effect in the coastal and port structures and elements constituted them. In this case, the occurrence of long-time disruptions in port operation of related structure or element and even completely cancellation of related port service is possible (TSDC-CRA, 2007).

3.2.3.4 Collapse Failure Damage Case (CFD)

In this case, full collapse failure damage occurs under earthquake effect in the coastal and port structures and the elements constituted them. The port operation of related structure or element is not going to be continued (TSDC-CRA, 2007).

The damage performance levels projected on several earthquake level and class of structure are summarised below Table 3.1.

Table 3.1 Performance levels projected on several earthquake levels, (TSDC-CRA, 2007)

Class of Structure	(L1) Earthquake Level	(L2) Earthquake Level	(L3) Earthquake Level
Private	-	MD	CD
Normal	MD	CD	(AD)
Basic	CD	(AD)	-
Unimportant	(AD)	(CFD)	-

3.3 Analysis Method

The performance-based design method is classified into three types: simplified analysis, simplified dynamic analysis and dynamic analysis. In this thesis, simplified analysis and dynamic analysis were taken consideration.

3.3.1 Simplified Analysis

The engineer may be able to evaluate the threshold displacement limits either permanent or elastic following the Simplified Analysis procedure for a seismic event (PIANC, 2001). The simplified analysis is based on pseudo-static approach. In this approach, a seismic coefficient, stated in terms of acceleration of gravity, is used to calculate inertia force which is used in analysis and design.

3.3.1.1 The Design of Gravity Type Quay Walls under Earthquake Effect

Gravity type quay walls are classified as reinforced concrete caisson type walls, reinforced concrete L type walls with buttress and without buttress, block type walls, cellular type concrete blocked walls and cast-in place concrete walls. In this study, block type quay wall was scrutinized.

Horizontal and vertical load combinations that will be taken into account under earthquake effect are defined below:

$$\text{Horizontal loads} = (F_{dw}) + (F_s) + (F_{de}) + 0.5(F_{ss}) + 0.5(F_{ds}) + (F_h) + 0.5(B) \quad (3.1)$$

$$\text{Vertical loads} = (W_b) + (W_s) \quad (3.2)$$

3.3.1.2 Static Horizontal Forces

The static horizontal forces are active earth forces, surcharges and hydrostatic forces.

3.3.1.2.1 Active Earth Force. Active earth pressure coefficient which is the ratio of horizontal and vertical principal effective stresses is found using Mononobe-Okabe equation. After calculating the active earth pressure coefficients, static earth pressures and earth forces are calculated for the crown wall as well as for each block. The related equations are defined below.

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2(\theta) \cos(\theta + \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\cos(\delta + \theta) \cos(\beta - \theta)}} \right]^2} \quad (3.3)$$

$$P_A = \frac{1}{2} K_A \gamma H^2 \quad (3.4)$$

$$F_{i_se} = \frac{(P_{i_se} + P_{i-1})}{2} h_i \quad (3.5)$$

K_A : active earth pressure coefficient

ϕ : friction angle

δ : friction angle between soil and block (this angle was taken as $\delta = \phi/3$)

β : inclination angle with horizontal

θ : slope of back of wall to vertical

P_A : active earth pressure

P_{i_se} : static earth pressure

F_{i_se} : static earth force

h_i : the height

3.3.1.2.2 Surcharge. In this study, the surcharge distribution is accepted as uniform and is considered to specific distance from the top of the wall. Firstly, static surcharge pressures are calculated and then statics surcharge forces calculated related to static surcharge pressure for each block using following equations.

$$P_{i_ss} = q \times K_A \quad (3.6)$$

$$F_{i_ss} = P_{i_ss} \times h_i \quad (3.7)$$

P_{i_ss} : static surcharge pressure

F_{i_ss} : static surcharge force

3.3.1.2.3 Hydrostatic Forces. In this study, hydrostatic pressure is not considered due to water levels which is assumed as same on both sides of the wall are assumed as same.

3.3.1.3 Dynamic Horizontal Forces

The dynamic horizontal forces consist of active earth, surcharge, hydrodynamic and inertia forces.

3.3.1.3.1 Dynamic Active Earth Pressure Coefficient. Dynamic active earth pressure coefficient which is the ratio of horizontal and vertical principal effective stresses can be calculated with below equation:

$$K_{AE,T} = \frac{\cos^2(\phi - \psi - \theta)}{\cos(\psi) \cos^2(\theta) \cos(\theta + \delta + \psi) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)}} \right]^2} \quad (3.8)$$

$$P_{AE} = \frac{1}{2} K_A \gamma H^2 \quad (3.9)$$

- K_{AE} : dynamic active earth pressure coefficient
- ϕ : friction angle
- δ : friction angle between soil and block ($\delta = \phi/3$)
- β : inclination angle with the horizontal
- θ : slope of back of wall to vertical
- ψ : seismic inertia angle
- P_{AE} : total active thrust
- H : the height

Two parameters used for calculation of dynamic active earth pressure coefficient are obtained from Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007). According to the coordinate of the study area, the spectral acceleration at $T=0.2$ seconds (S_s) and the spectral acceleration at $T=1.0$ seconds (S_1) is defined. F_a values with the respect to soil classification are given in Table 3.2. The spectral acceleration value is calculated as below:

$$S_{ms} = F_a \times S_s \quad (3.10)$$

Table 3.2 The short period soil coefficient (F_a), (TSDC-CRA, 2007)

Soil Class	Spectral acceleration (g) ^a for $T=0.2$ sec				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	_{-b}	_{-b}	_{-b}	_{-b}	_{-b}
^a Interpolation is made to find S_s .					
^b Special geotechnical analysis and dynamic soil attitude analysis is made.					

Effective seismic coefficient is defined depended on probability of exceedance and earthquake motion. The probability of exceedance %50 for L1 earthquake motion is A_{10} and the probability of exceedance %10 for L2 earthquake motion is A_{20} . They are calculated by using below formulation:

$$A_{10} = 0.4 \times S_{ms} \quad (3.11)$$

$$A_{20} = 0.4 \times S_{ms} \quad (3.12)$$

Horizontal seismic coefficient is related to earthquake motions. It is calculated with the following equations.

$$\text{For L1; } k_h = (2/3) \times A_{10} \quad (3.13)$$

$$\text{For L2; } k_h = A_{20} \quad (A_{20} \leq 0.20) \quad (3.14)$$

$$k_h = (1/3) \times A_{20}^{(1/3)} \quad (A_{20} > 0.20) \quad (3.15)$$

Seismic inertia angle is calculated according to horizontal seismic coefficient with below equations. If the soil is partially submerged, the modified seismic coefficient is used.

$$\psi = \arctan(k_h) \quad (3.16)$$

$$k'_h = \frac{\sum_{j=1}^{ND} (\gamma_j h_j) + \sum_{j=ND+1}^N (\gamma_{sj} h_j) + q_0}{\sum_{j=1}^{ND} (\gamma_j h_j) + \sum_{j=ND+1}^N (\gamma_{bj} h_j) + q_0} \times k_h \quad (3.17)$$

$$\psi = \arctan(k'_h) \quad (3.18)$$

γ_j : dry unit weight of soil

h_j : height of the soil

ND : number of the dry soil

q_o : surcharge

γ_{bj} : submerged unit weight of soil

γ_{sj} : saturated unit weight of soil

3.3.1.3.2 Dynamic Surcharge Pressures and Dynamic Earth Forces. The surcharge load is taken into consideration as one half (i.e. $q/2$) in design. The weight is calculated by multiplying seismic coefficient. However, if the soil is submerged, modified seismic coefficient is used as multiplication. The formulations are given below:

$$P_{i_ds} = (q/2) \times K_A \quad (3.19)$$

$$F_{i_ds} = P_{i_ds} \times h_i \quad (3.20)$$

3.3.1.3.3 Hydrodynamic Forces. During earthquake, if there is free water in front of the wall, the cyclic dynamic loading on the wall shall be considered. The critical mode shows up when suction pressure acts on the wall. The hydrodynamic water pressure is calculated using Westergaard's expression as follows:

$$P_{dw} = (7/8) k_h \gamma_w \sqrt{H_w h} \quad (3.21)$$

$$F_{dw} = \left(\frac{p_{dwi} + p_{dw(i-1)}}{2} \right) h_i \quad (3.22)$$

3.3.2.3.4 Inertia Forces. The inertia force is calculated by using pseudo-static analysis by using Equation 3.21 as follows:

$$F_h = \left(\frac{a_h \times W}{g} \right) = k_h \times W \quad (3.23)$$

F_h : horizontal force

a_h : maximum gravitational acceleration

W : weight of the block/soil

g : the gravity acceleration

k_h : horizontal seismic coefficient

k'_h : modified horizontal seismic coefficient

The calculation of the weight of the soils contiguous to the block is used as unit volume weight according to dry and submerged of the soil. The buoyancy effect is considered during the calculation of the weight of the block and the weight of block is calculated by multiplied with the horizontal seismic coefficient without considering dry or submerged. However, when the weight of the soil is calculated, the horizontal seismic coefficient and modified seismic coefficient is changed based on soil condition like as dry or submerged. When the weights of soil or blocks are multiplied with seismic coefficient, the vertical forces convert to horizontal forces.

3.3.1.4 Stability Analysis in Gravity Type Quay Walls

Under the identified load combinations, investigation of stability will be done for gravity type quay walls. In the pseudo-static approach, the stability of gravity quay walls is examined as sliding, overturning and bearing capacity. When the wall has a large width to height ratio, sliding factor is critical condition. On the other hand, when the wall has a small width to height ratio, overturning and loss of bearing capacity is more critical condition than sliding. Under the strong shaking, the effect of overturning or bearing capacity is much more serious than sliding because the wall is tilted and will be lead to collapse. The coefficient of safety of gravity type quay walls against sliding F_{ss} is calculated with the following equation.

$$F_{ss} = \frac{\mu W}{P} \quad (3.24)$$

Here, W and P represent the resultant vertical and horizontal loads taken into consideration and μ represent the coefficient of bottom friction that described in Table 3.3.

Table 3.3 The coefficients of bottom frictions in gravity type quay walls, (TSDC-CRA, 2007)

Friction Surfaces	Coefficient of Bottom Friction
Concrete-concrete	0.5
Concrete-bedrock	0.5
Underwater concrete-base rock	0.7-0.8
Concrete - rubble mound	0.6
Stone fill – rubble mound	0.8

The coefficient of safety against sliding F_{ss} will be at least 1.2 for Minimum Damage (MD) Performance Level, and at least 1.0 for Controlled Damage (CD) Performance Level as shown in Table 3.4.

Table 3.4 Factor of safety against sliding and overturning according to performance level (TDSC-CRA, 2008)

Factor of Safety	Minimum Damage	Controlled Damage
Sliding	>1.2	>1.0
Overturning	>1.3	>1.1

The coefficient of safety of gravity type quay walls against overturning F_{so} is calculated with the following equation.

$$F_{so} = \frac{We}{M_d} \quad (3.25)$$

Here, e shows the horizontal distance of resultant vertical load that is taken into consideration to toe point at the sea side of the wall, and M_d shows the sum of moments of considered horizontal loads that are taken according to wall base, in other words, it shows the total overturning moments.

In the calculation of M_d , overturning moments coming from dynamic earth force and dynamic surcharge force will be increased at the rate of 50% (TDSC-CRA, 2007). The coefficient of safety against overturning F_{so} will be at least 1.3 for Minimum Damage (MD) Performance Level and at least 1.1 for Controlled Damage (CD) Performance Level as shown in Table 3.4.

The pseudo-static analysis calculation for L2 earthquake level and peak ground acceleration obtained from Kocaeli Station during 1999 Marmara Earthquake is given in the Appendix I and Appendix II.

3.3.2 Dynamic Analysis

When an earthquake happens, it causes shaking of the ground through the seismic waves that spread from source till surface of the ground. Hence, an earthquake leads to different effects such as landslides, structural damages, tsunamis and soil liquefaction. Among these effects, soil liquefaction leads to several significant problems, as well as the other effects do. Liquefaction is a term which is used to define a several phenomena which appear in saturated cohesionless soils under drained conditions. Under the conditions of static and cyclic loading, densification is more likely to be observed in dry cohesionless soils. When dry cohesionless soils become saturated and the applied load begins as in the case of an earthquake, the propensity of densification leads to an increase in the pressures of excess pore water which cannot be decreased in rapid way and so leads to a decrease in the effective stresses (Plaxis, 2019). In all these processes, the soil acts as a fluid and the soil loose its strength.

When the major earthquake occurs, one of the main reasons behind the soil structure damage in saturated condition is explained with liquefaction (Galavi, Petalas & Brinkgreve, 2013). Liquefaction causes significant damages in marine structures, as well. For instance, when the recent strong earthquakes such as Chile (1985), USA (1989), Japan (1995), Turkey (1999), Taiwan (1999) and Haiti (2010) are taken into consideration, it can be readily seen that liquefaction have caused damages in marine structures.

Recent years, several researches have conducted several studies on liquefaction problems. These studies have particularly concentrated on liquefaction behavior in models during earthquake and prevention of potential negativities that will happen after liquefaction. The studies are focused to understand how the soil behavior can be analyzed with finite elements method under dynamic loading. Today, the liquefaction behavior can be modeled using UBC3D-PLM (University of British Columbia 3D-Post Liquefaction Model) model which is within the structure of Plaxis and defined as a user defined model.

Several researchers have also continued to examine UBC3D-PLM material model in several geotechnical problems. Consequently, UBC3D-PLM model is added into material model manual and detailed information about the use and calibration of the model is provided in 2018 Plaxis version due to increase the use of UBC3D-PLM.

When the conducted researches examined, the studies of Galavi and Petalas (2013) and Makra (2013) played a significant role to development of the UBC3D-PLM material model. It can also be readily said that the process of using UBC3D-PLM has started with Petalas and Galavi (2013). Petalas and Galavi gave detailed information about model formula and presented a report regarding calibration and implication of the model in their study. As a result of their studies, they stated that UBC3D-PLM model can model the residual excess pore water pressures occurred under low environmental pressure better than residual excess pore water pressures occurred under high environmental pressures (Galavi & Petalas, 2013).

Galavi et al. (2013) conducted a case study and they modeled a quay wall which is located in the region of Kobe in Japan, on the Rokko Island using UBC3D-PLM material model in Plaxis. From the analysis results, they obtained the change of vertical and horizontal displacements which are made by top corner point of the wall and it was compared with site report. It is observed that the obtained results from numerical simulation were same within the range of displacements in the recorded site report. Additionally, by changing the boundary conditions of the model like as free field boundaries and tied degrees of freedom, they have been examined the effects of boundary conditions on the change of horizontal displacement (Galavi et al., 2013).

Makra (2013) numerically modeled the San Fernando barrage in California with the UBC3D-PLM model in his thesis. It focused the extent of how UBC3D-PLM model can model the beginning of liquefaction and whether the model can predict the deformations after liquefaction. The findings of the study indicated that UBC3D-PLM model was successful at the onset of liquefaction. However, it was noted that UBC3D-PLM model cannot handle the behavior after liquefaction in a better way (Makra, 2013).

3.3.2.1 UBC3D-Post Liquefaction Model (PLM)

UBC3D is described as an extension of the 2D UBCSAND model which was developed at University of British Columbia by Pueble et al. (1997) and Beaty & Byrne (1998). The UBC3D-PLM which is defined as user defined soil model is a 3-D extension of the UBCSAND model. Accordingly, the UBC3D-PLM model utilizes yield criterion of the Mohr-Coulomb in a three-dimensional way. The UBC3D-PLM model is defined as an effective stress elasto-plastic model which is able to stimulate the liquefaction behavior of sands and silty sands under seismic loading (Petalas & Galavi, 2013; Tsegaye, 2010). In fact, the formulation of the UBC3D-PLM model is based on the original UBSAND model developed by University of British Columbia Sand. However, the formulation of the model is developed and implemented the 3D formulation in PLAXIS.

Depending upon the original Duncan-Chang model, the original UBCSAND is a two-dimensional model which is formulated with a hyperbolic strain hardening rule in the classical plasticity theory (Plaxis, 2019). At a given stress level, the hardening rule discloses the mobilized friction angle to the plastic sheer strain. As stated above, the UBCSAND model includes a two-dimensional Mohr-Coulomb yield surface and an equivalent non-associated plastic potential function. The flow rule in the model is depending upon the stress-dilatancy theory which was developed by Rowe (1962) and it is linearized and simplified in accordance with energy considerations (Plaxis, 2019).

The essential difference between the UBCSAND and UBC3D-PLM models is generalized with 3D formulation for the UBC3D-PLM model. As stated above, the UBC3D-PLM model adopts the yield condition of Mohr-Coulomb in a three-dimensional principal stress space with respect to primary loading, and a yield surface having a simplified kinematic hardening rule with respect to secondary loading (Plaxis, 2019). Additionally, a modified non-associated plastic potential function depending upon criterion of Drucker-Prager which is utilized for the primary yield surface to keep

up the stress-strain co-axiality assumption in the deviatoric plane with respect to stress path that begins from the isotropic line (Tsegaye, 2010).

3.3.2.1.1 Yield Function. Two yield surfaces are utilized to model soils cyclic behaviour in the UBC3D-PLM. These two yield surfaces are primary and secondary yield surfaces. The primary yield surface is depending upon isotropic hardening and if the mobilised friction angle is corresponded to the maximum mobilised friction angle which the soil has ever touched, it evolves into active. In such a case, the current stress ratio is considered as the highest stress ratio that can be observed in the loading history (Galavi et al., 2013).

A simplified kinematic hardening rule is used for the secondary yield surface which turns into active, if the mobilised friction angle is lower than the maximum mobilised friction angle. Hence, it is also considered as a case in which the maximum stress ratio is bigger than the current stress ratio in the loading history. The distinction between these two yield surfaces lead to a densification rule which refers to higher rate of hardening in the secondary yield surface (Galavi et al., 2013)

The yield surfaces are displayed schematically in Figure 3.1. In order to describe the effect of primary and secondary yield surfaces, a stress state is accepted on the isotropic axis and both yield surfaces are at the identical position. According to Figure 3.1, as loading from isotropic stress, both two yield surfaces increase in accordance with the same hardening rule. When the soil is unloaded, the secondary yield surface becomes smaller and the soil behaviour becomes elastic. However, as the soil is reloaded, the secondary yield surface turns into activate and the behaviour will become elasto-plastic, again. When the mobilised friction angle attains the maximum mobilized friction angle, the primary yield surface turns into active again and the behaviour is getting softer (Galavi et al., 2013).

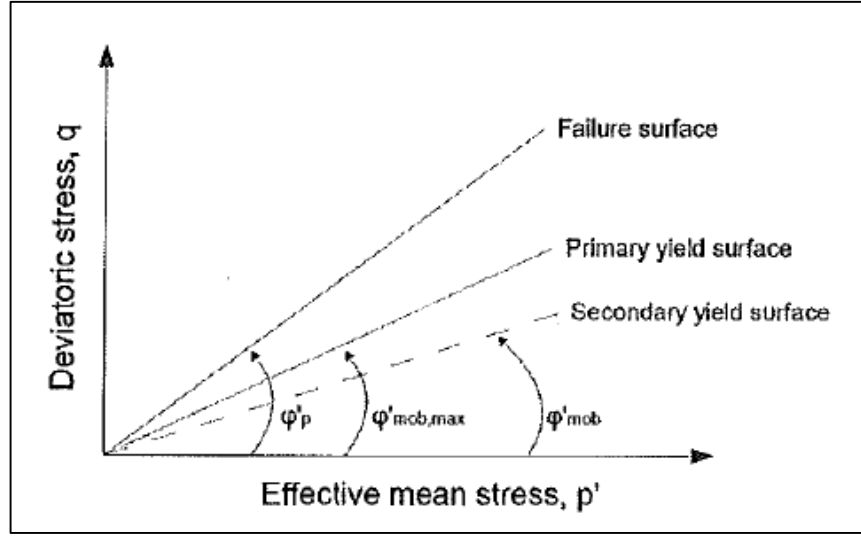


Figure 3.1 Yield surfaces (Galavi et al., 2013)

3.3.2.1.2 Elastic Behavior. The elastic behaviour can be observed within the secondary yield surface and it is directed by a stress depending upon non-linear rule which is identified by Puebla et al. (1997). The elastic behaviour is described in terms of the elastic bulk modulus, which is stated as K^e , and the elastic shear modulus which is stated as G^e . The elastic bulk modulus and the elastic shear modulus are depending on stress and are formulated with below equations:

$$K^e = k_B^{*e} * p_{ref} * \left(\frac{p'}{p_{ref}}\right)^{me} \quad (3.26)$$

$$G^e = k_G^{*e} * p_{ref} * \left(\frac{p'}{p_{ref}}\right)^{ne} \quad (3.27)$$

k_B^{*e} : the bulk modulus factors in UBC3D-PLM model

k_G^{*e} : the shear modulus factors in UBC3D-PLM model

p_{ref} : the reference pressure

me : stress dependency of elastic bulk modulus

ne : stress dependency of elastic shear modulus

During the unloading process, pure elastic behaviour is anticipated with the model. If the stress state attains the yield surface, the plastic behaviour is taken into consideration, as long as the stress point is not returning immediately into the elastic zone.

3.3.2.1.3 *Flow Rule*. The plastic strain direction is determined by the plastic potential function. In this context, a non-associated flow rule depending upon the Drucker-Prager plastic potential function is utilized in the UBC3D-PLM model (Tsegaye, 2010). This flow rule is given in the formulations below:

$$g = q - M * (p' + c * \cot \varphi_p) \quad (3.28)$$

$$M = \frac{6 * \sin \psi_m}{3 - \sin \psi_m} \quad (3.29)$$

g : the plastic potential function

ψ_{mob} : the mobilised dilation angle

q : the deviatoric stress

p' : the mean effective stress

Initiated by this equation, it is concluded that in point of compression, the Drucker-Prager surface is fixed. The mobilised dilatancy angle, ψ_{mob} , is calculated depending upon the flow rule adopted by Puebla et al. (1997) which is obtained from the stress dilatancy theory and linearized in accordance with energy considerations in following relation (Plaxis, 2019). Accordingly, the equation is formulated as follows:

$$\sin \psi_m = \sin \varphi_m - \sin \varphi_{cv} \quad (3.30)$$

φ_{cv} : the constant volume friction angle

φ_m : the constant mobilised friction angle

Initiated by the mobilises, friction angle and unloading-reloading criterion are described in the model given below:

$$\sin \varphi_{mob}^e < \sin \varphi_{mob}^0 \quad (\text{Unloading; elastic behaviour})$$

$$\sin \varphi_{mob}^e > \sin \varphi_{mob}^0 \quad (\text{Loading or reloading; plastic behaviour})$$

The previous mobilised friction angle, $\sin \varphi_{mob}^0$, is taken from the previous calculation step; whereas the current one $\sin \varphi_{mob}^e$ is computed on the basis of the current stresses. During loading, the friction angle is mobilised and the hardening

plasticity appears. However, during unloading, pure elastic behaviour is anticipated just before the stress point arrives the p' axis.

3.3.2.1.4 Hardening Rule. The hardening rule controls the plastic strain amount by reason of mobilisation of the shear strength ($\sin\phi_{mob}$). The mobilised friction angle is acquired from the yield criterion of Mohr-Coulomb. The hardening rule transfers the increment of the sine mobilised friction angle to the plastic shear strain increment and this is formulated with the equations given below:

$$d\gamma_p = \left(\frac{1}{G^*}\right) d\sin\phi_{mob} \quad (3.31)$$

$$G^* = k_G^{*p} \left(\frac{p'}{p_{ref}}\right)^{np} \left(1 - \left(\frac{\sin\phi_{mob}}{\sin\phi_p}\right) R_f\right)^2 \quad (3.32)$$

k_G^{*p} : the plastic shear modulus factors

np : the plastic shear modulus exponent

ϕ_{mob} : the mobilised friction angle

ϕ_p : the peak friction angle

R_f : failure ratio ranging from 0.5 to 1

η_f : the stress ratio at failure

η_{ult} : the asymptotic ratio from the best fit hyperbola

The hardening rule is reformulated by Tseguya (2010) in UBC3D-PLM model and this is given as:

$$d\sin\phi_{mob} = 1.5 * k_G^{*p} \left(\frac{p'}{p_{ref}}\right)^{np} \left(\frac{p_{ref}}{p'}\right) \left(1 - \left(\frac{\sin\phi_{mob}}{\sin\phi_p}\right) R_f\right)^2 d\lambda \quad (3.33)$$

$d\lambda$: the plastic strain increment multiplier

3.3.2.1.5 Load State During Liquefaction. The UBC3D-PLM model uses two yield surfaces in order to assure a smooth transition into the soil's liquefied state and to create the distinction between primary and secondary loading. A densification law is incorporated by the UBC3D-PLM model. A soil densification law is taken into

consideration to get higher accuracy in the predicted evolution regarding excess pore pressures. The secondary yield surface causes lower plastic deformations to the primary yield surface. An anisotropic and a simplified kinematic hardening rule were used for the primary yield surface and the secondary yield surface, respectively (Plaxis, 2019).

During primary loading, the plastic shear modulus factor which is indicated as k_G^{*p} , is equal with the one entered as input parameters, and it is used in the hardening rule which directs the hardening of the primary yield surface. On the other hand, during secondary loading, the plastic shear modulus factor, k_G^{*p} , is considered as a function of the number of cycles followed during the loading process. This is the rule adopted for defining the counting cycles. This paves the way for increase of the excess pore pressures concurrently with an activity of undrained cyclic loading with a decreasing rate as far as the liquefied state is reached (Plaxis, 2019). This is calculated with the equation given below:

$$k_{G,secondary}^{*p} = k_G^{*p} \left(4 + \frac{n_{rev}}{2} \right) hard f_{dens} \quad (3.34)$$

k_G^{*p} : the input plastic shear modulus factor

$k_{G,secondary}^{*p}$: the secondary plastic shear modulus factor

n_{rev} : the number of shear stress reversals from loading to unloading or vice versa

$hard$: the factor which is correcting the densification rule for loose soils

f_{dens} : the multiplier which is a user input parameter to adjust the densification rule

For the densification rule for loose soils, a correction is also made ($5 < (N_1)_{60} < 9$) with respect to the experimental observation. Hence, the UBCSAND model formulation which was suggested by Beaty and Byrne (2011) and reported by Naesgaard (2011) is given as follows:

$$hard = \min(1 ; \max(0.5; 0.1(N_1)_{60})) \quad (3.35)$$

The plastic shear modulus is limited with respect to maximum corrected SPT value $(N_1)_{60}$ of corresponding dense soils (Plaxis, 2019).

$$k_{G,max}^{*p} = k_G^{*p} (k_G^{*p} (max N_{1,60}^2) 0.003 + 100) \quad (3.36)$$

3.3.2.1.6 Post Liquefaction Rule and Cyclic Mobility. A significant point during the cyclic liquefaction modelling in sands is the volumetric locking. The evolution of the plastic volumetric strains occurred after the stress path attains the yield surface is described with the peak friction angle which evolves into constant because of the formulation of the flow rule. In this case, $\sin\phi_{mob}$ turns into $\sin\phi_p$ and stays constant which means that $\sin\psi_m$ is constant, as well (Plaxis, 2019).

As a result, it is not possible to model the loose non-cohesive soils' stiffness degradation which occurs because of the post liquefaction behaviour, besides the dense non-cohesive soils because of the cyclic mobility. This limitation is defeated with the UBC3D-PLM model formulation with the equation that gradually diminished the plastic shear modulus as a function of the generated plastic deviatoric strain in the course of dilation because of the deconstruction of the skeleton of soils that appears concurrently with dilative behaviour. This causes soil stiffness degradation while contraction phase takes place that following the unloading phase. The process of cyclic mobility for dense sand is given in Figure 3.2 (Plaxis, 2019). Additionally, the stiffness degradation which is mentioned is calculated as follows:

$$k_{G,post-liquefaction}^{*p} = k_G^{*p} E_{dil} \quad (3.37)$$

$$E_{dil} = \max(e^{-110\varepsilon_{dil}}; f_{Epost}) \quad (3.38)$$

k_G^{*p} : the input plastic shear modulus factor

$k_{G,post-liquefaction}^{*p}$: the plastic shear modulus factor during liquefaction

ε_{dil} : plastic deviatoric strain due to dilation

The minimum value of E_{dil} term mentioned in the equation given above is limited by the input parameter f_{Epost} .

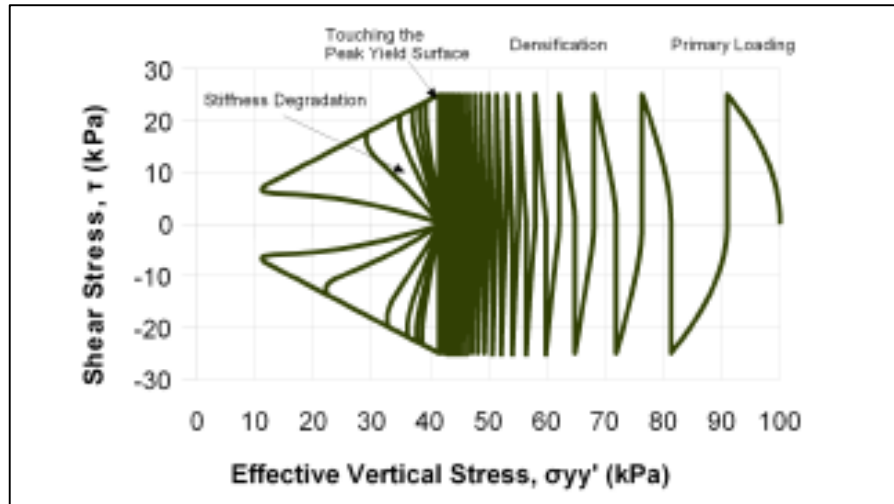


Figure 3.2 The process of cyclic mobility for dense sand (Plaxis, 2019)

3.3.2.1.7 Dynamic Boundary Conditions. In order to limit the boundaries of the FE model dynamic boundary conditions are imposed to imitate the infinite soil boundaries of the field. Waves that would not be present in the earthquake response in the field are generated along the boundaries of FEM model as a result of its finite nature. Therefore, absorbing boundaries are placed along the edges of the model in order to avoid wave reflection that causes artificial wave motion. Decision making process regarding the details of the boundary conditions, however, depends on the nature of the problem, accuracy and stability of the boundary condition (Galavi et al., 2013).

3.3.2.1.8 Parameters of the UBC3D-PLM Model. The input parameters for the UBC3D-PLM model are given in this section. In earthquake engineering, if the modelling target is the onset of liquefaction, all the parameters for the UBC3D-PLM model can be extracted with a cyclic triaxial or a cyclic direct simple shear test. In addition to this, obtained data from drained triaxial tests (CD txC) or in-situ test are also suitable. Finally, some equations are derived from parameters and published by Beaty and Byrne (2011) for the initial calibration of the model. The correlations for input parameters are based on SPT values after calibration of the UBCSand model. The input parameters separated as stiffness parameters, strength parameter and advanced parameters. The input parameters according to their categorizations are summarized below:

Stiffness parameters:

k_B^e	: Elastic bulk modulus of the soil	[-]
k_G^e	: Elastic shear modulus of the soil	[-]
k_G^p	: Plastic shear modulus of the soil	[-]
m_e	: Elastic bulk modulus index	[-]
n_e	: Elastic shear modulus index	[-]
p_{ref}	: Reference pressure	[kN/m ²]

Strength parameters:

ϕ_{cv}	: Constant volume friction angle	[°]
ϕ_p	: Peak friction angle	[°]
c	: Cohesion	[kN/m ²]
σ_t	: Tension cut off and tensile strength	[kN/m ²]

Advanced parameters:

R_f	: Failure Ratio	[-]
$(N_1)_{60}$	: Corrected SPT value	[-]
f_{dens}	: Densification factor	[-]
F_{Epost}	: Post-liquefaction factor	[-]

There are two drainage types that are allowed by the UBC3D-PLM model as drained and undrained A. The other undrained calculations are not used because of the effective stress nature of the model (Plaxis, 2019). The input parameters of the UBC-3D-PLM model in Plaxis are shown in Figure 3.3.

- Stiffness parameters

The equations based on the corrected SPT value, $(N_1)_{60}$, for the initial generic calibration of the UBCSand 904aR model are proposed by Beaty and Byrne (2011). The proposed equations by Byrne are modified by Makra (2013). The differences between the original UBCSand and the UBC3D-PLM formulation were highlighted the modified equations of stiffness modulus are given below.

$$k_G^{*e} = 21.7 \times 20 \times (N_1)_{60}^{0.3333} \quad (3.39)$$

$$k_B^{*e} = 0.7 \times k_G^{*e} \quad (3.40)$$

$$k_G^{*p} = k_G^{*e} \times (N_1)_{60}^2 \times 0.003 + 100 \quad (3.41)$$

The index parameters which are m_e , n_e and n_p in the range between 0 and 1 should be calibrated by curve fitting. However, the default values are suggested for the user in Plaxis. The suggested value for the elastic bulk modulus index, m_e , and the elastic shear modulus index, n_e , is 0.5. Likewise, the suggested value for the plastic shear modulus index, n_p , is 0.4.

Additionally, the aforementioned parameters can be calibrated using relative density correlations as suggested by Souliotis and Gerolymos (2016). The reference pressure is standard value and accepted as 100 kN/m².

- Strength parameters

The strength parameters can be obtained directly using data of drained triaxial test (CD TxC) or direct simple shear test (DSS). In most cases, the soil is granular soil without relevant fine component so the cohesion, c , is entered as 0 in the program. The peak friction angle, ϕ_p , are calculated with the following equation. The constant volume friction angle, ϕ_{cv} , is calculated with the correlation available in literature that is based on SPT test. The tension cut off and tensile strength, σ_t , is accepted as 0 in the calculations.

$$\phi_p = \phi_{cv} + \frac{(N_1)_{60}}{10} + \max(0; \frac{(N_1)_{60}-15}{5}) \quad (3.42)$$

- Advanced parameters

The advanced parameters are corrected SPT value, $(N_1)_{60}$, the densification factor, f_{dens} , the post liquefaction factor, f_{Epost} , and the failure ratio, R_f , respectively.

The corrected SPT value is calculated by multiplying the SPT blow count (N_{SPT}) with the energy correction factor (C_e), the overburden correction factor (C_n), the rod length correction factor (C_r), the borehole diameter correction factor (C_b), and the liner correction factor (C_s). The formula of corrected SPT value is indicated below.

$$(N_1)_{60} = N_{30} \times C_e \times C_n \times C_r \times C_b \times C_s \quad (3.43)$$

The densification factor, f_{dens} , is a multiplier which varied between 0 and 1. If the value is below 1, the plastic shear modulus becomes lower and the behavior becomes softer. The densification factor value is recommended as 1 by Petalas and Garavani (2012) because the liquefaction triggering is not affected from this value.

The other parameter is post liquefaction behaviour, f_{Epost} . The acceptable range of the factor varied between 0 and 1, but also the recommended value is between 0.2 and 1. If there is a very dense sand which is balanced by an increase of the post liquefaction behaviour parameter, resistance can be neglected for this situation.

Although the default value of failure ratio, R_f , is suggested as 0.9, this value can be also calculated using SPT test as in the following equation.

$$R_f \approx 1.1 (N_1)_{60}^{-0.15} < 0.99 \quad (3.44)$$

Property	Unit	Value
Stiffness		
k_B^e		0,000
k_G^e		0,000
k_G^p		0,000
m_e		0,000
n_e		0,000
r_p		0,000
Strength		
φ_{cv}	°	0,000
φ_p	°	0,000
c	kN/m ²	0,000
σ_c	kN/m ²	0,000
Field data		
$(N_1)_{60}$		0,000
Advanced		
Set to default values ☒		
Stiffness		
f_{dens}		0,000
f_{Epost}		0,000
p_{ref}	kN/m ²	100,0
Strength		
R_f		0,9000
Undrained behaviour		
Undrained behaviour	Standard	
Skempton-B		0,000
v_u		0,4950
$K_{w,ref} / n$	kN/m ²	0,000

Figure 3.3 The input parameters of the UBC-3D-PLM model (Plaxis, 2019)

3.3.2.1.9 State Parameters in the UBC3D-PLM Model. The output parameters for the UBC3D-PLM model are given in this section. This model provides output of state parameters such as the excess pore pressure ratio in terms of vertical effective and mean stresses and the mobilised peak friction angle. In this study, the analysis results are discussed according to the excess pore water pressure ratio in terms of vertical stresses. The state parameters option is selected from Stresses menu.

3.3.2.1.10 UBC3D-PLM Model in Dynamic Calculations. “The UBC3D-PLM model has been developed for simulating the dynamic behaviour of non-cohesive soils and it is particularly suitable for analysing the problems involving generation of pore pressure in undrained behaviour and liquefaction” (Plaxis, 2019, p.108). The UBC3D-PLM model is improved to model cohesive soil under dynamic behavior so it is not

suitable in static analysis. So, the hardening soil material model is recommended to generate the initial stresses due to gravity loading instead of the UBC3D-PLM model. The Hardening Soil material model parameter is summarized below and presented in Figure 3.4.

Failure parameters as in Mohr-Coulomb Model:

c	: Cohesion (effective)	[kN/m ²]
φ	: Angle of internal friction (effective)	[°]
ψ	: Angle of dilatancy	[°]
σ_t	: Tension cut-off and tensile strength	[kN/m ²]

Basic parameters for soil stiffness:

E_{50}^{ref}	: Secant stiffness in standard drained triaxial test	[kN/m ²]
E_{oed}^{ref}	: Tangent stiffness in primary oedometer loading	[kN/m ²]
E_{ur}^{ref}	: Unloading/reloading stiffness ($E_{ur}^{ref} = 3E_{50}^{ref}$)	[kN/m ²]
m	: Power for stress-level dependency of stiffness	[kN/m ²]

Advanced parameters (it is advised to use the default setting):

ν_{ur}	: Poisson's ratio for unloading/reloading ($\nu_{ur} = 0.2$)	[-]
p^{ref}	: Reference stress for stiffness ($p^{ref} = 100$ kN/m ²)	[kN/m ²]
K_0^{nc}	: K_0 value for normal consolidation ($K_0^{nc} = 1 - \sin\varphi$)	[-]
R_f	: Failure ratio q_f/q_a ($R_f = 0.9$)	[-]
$\sigma_{tension}$	: Tension strength ($\sigma_{tension} = 0$ stress units)	[kN/m ²]

Soil - Hardening soil - <NoName>

General Parameters Groundwater Interfaces Initial

Property	Unit	Value
Stiffness		
E_{50}^{ref}	kN/m ²	0.000
E_{oed}^{ref}	kN/m ²	0.000
E_{ur}^{ref}	kN/m ²	0.000
power (n)		0.5000
Alternatives		
Use alternatives		☐
C_c		10.00E9
C_s		10.00E9
e_{init}		0.5000
Strength		
c'_{ref}	kN/m ²	0.000
ϕ' (phi)	°	0.000
ψ (psi)	°	0.000
Advanced		

Next OK Cancel

Figure 3.4 The input parameters of the Hardening soil model (Plaxis, 2019)

CHAPTER FOUR

A CASE STUDY: NO.6 GRAVITY QUAY WALL OF DERİNCE PORT

4.1 Introduction

Performance based assessment of gravity type quay walls was studied by means of a relatively well documented case history. The quay wall that is considered is in İzmit Bay, which is one of the most important industrial zones of Turkey in terms of both its geographic position and structural characteristics. Besides, İzmit Bay is safeguarded against environmental effects and it has a secure geography geopolitically, it is very close to İstanbul. Therefore, several coastal structures have been built along the coastline of this bay (Yüksel & Güler, 2005). Yüksel (2005) stated that İzmit Bay is of extremely vital importance for State of Turkish Republic because of its industrial potential. Ports, berthing and mooring structures serving for the industry in this area are displayed in Figure 4.1.

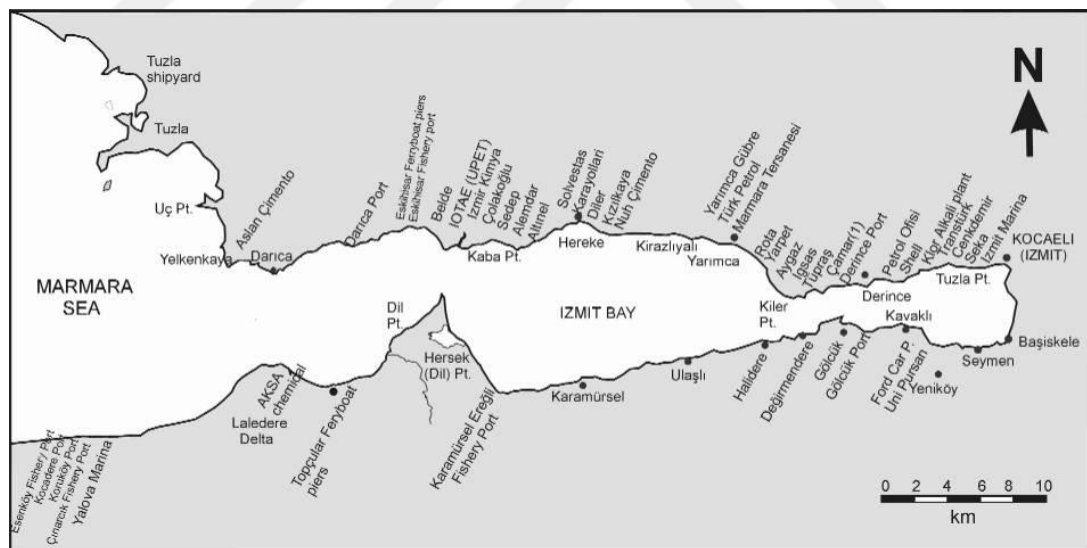


Figure 4.1 The major marine structures and ports in İzmit Bay, (Yüksel et al., 2002)

The Great Marmara Earthquake occurred in 17th August 1999 has caused several damages in several coastal structures placed in coast of İzmit Bay. The 1999 Marmara Earthquake's magnitude was measured as 7.4. The earthquake data as downloaded from Kandilli Observatory is given in Table 4.1.

Table 4.1 The earthquake data (Kandilli Observatory)

Date and hour	03:01:37
Body-wave Magnitude, M_b	6.3 (USGS)
Body-wave Magnitude, M_s	7.8 (USGS)
Moment Magnitude, M_w	7.4 (Kandilli, USGS)
Local Magnitude, M_L	6.7 (Kandilli)
The epicenter of earthquake	40.70N, 29.91E
The earthquake center	Gölcük (Kocaeli)
Depth	18 km

Several facilities including the ports were damaged during this earthquake. The marine structures and ports like Derince Port, Petrol Office Marine Terminal, Tüpraş Oil Refinery Marine Terminal suffered damages. Although Derince Port and Tüpraş coastal structures were moderate damage and Petrol Office was heavily damaged, these marine structures remained partially operational after the earthquake (Yüksel & Güler, 2005).

Petrol Office Marine Terminal serving for loading and unloading of oil is located at the northern coast of İzmit Bay consists of two piers. The structure of Pier #1 is supported with concrete square piles. These piles in the facility are 12-14 m long in the shallow water whereas they are 24-28 m in the deeper water. After the earthquake, no vertical displacement was observed but lateral displacement occurred on the order of 40 cm towards the sea. It was concluded that use of prefabricated driven piles did not provide adequate lateral stiffness being damaged under earthquake induced lateral forces either inertial or kinematic. The use of steel pipe piles might have been much more suitable than reinforced piles. The Pier #2 consists of an old and a new section. The old section is 139 m long with reinforced concrete piles and the new section is 154 m long supported by steel pipe piles. After the earthquake, it was observed that the older section of Pier 2 tilted and displaced away from the adjacent new section because of different dynamic responses of support elements in both parts of the pier becoming out of service after earthquake (Yüksel et al., 2002).

Tüpraş which consists of three various piers and dolphins for berthing ships is the largest oil refinery terminal in Turkey. Two piers in the terminal was supported by inclined concrete piles connected with prefabricated beams on top. After the earthquake, no significant vertical or horizontal displacements were observed so these

two piers were left in service. Only, there were cracks around the pile-to-beam connections without causing serious problems. The third pier, however, was being supported by steel piles. Some of this pier's piles faced buckling type damages and left in service. Evaluation of the pier after the earthquake revealed that corrosion within the splash zone caused strength loss against torsion (Yüksel et al., 2002).

The gravity type quay wall that of which performance is investigated within the scope of this thesis is part of the Derince Port, which is the most important and largest one around the İzmit Bay. The No.6 Quay Wall of Derince Port experienced moderate level damage in 1999 Marmara Earthquake exhibiting seaward displacement, settlement and tilting as a consequence of the liquefaction of the backfill soil.

The gravity quay wall is 12 m high reaching to 15 m with the crown block at the top and 220 m long inside the Derince Port (Figure 4.2 and 4.3). The section of the wall is shown in Figure 4.4. Site measurements following the earthquake showed that seaward movement took place without any vertical displacements. Liquefaction was also observed in the area behind No.6 Gravity Quay Wall around the warehouse (Figure 4.5a). Along with liquefaction, the backfill settlement and seaward movement occurred (Figure 4.5b). The peak ground acceleration was approximately 0.25g to 0.3g (Yüksel et al., 2002). The lateral and vertical displacements were 30 and 50~80 cm, respectively. The lateral displacement pattern as noted following the earthquake is given in Figure 4.5c.



Figure 4.2 Gravity type quay wall inside the Derince Port

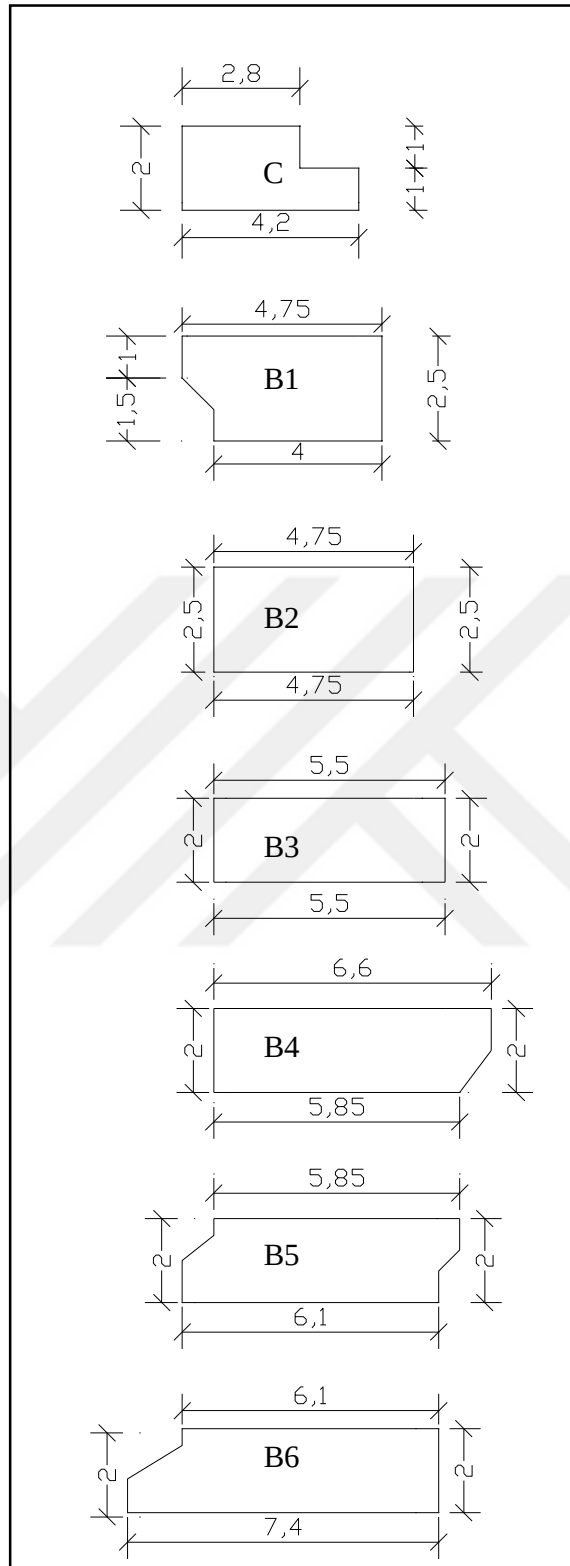


Figure 4.4 Dimension of block type quay walls (unit: m)

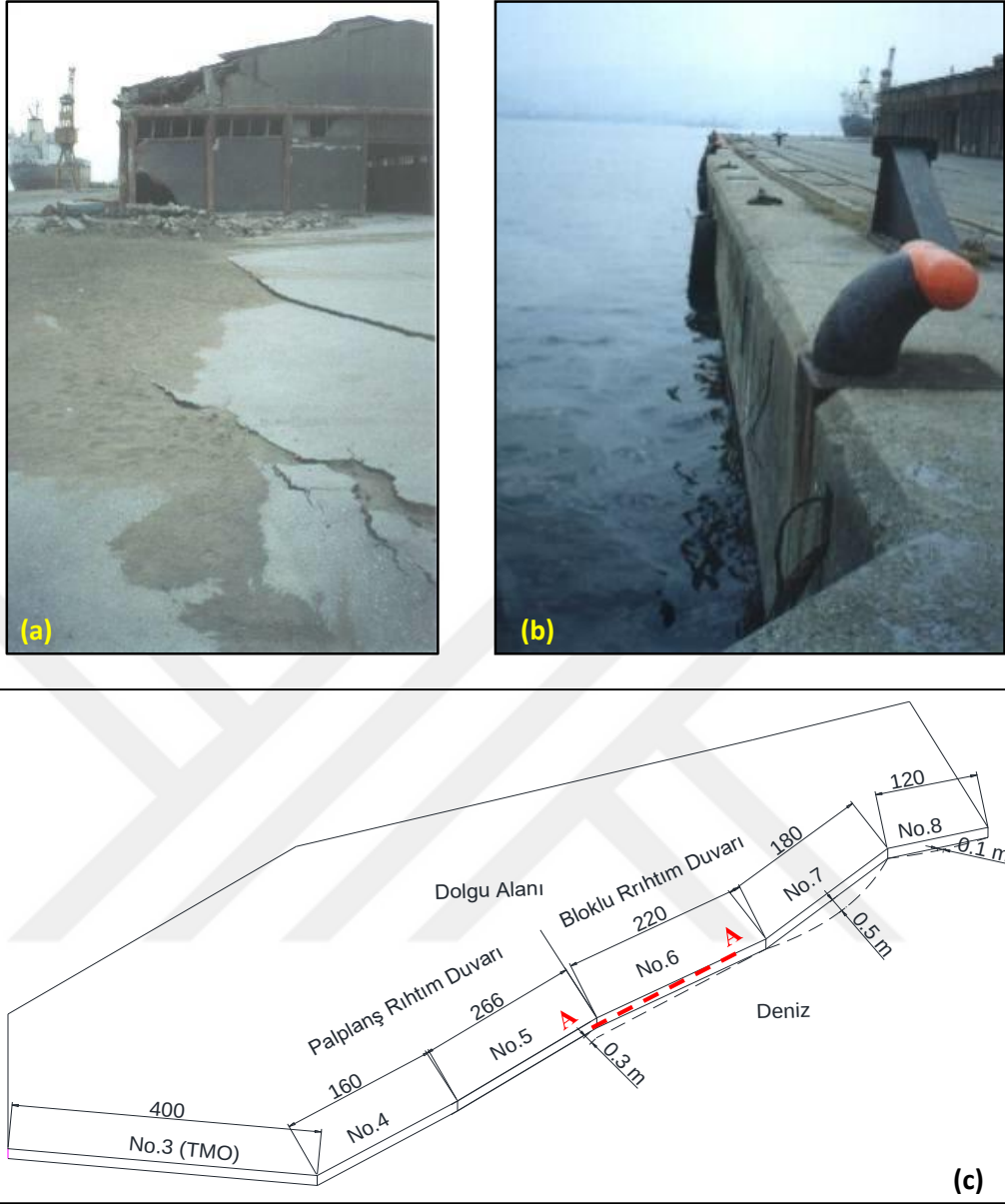


Figure 4.5 Damages due to 1999 Marmara Earthquake in gravity quay wall of Derince Port: (a) The liquefaction traces in backfill between the warehouse and the quay wall; (b) Seaward movement of the quay wall; (c) The lateral displacement pattern after the earthquake (Yüksel et al., 2002)

4.2 The Soil Properties for No.6 Gravity Quay Wall

The soil properties nearby the No.6 Gravity Quay Wall are obtained from the various sources. One such sources are a site investigation report (Geosan, 2016) prepared during the project phase of the extensions planned for the Derince Port well after the 1999 Marmara Earthquake. The location plan of five boreholes (one of them is at the sea and four of them are at the ground) drilled during the site investigation is

given Figure 4.6. The landside boreholes except YKS-08 were 5.0 m deep. YKS-08 was drilled 6.5 m long. On the other hand, YDS-06, which was a seaside borehole, was 30 m long including the water depth. Considering the fact that seabed is at 12 m depth at the site. The bedrock was encountered at 27 m. The borehole logs are provided in Appendix IV of this thesis.



Figure 4.6 The borehole locations on the satellite image (Geosan, 2016)

There are also published literature that involve soil data regarding the gravity type block walls of the port's Wharves 6, 7 and 8 (Yuksel et al. 2000 and 2002). The soil profile that belongs to Section A-A of Figure 4.4c is provided in Figure 4.7. As one can follow on this figure the soil profile prior to the construction of the wharves pose typical sea sediment character. Although the thickness varies with the location there is sand and gravel layer at the top just below the water level. The soft to medium stiff clay layer is approximately 10 m below the sea level at S-IX getting closer to water surface towards S-V location. Standard Penetration Test blow counts given in Figure 4.7 remind that the sandy portions of the “sand and gravel” layer are in loose to medium dense state since N_{30} varies between 4 and 17 whereas the gravel dominant portion at S-103 location may be considered as very dense with $N_{30}=43\sim67$. However, such high blow counts might have been due to the presence of large size soil grains causing the SPT resistance to be artificially high by clogging the SPT spoon. The clay layer underlying the “sand-gravel” attains SPT resistance as low as 4 for some five meters posing N_{30} values of 11 to 23 below that layer.

The engineers responsible of the wharf design at the time recommended that dredging should be made down to -17.5 elevation at S-IX and -15 at S-V to provide a satisfactory foundation soil for the gravity walls. However, the dredging operation could not be made as planned since the ship used for this purpose was able to reach -16 elevation. Therefore, some medium stiff clay layer had to be left below the gravity wall at S-IX location where settlements took place under static loads after the wharves were in service. This is not surprising since the net contact pressure beneath the wall is estimated on the order of $q=140 \text{ kN/m}^2$ high enough to generate undesirable settlements being on the order of at least 10 cm depending on the thickness of the clay layer below the wall.

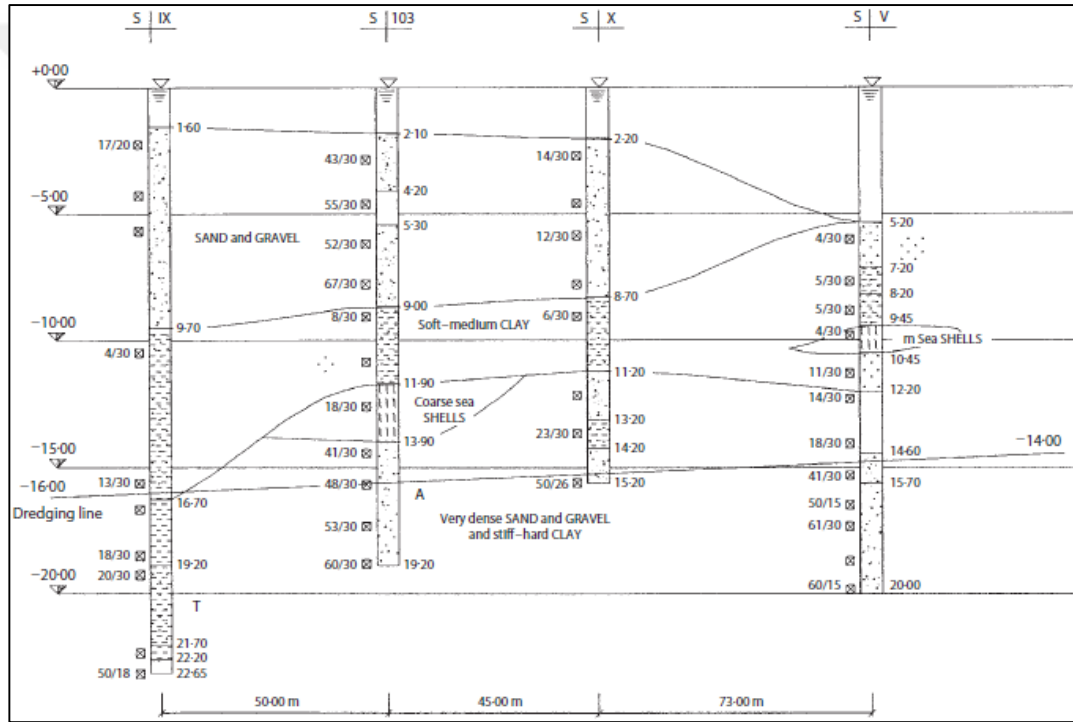


Figure 4.7 Soil profile along Section A-A along Wharf #6 (Yüksel et. al, 2002)

Fines content is an essential parameter for liquefaction potential evaluation, and the data about this parameter is rather limited for the profile in Quay Wall No.6 location. Fines content in non-plastic silt layer in 12 and 14 m depth range below the seabed was determined as approximately 58%. It was found as 7~11% for underlying layers.

Three separate geotechnical models were established in this thesis based on the above-mentioned data. The first model basically depends on the Geosan (2016) data

and represents the stratification with respect to average SPT resistances being $N_{30_ave}=15$ for the backfill where SPT data varies between $N_{30}=7\sim42$, and $N_{30_ave}=20$ and $N_{30_ave}=49$ for the layers underneath the backfill down to the bedrock with SPT resistance varying in between 20 and 76. In the field, sea level controls the groundwater level and GWL was reported as 2.5 m below the surface. Idealized geotechnical model belonging to average parameters is shown in Figure 4.8. It is understood that the first idealized geotechnical model shown in Figure 4.8 is not compatible with site as a result of the dynamic analysis.

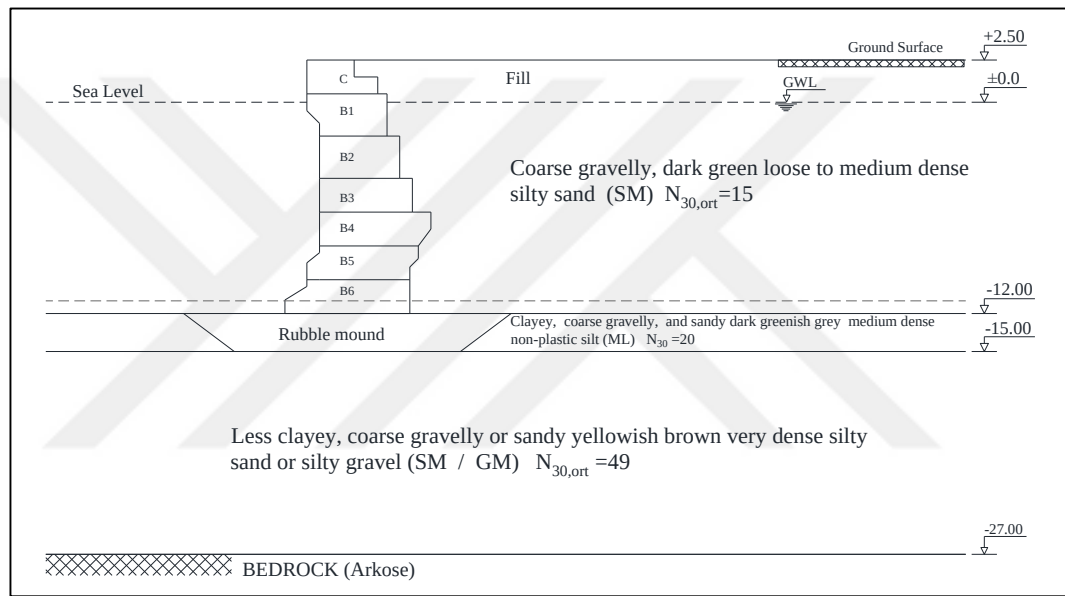


Figure 4.8 Idealized geotechnical model based on average SPT blow count data

The second idealized geotechnical model is established considering SPT variations with depth and geometry information regarding the dredging operation performed during the construction of the quay wall (i.e. the dredging was made through a 19 m wide band down to 16 m depth). Data obtained from Geosan and Section A-A (see Figure 4.7) were utilized during idealization. The stratified model as drawn in this manner is presented in Figure 4.9 but the displacement results in analyses are higher than site observation. So, the third idealized geotechnical model is established considering the non-liquefiable backfill behind the wall as in Figure 4.10.

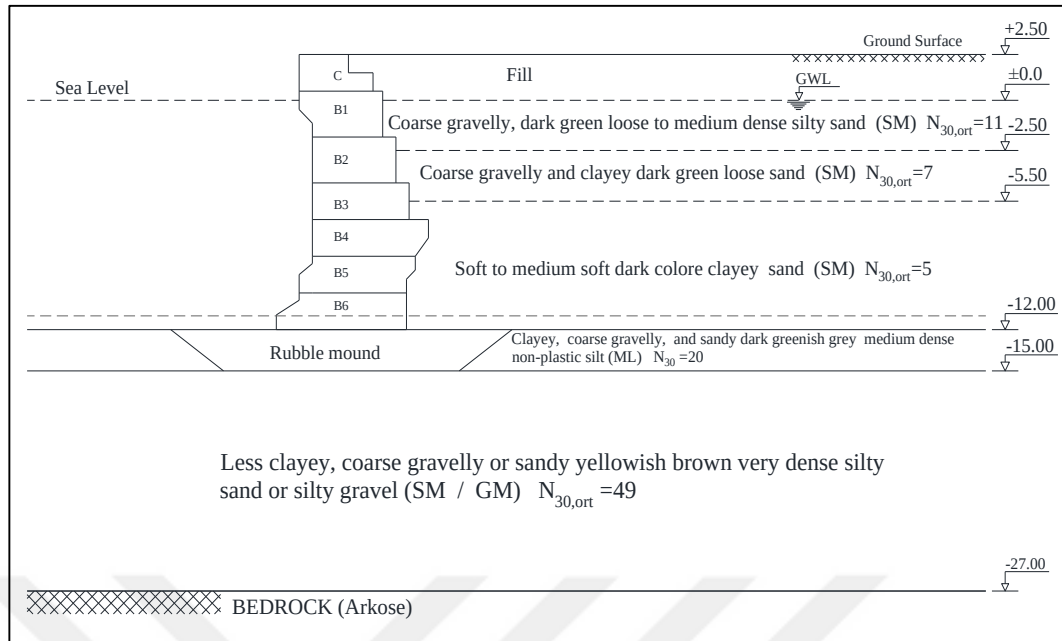


Figure 4.9 The second idealized geotechnical model based on variable SPT blow count data

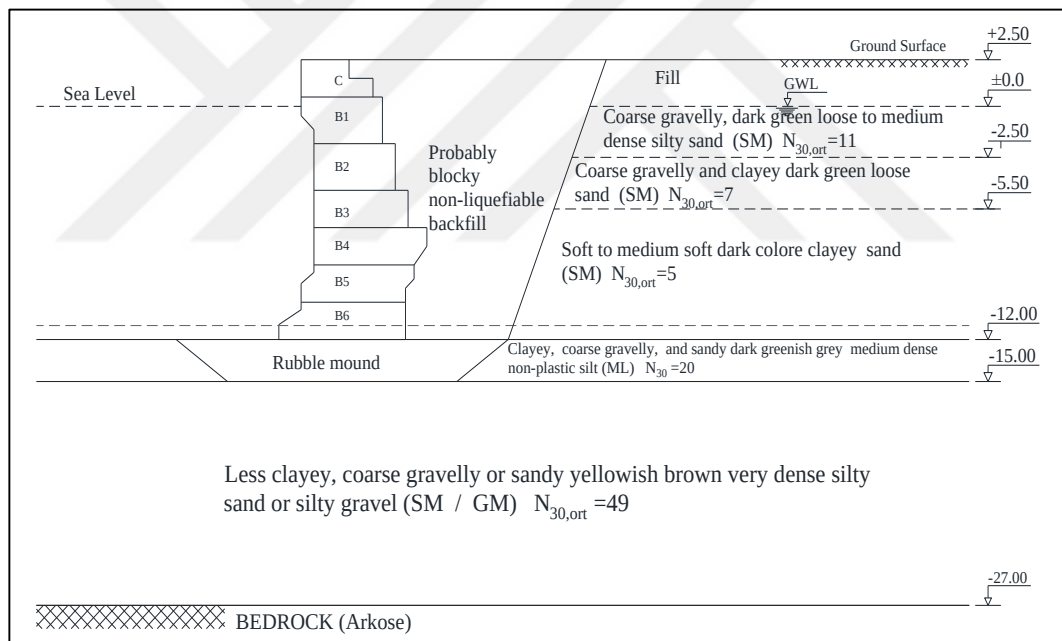


Figure 4.10 The third idealized geotechnical model based on variable SPT blow count data

4.3 Liquefaction Potential of Site Soils According to NCEER Method

As mentioned in the above sections, damages observed alongside the wharves were associated with liquefaction phenomenon. Liquefaction traces could be apparently observed in the field (Figure 4.5a). According to the SPT- $N_{1,60}$ based liquefaction assessment chart as part of the Seed and Idriss Modified Method (Youd et al, 2011),

which is still in use as a standard liquefaction analysis method the medium dense sand and non-plastic silt layers in the soil profile stay within the liquefiable zone (Figure 4.11). Factor of safety values calculated for the Kocaeli Station Record of the 1999 Marmara Earthquake reveals that can be as low as 0.49 depending on borehole location, SPT resistance and analysis depth (see Figure 4.12). As one can notice in Figure 4.12, the threshold SPT- N_{30} resistance below which liquefaction phenomenon is expected to take place is $N_{30}=11$ nearby the ground surface and $N_{30}=15$ after 5.0 m below the surface for the given earthquake ($M=7.5$ and $a_{max}=0.22g$) conditions. It appears that the backfill behind the wall shall have been evaluated as completely liquefiable for design level earthquake conditions. The backfill behind the wall must have been either compacted or the material must have been in such a manner that its grain size distribution curve was not within the liquefiable soil range. Although this aspect of the gravity wall design is well known worldwide, site conditions dictated use of loose and uncompacted soils. There are cases even hydraulic fills were pumped behind the walls as backfills.

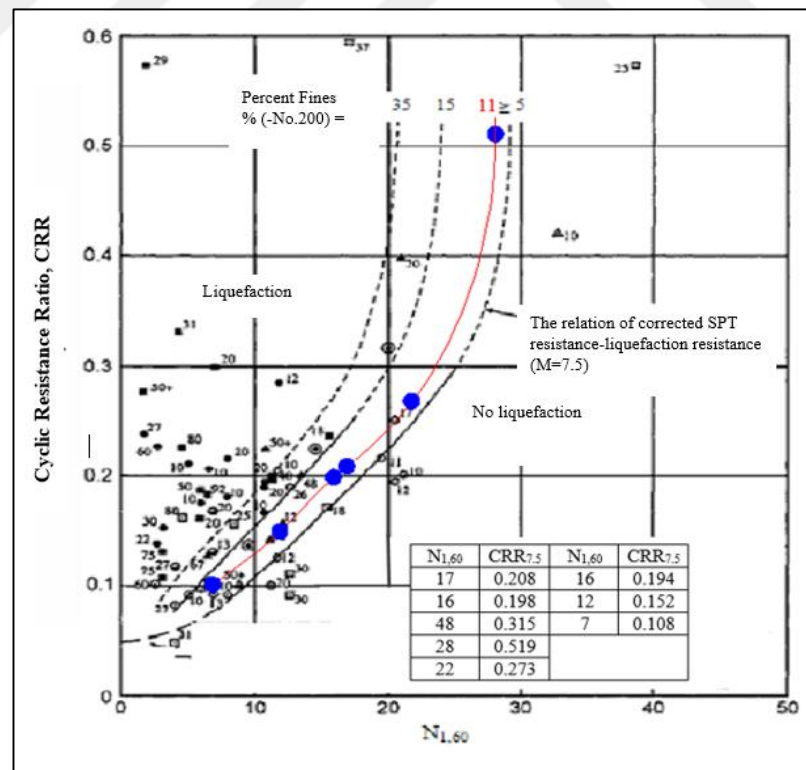


Figure 4.11 The relation between cyclic resistance ratio and corrected SPT (Youd et al., 2001)

BH #	d (m)	$\sigma_{vo,2}$	$\sigma'_{vo,2}$	r_d	CSR	SPT-N	$\sigma'_{vo,1}$	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC	a	b	$(N_1)_{60cs}$	$CRR_{7.5}$	I_D	f	K_s	FS
YKS-5	3.3	61	53	0.977	0.161	9	53	1.37	0.75	1.00	1.00	1.20	11.1	11.0	1.21	1.03	12.6	0.137	45	0.78	1.00	0.85
YKS-5	4.8	91	68	0.967	0.185	12	68	1.21	0.75	1.00	1.00	1.20	13.1	11.0	1.21	1.03	14.7	0.157	47	0.77	1.00	0.85

GWL1	g	gsat	g_w	g'	P_a	ER	BD	Sampler	a_{max}/g	M_w	MSF
2.5	18	20	10	10	100	0.75	10	2	0.22	7.5	1.00

(a) Liquefaction analysis for YKS-5 borehole

BH #	d (m)	$\sigma_{vo,2}$	$\sigma'_{vo,2}$	r_d	CSR	SPT-N	$\sigma'_{vo,1}$	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC	a	b	$(N_1)_{60cs}$	$CRR_{7.5}$	I_D	f	K_s	FS
YKS-6	3.3	61	53	0.977	0.161	11	53	1.37	0.75	1.00	1.00	1.20	13.6	11.0	1.21	1.03	15.2	0.162	50	0.75	1.00	1.01
YKS-6	4.8	91	68	0.967	0.185	12	68	1.21	0.75	1.00	1.00	1.20	13.1	11.0	1.21	1.03	14.7	0.157	47	0.77	1.00	0.85

GWL1	g	gsat	g_w	g'	P_a	ER	BD	Sampler	a_{max}/g	M_w	MSF
2.5	18	20	10	10	100	0.75	10	2	0.22	7.5	1.00

(b) Liquefaction analysis for YKS-6 borehole

BH #	d (m)	$\sigma_{vo,2}$	$\sigma'_{vo,2}$	r_d	CSR	SPT-N	$\sigma'_{vo,1}$	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC	a	b	$(N_1)_{60cs}$	$CRR_{7.5}$	I_D	f	K_s	FS
YKS-7	1.8	32	32	0.988	0.141	8	32	1.70	0.75	1.00	0.75	1.20	9.2	11.0	1.21	1.03	10.6	0.119	44	0.78	1.00	0.84
YKS-7	3.3	61	53	0.977	0.161	20	53	1.37	0.75	1.00	1.00	1.20	24.7	11.0	1.21	1.03	26.6	0.327	67	0.67	1.00	2.03
YKS-7	4.8	91	68	0.967	0.185	14	68	1.21	0.75	1.00	1.00	1.20	15.3	11.0	1.21	1.03	16.9	0.180	50	0.75	1.00	0.97

GWL1	g	gsat	g_w	g'	P_a	ER	BD	Sampler	a_{max}/g	M_w	MSF
2.5	18	20	10	10	100	0.8	10	2	0.22	7.5	1.00

(c) Liquefaction analysis for YKS-7 borehole

BH #	d (m)	$\sigma_{vo,2}$	$\sigma'_{vo,2}$	r_d	CSR	SPT-N	$\sigma'_{vo,1}$	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC	a	b	$(N_1)_{60cs}$	$CRR_{7.5}$	I_D	f	K_s	FS
YKS-8	3.3	61	53	0.977	0.161	11	53	1.37	0.75	1.00	1.00	1.20	13.6	11.0	1.21	1.03	15.2	0.162	50	0.75	1.00	1.01
YKS-8	4.8	91	68	0.967	0.185	7	68	1.21	0.75	1.00	1.00	1.20	7.6	11.0	1.21	1.03	9.1	0.105	36	0.80	1.00	0.57
YKS-8	6.3	121	83	0.955	0.199	7	83	1.10	0.75	1.00	1.00	1.20	6.9	11.0	1.21	1.03	8.3	0.099	33	0.80	1.00	0.49

GWL1	g	gsat	g_w	g'	P_a	ER	BD	Sampler	a_{max}/g	M_w	MSF
2.5	18	20	10	10	100	0.8	10	2	0.22	7.5	1.00

(d) Liquefaction analysis for YKS-8 borehole

Figure 4.12 Liquefaction analyses with respect to NCEER Method for Kocaeli Station Record

4.4 Backward Numerical Analyses of Quay Wall No.6

4.4.1 Finite Element Model

Performance analyses of civil engineering structures with complicated boundary conditions can only be performed using numerical solution algorithms. The finite element method is one of such techniques utilized for analysis of gravity type quay walls under earthquake loading.

The above presented methodology was followed in backward analyses of Quay Wall No.6. The method followed over here consists of three stages as suggested by (Iai, 2011). In the first stage, the problem in terms of displacement which causes inadequacy rather than destruction is defined. In the second stage, the processes, which start from initial state before earthquake happened and continue to residual state after earthquake took place (e.g., construction stage, backfilling the ground, the location of the bedrock where earthquake motion is defined and numerical analysis model), are identified. In the final stage, numerical analysis is performed.

The earthquake analysis of coastal structures which are either built or present in Turkey is performed according to Turkish Seismic Design codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007). The performance levels of gravity quay walls are identified in relevant codes (Table 4.2). The maximum permanent horizontal displacement ratio of No.6 Gravity Quay Wall is 2.5% ($\cong \delta_{h_max}/h_{wall}$) and this ratio can be accepted within the controlled damage performance level limits at which less severe and repairable damages are allowed under earthquake effect.

Tablo 4.2 Proposed damage criteria for gravity quay walls, (TSDC-CRA, 2007)

The limits of displace/deformation	Performance Level	
	MD	CD
The ratio of permanent horizontal displace over height (height) (%)	< 1.5	1.5-5
Residual tilting towards sea (degree)	<3	3-5
Differential settlement between apron and non-apron areas (cm)	30-70	-
Differential settlement on apron (cm)	3-10	-

* Minimum Damage Performance Level

** Controlled Damage Performance Level

Dynamic finite element model which is established using the wall geometry and idealized soil profile belonging the gravity quay wall is shown in Figure 4.13. The boundary condition for the vertical model boundaries (x-max and x-min component) are chosen as tied degrees of freedom. The reason of the selection of tied degrees of freedom is Petalas and Galavi's studies. The information about model geometry and dynamic vertical boundary condition are given in detail in their studies. In this boundary condition, the geometry has to be mirrored and the problem has to be fully symmetric in order to properly model the motion due to earthquake. To have the same

displacements, this boundary condition is connected the nodes on the same elevation. This boundary provides the laminar box behaviour as more realistic than the other dynamic boundary conditions. Acceleration-time history is applied to the model at the bedrock level, and along both vertical boundaries, the boundary x_{min} and boundary x_{max} , standard tied degrees of freedom boundary conditions are defined. The horizontal model boundaries, boundary y_{min} and the boundary y_{max} , are chosen compliant base and none, respectively. The number of elements and nodes in this case are 9007 and 73382 respectively, and the mesh is very fine.

The locations of the nodal points used for evaluating analysis results are also presented on the same figure. Hardening soil model (HS) is assigned to Layer #1 and #7, and UBC3D PLM is assigned to Layer #2, #3, #4, #5, #6 and #8. Model parameters are given in Table 4.3 and Table 4.4, respectively.

For concrete blocks of the gravity type quay wall non-porous linear elastic material model is used. The unit weight, Young's modulus and Poisson's ratio are assumed as $\gamma_{\text{concrete}}=24 \text{ kN/m}^3$, $E=32 \times 10^6 \text{ kPa}$ and $\nu=0.2$, respectively. The hardening soil model is used to generate initial stresses due to gravity loading in the initial phase. Hardening soil material model (HS) parameters are summarized in Table 4.3.

UBC3D-PLM model, which enables to model liquefaction behavior of sand and silty sand under seismic loadings, is initially developed in University of British Columbia (Puebla et. al., 1997; Beaty & Byrne, 1998). UBCSand model is later modified by Tsegaye (2010) and Petalas and Galavi (2013). The required parameters for UBC3D-PLM model are revised by Makra (2013) based on correlations depending upon $N_{1,60}$ value in UBCSand model developed by Beaty (1998). The parameters assigned to finite element model in this model are presented in Table 4.4. The relationships used in generating model parameters of this table are given between Equations 4.1~4.4:

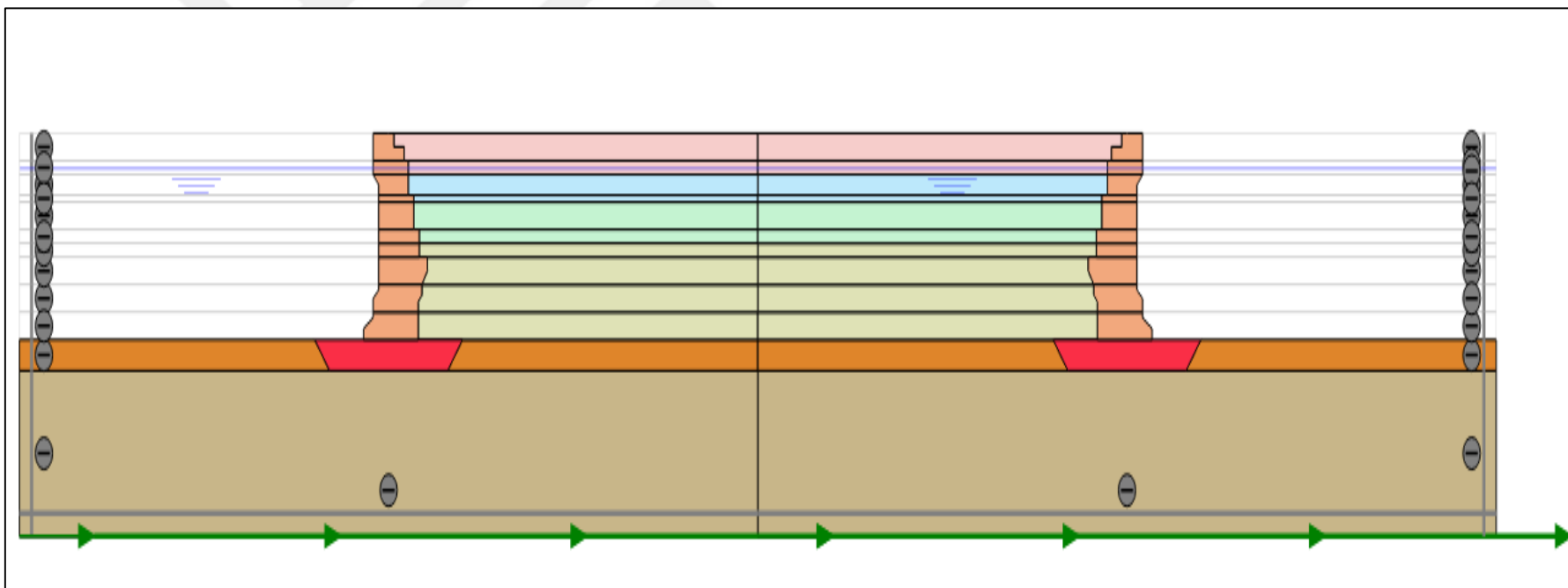


Figure 4.13 The model geometry of dynamic finite element

$$\phi_p = \phi_{c_v} + (N_{1,60}/10) + \max(0, (N_{1,60} - 15)/5) \quad (4.1)$$

$$k_B^e = 0.7 * k_G^e \quad (4.2)$$

$$k_G^e = 21.7 * 20 * (N_{1,60})^{0.3333} \quad (4.3)$$

$$k_G^p = k_G^e * (N_{1,60})^2 * 0.003 + 100 \quad (4.4)$$

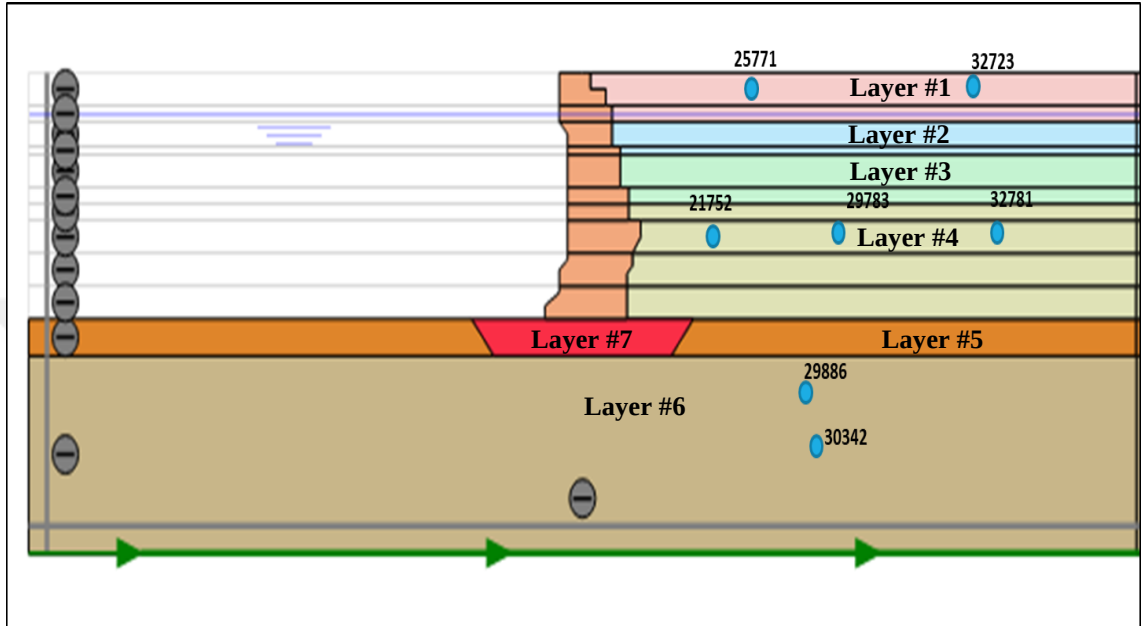


Figure 4.14 The model geometry for stratified soil profile

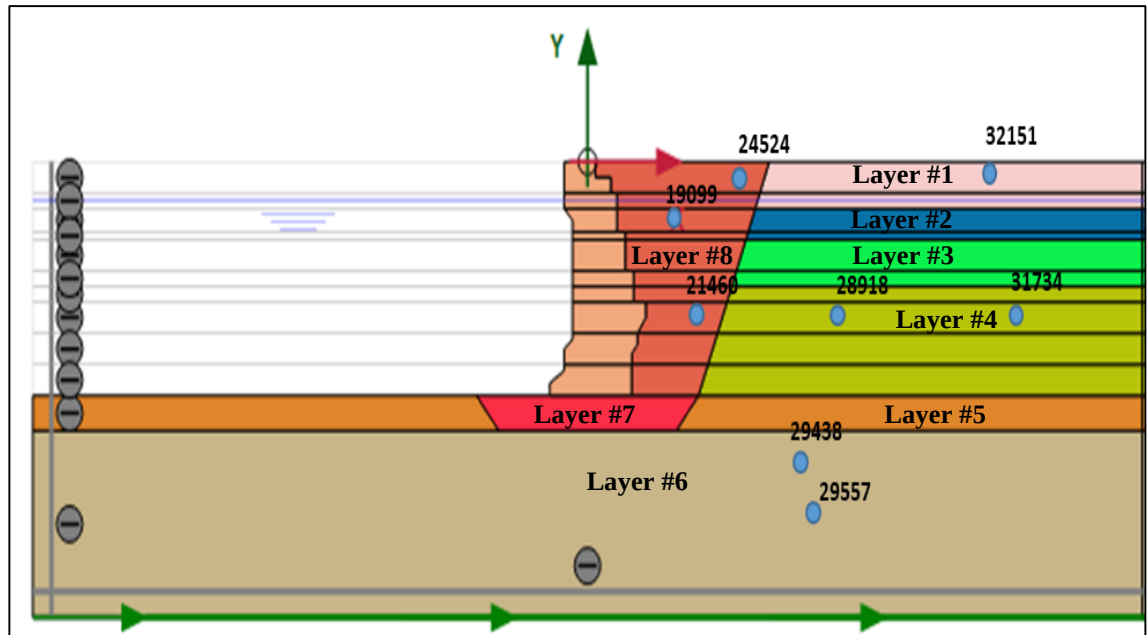


Figure 4.15 The model geometry for stratified soil profile with non-liquefiable backfill

Table 4.3 Hardening soil material model (HS) parameters

Parameters	Unit	Layer #1	Layer #2	Layer #3	Layer #4	Layer #5	Layer #6	Layer #7	Layer #8
ϕ	°	30	27	25	25	35	38	38	40
c'	kPa	10	0	0	0	0	0	0	0
γ_{unsat}	kN/m ³	16	16	16	16	16	16	16	18
γ_{sat}	kN/m ³	18	19	19	19	19	19	19	19
v	-	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
ψ	°	0	0	0	0	5	8	8	10
$E_{50\text{ref}}$	kN/m ²	22000	16000	12000	10000	42000	104000	50000	30000
E_{oedref}	kN/m ²	22000	16000	12000	10000	42000	104000	50000	30000
E_{urref}	kN/m ²	66000	48000	36000	30000	126000	312000	150000	90000

Table 4.4 UBC3D-PLM model parameters

Parameters	Unit	Layer #2	Layer #3	Layer #4	Layer #5	Layer #6	Layer #8
$N_{1,60}$	-	8	6	5	21	52	34
ϕ_{cv}	°	26	24	24	32	25	32
ϕ_{p}	°	27	25	25	35	38	38
c	kPa	0	0	0	0	0	0
k_B^e	-	607.6	552	519	838	1134	984.2
k_G^e	-	868	789	742	1197	1620	1406
k_G^p	-	267	185	156	1684	13241	4976
m_e	-	0.5	0.5	0.5	0.5	0.5	0.5
n_e	-	0.5	0.5	0.5	0.5	0.5	0.5
n_p	-	0.4	0.4	0.4	0.4	0.4	0.4
R_f	-	0.9	0.9	0.9	0.9	0.9	0.9
P_A	kPa	100	100	100	100	100	100
σ_t	kPa	0	0	0	0	0	0
fac_{hard}	-	1	1	1	1	1	1
fac_{post}	-	1	1	1	1	1	1

4.4.2 FEM Analysis Stages

The finite element model geometry is established following the standard methodology. After the determination of geometry, the points, in which lines of the geometry begins and ends are determined and input via the interface of the software. The lines are used to determine the physical boundaries of the FEM geometry. The lines are useful in determination of the soil clusters. The soil properties in a single cluster are homogeneous and isotropic. The material models are assigned to the clusters. Finally, the interface is defined to model the interaction between the soil and

the structure. The finite element mesh is formed using the automatic meshing algorithm of the software and 15 node triangular elements are used since they are suitable for nonlinear analysis.

After the FEM model is completed, the earthquake motion is defined by imposing a prescribed line at the bottom boundary. The prescribed displacement is defined at the bottom of the model. The x-component of the prescribed displacement is set as “prescribed” and assigned a value as 1 for the base because only the upward propagation motion is taken into account for the compliant base. The y-component of the prescribed displacement is set as “fixed”.

Dynamic finite element analyses were conducted for three earthquake motions that approximately represent the earthquake levels defined in TSDC-CRA of DLH (Turkish Seismic Design Code for Coastal-Railway-Airport Structures). In this manner the Düzce Earthquake Record may be considered as the DD-3 level motion, whereas the Kobe and Kocaeli Station records correspond to DD-2 and DD-1 levels, respectively.

Horizontal component acceleration-time series belonging to three earthquakes (two of them occurred on the North Anatolian Fault) and the general characteristics of the records are given in Figure 4.16 and Figure 4.17. Only the effective periods of the earthquakes during which significant cycles occur were used in the analyses. As one can observe that predominant periods of the three earthquakes are low being $T_0=0.16$, 0.39 and 0.47 seconds for Kocaeli, Düzce, and Kobe records, respectively. When the predominant periods are reviewed, Kobe has the higher periods components than the others. It is mean that the higher energy harmonics are involved especially in the high period area. Second and third periods are also read as (0.44; 0.82), (0.77; 1.36) and (0.88; 1.44) seconds for Kocaeli, Düzce, and Kobe earthquakes. Spectral acceleration values corresponding to the third period of Kocaeli Station are higher than the others. It should be mentioned that the Kocaeli Station is only 7.0 km away from the Derince Port.

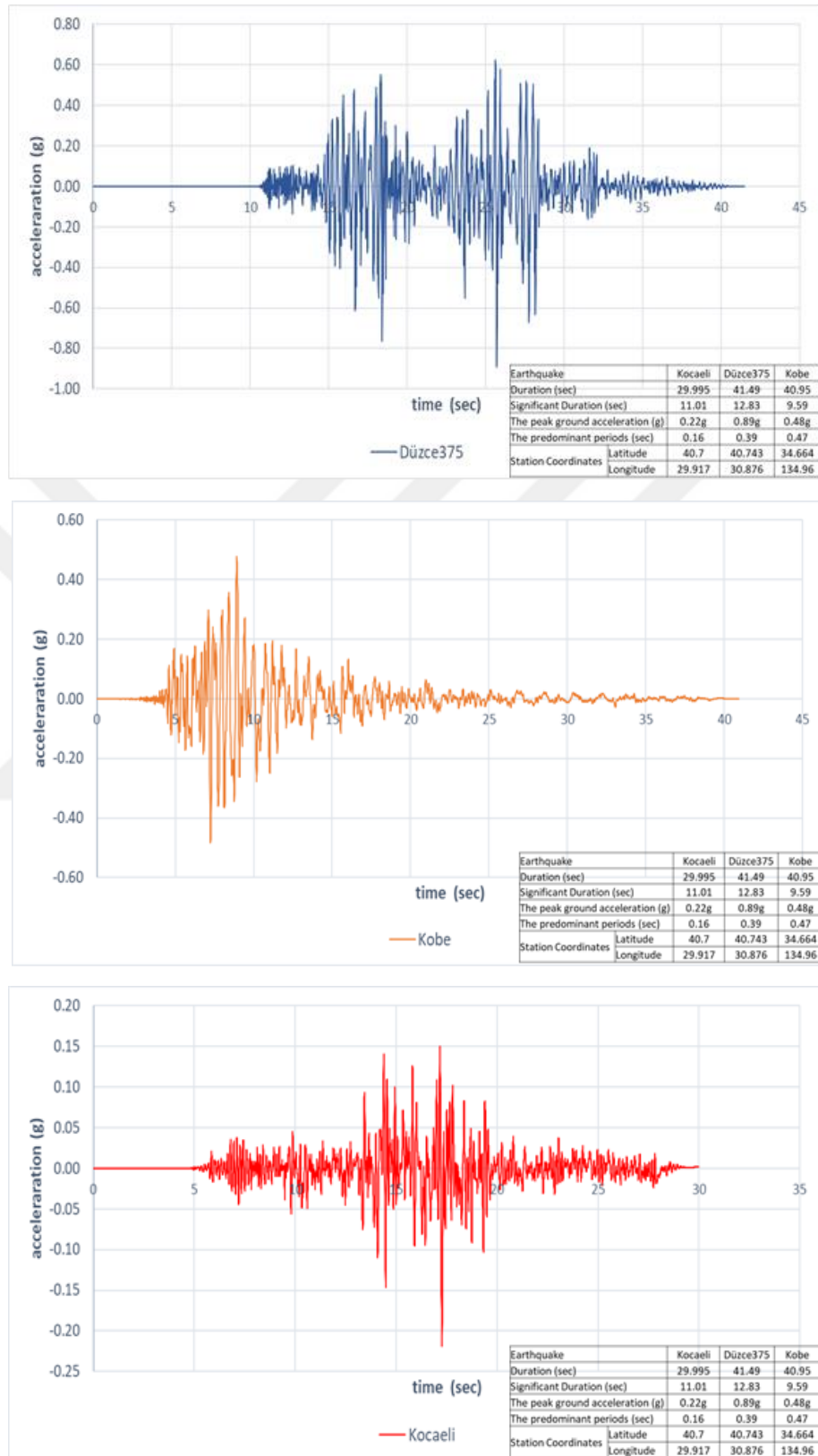


Figure 4.16 The acceleration time series related to analyses and general properties related to series

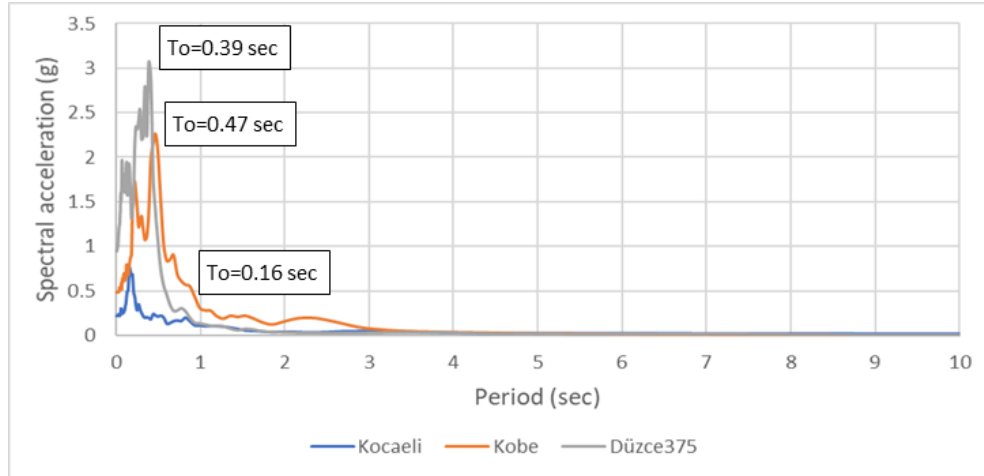


Figure 4.17 The elastic acceleration spectrum for %5 damped ratio at the bedrock level

Although finite element computation consists of four phases, one should pay special attention to three of them. These are the “initial conditions”, the “switch to UBC3D-PLM material model for the liquefiable soils following the initial conditions analysis” phase, and the earthquake analysis phase. The phases are described in the following and summarized in Table 4.5 along with the other sub phases:

1. The initial phase is responsible for generating the body forces to each one of the finite elements encompassing the soil layers. As the body forces are implemented to the soil layers, either K_0 stress condition or the so-called gravity loading is preferred. The first option is suitable for soil profiles where the stratification is completely horizontal and shear stresses do not exist along the horizontally aligned surfaces. The gravity loading option, however, is applicable to cases where the ground surface or sub soil layers are sloped.
2. It is a well-known fact that the UBC3D-PLM model is applicable only to the analysis of dynamic behavior of cohesionless soils. The body forces cannot be generated with this model in the initial phase. Therefore, it is recommended to employ hardening model or hardening small strain model in the initial stage and then switch to the UBC3D-PLM model in earthquake analysis stage (Galavi et al., 2013). The switching process is realized in a separate plastic analysis stage by assigning UBC3D to the clusters that belong to liquefiable soil layers.

3. The dynamic analysis is performed following assignment of the UBC3D-PLM model of which is drainage condition is selected as “Undrained A” in Plaxis. In this stage dynamic loading options and earthquake boundary conditions are activated.

Table 4.5 The construction stages in finite element analyses

Phase	Explanation
Initial Phase	1-The default settings of the initial phase are used. 2-Body forces are implied in a such a manner that material models suitable for the problem of interest are assigned to soil clusters. 3-The ground water and flow conditions are determined.
Phase-1	1-The UBC3D-PLM material model is defined as the material model for liquefiable soil layers.
Phase-2	1-The concrete blocks of the quay wall are defined in this phase.
Phase-3	1-The excavation is carried out and geometry of the analysis model is complete.
Phase-4	1-This step is dynamic calculation. 2-The significant earthquake duration is input to the dynamic time interval. The significant duration is determined using DeepSoil software. 3-The reset displacement to zero is clicked. 4-The maximum steps value is entered as number of the acceleration input data. Briefly, the max step is equal to number of data points. 5-The dynamic boundary is chosen in the model explorer menu. The x-max and x-min component are chosen as tied degrees of freedom and the y-min component is chosen as the compliant base in the dynamics menu. 6-The x-min, x-max, y-min and y-max components are chosen as free in the deformation menu.

4.4.3 Discussion of Analysis Results

Analysis results were discussed especially in terms of excess pore pressure development since back analysis of the liquefaction phenomenon and assessment of the wall performance in relation with this mechanism is the major goal of this thesis. The computed results we especially compared on selected displacement nodes and stress points as shown in Figure 4.14 and Figure 4.15.

Among the acceleration-time series used in the analyses, Düzce record has the highest acceleration value. This record is followed by Kobe and Kocaeli records. In this analyses, Kobe and Düzce accelerations are scaled according to Kocaeli acceleration, so the peak ground acceleration is the same in three earthquakes. In the analyses performed, it was observed that there is excess pore water pressure for all three earthquakes with consequent strength loss (Figure 4.18a~18c).

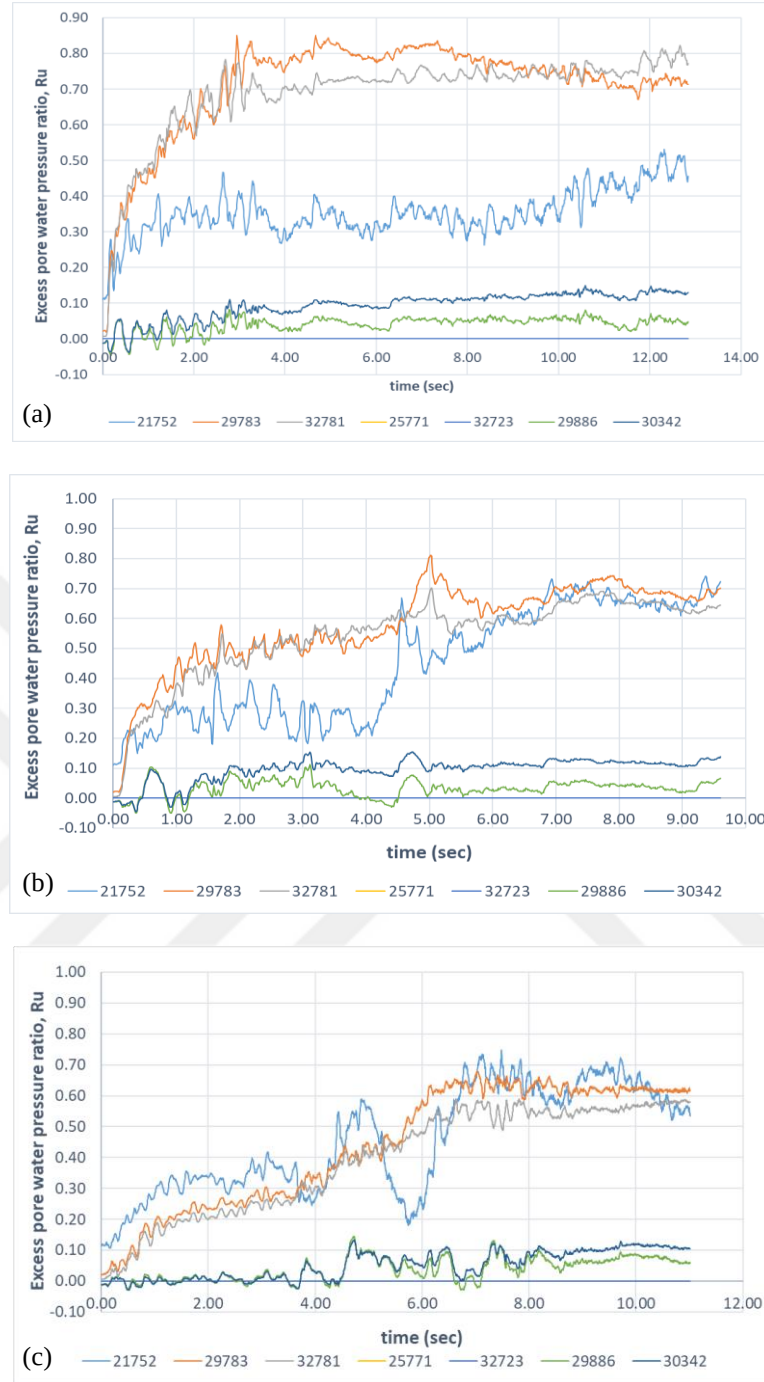


Figure 4.18 The change of excess pore water pressure ratio in time for stratified soil profile in undrained model (a) Düzce Earthquake scaled according to Kocaeli Earthquake; (b) Kobe Earthquake scaled according to Kocaeli Earthquake; (c) Kocaeli Earthquake

When the variation of excess pore water pressure ratio with time is examined, although the peak accelerations are the same, it is seen that its excess pore water pressure development for Düzce earthquake is more than the Kobe and Kocaeli Earthquake analysis in Figure 4.18. Additionally, although excess pore water occurred

at the middle of the Layer #4, in a position far away from quay wall (Point #: 29783), at approximately $t=3.0$ s and when r_u is greater than 0.80, it is seen that there is an accumulation in excess pore water pressure and soil loses strength especially in Düzce Earthquake (Figure 4.18a). Such as Düzce Earthquake, it is observed that excess pore water is gradually building up and r_u is approximately equal 0.8 at the same point at approximately $t=5.00$ s Kobe Earthquake (Figure 4.18b). The reason behind why although peak acceleration in three earthquakes is same, its excess pore water pressure development in Düzce Earthquake is more than Kobe Earthquake and Kocaeli Earthquake, can be explained with more high-energy harmonics which are found in high period area. During earthquake, the period of soil grows and the soil softens. If the earthquake record has high period component, the softening soil catches the earthquake predominant period. In brief, together with the development of excess pore water pressure, the period which extends to soil layer is followed by the high-period harmonics in acceleration-time series and re-accumulation occurs before excess pore water pressure is dissipating. If the earthquake has high period components and the soil is suitable for liquefaction, the liquefaction phenomena is in sight easier. High-period harmonics in Kobe Earthquake can be observed on nodes (Point #: 29783 and 32781) which are in very dense sand-gravel layer where accumulation of excess pore water pressure takes place like as Düzce Earthquake. As moving away from quay wall, the ratio of excess pore water pressure is increasing, and this can be related to decrease in inertia effect due to the acceleration of the gravity quay wall.

In the stratified soil with non-liquefiable backfill, the analysis results are shown similarity with the stratified soil profile except one point (Node # 19099, Figure 4.19). The point is behind the wall and in the non-liquefiable backfill. The negative pore water pressure is observed in this point. The reason behind why the negative pore water pressure occurred can be explained as the soil dilatation. The saturated dense soil exhibits expansion under dynamic load, so the effective stress decreases near ground surface since effective stresses are lower making dilatation of dense soil easier (Figure 4.19). The negative pore water pressure is observed clearer in Figure 4.22b-4.26b-4.30b.

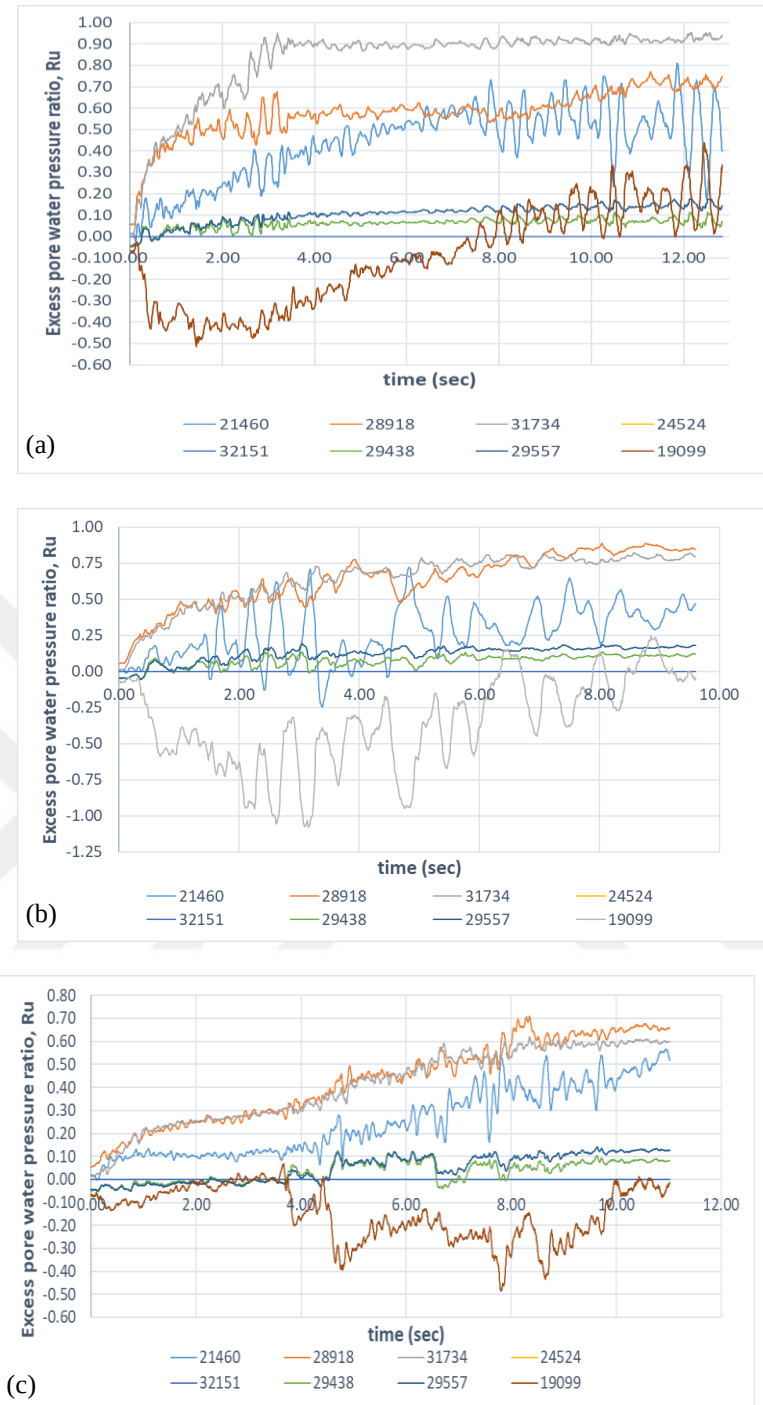


Figure 4.19 The change of excess pore water pressure ratio in time for stratified soil profile with non-liquefiable backfill in undrained model (a) Düzce Earthquake scaled according to Kocaeli Earthquake; (b) Kobe Earthquake scaled according to Kocaeli Earthquake; (c) Kocaeli Earthquake

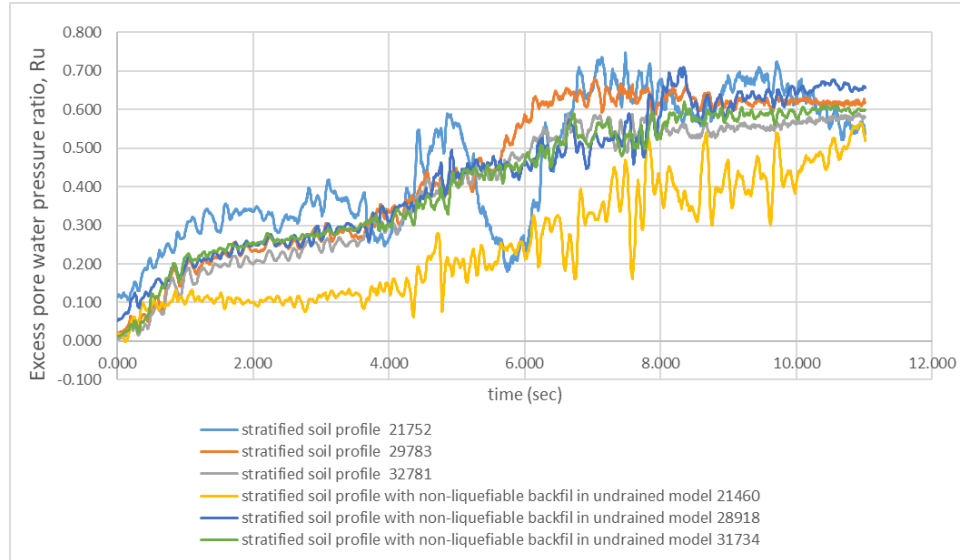


Figure 4.20 The comparison of excess pore water pressure ratio in time between stratified soil profile and stratified soil profile with non-liquefiable backfill in undrained model for Kocaeli Earthquake

The comparison of excess pore water pressure ratio between stratified soil profile and stratified soil profile with non-liquefiable backfill is shown in Figure 4.20. It is seen that non-liquefiable backfill behind the wall exhibited favorable effect against liquefaction. Likewise, the positive trend was also observed on displacements in Figure 4.23-4.27-4.31.

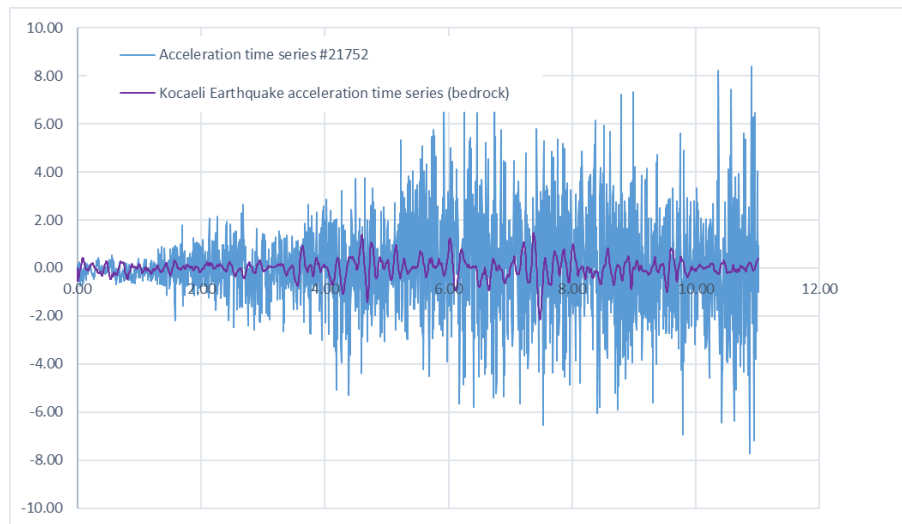


Figure 4.21 The calculated acceleration-time series behind the quay wall in Kocaeli Earthquake analysis and the comparison with the earthquake record defined at bedrock level

In Düzce and Kobe analysis, partial excess pore water pressure development occurs at the same nodes and a decrease in shear strength happens in the entire profile. In the Kocaeli Earthquake analysis, excess pore water pressure is gradually increasing at only one node (Point #: 21752) nearby the quay wall reaching to $r_u=0.75$. The reason why accumulation of excess pore water pressure at this point is higher than the others may be explained with inertia effect of the quay wall. The graph given in Figure 4.21 presents that for Point #: 21752, acceleration becomes higher than the value at the bed rock towards the end of significant duration of the earthquake. The spatial distribution of the zones where excess pore water pressure is $r_u>0$ in Düzce, Kobe and Kocaeli Earthquake analysis is displayed in Figure 4.22-4.26-4.30 according to soil profile. In the Figure 4.22-4.26-4.30, it can be followed that excess pore pressure development may also depend on boundary effects in the model as well as and the interaction between quay wall and soil. This case can also be observed in distribution of cumulative, horizontal and vertical displacements calculated at the end of the analysis. The analysis results are given as comparative according to soil profile (Figure 4.23~33). As moving away from the wall, calculated acceleration is decreasing.

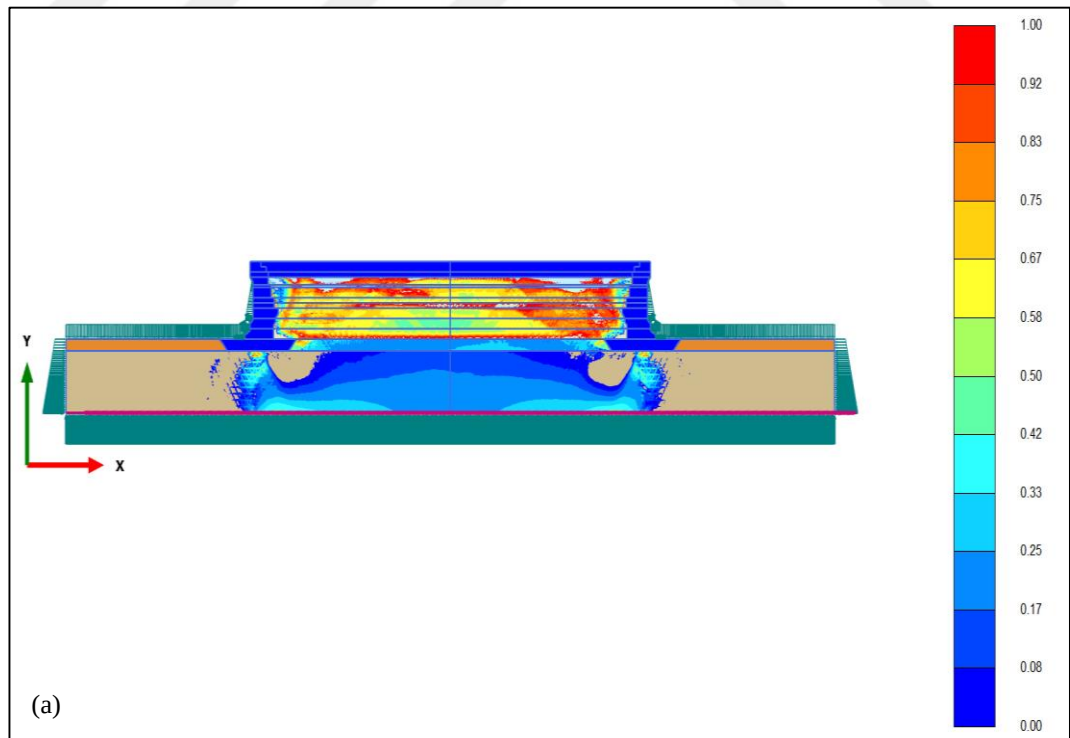


Figure 4.22 The distribution of excess pore water pressure ratio in dynamic finite element model in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

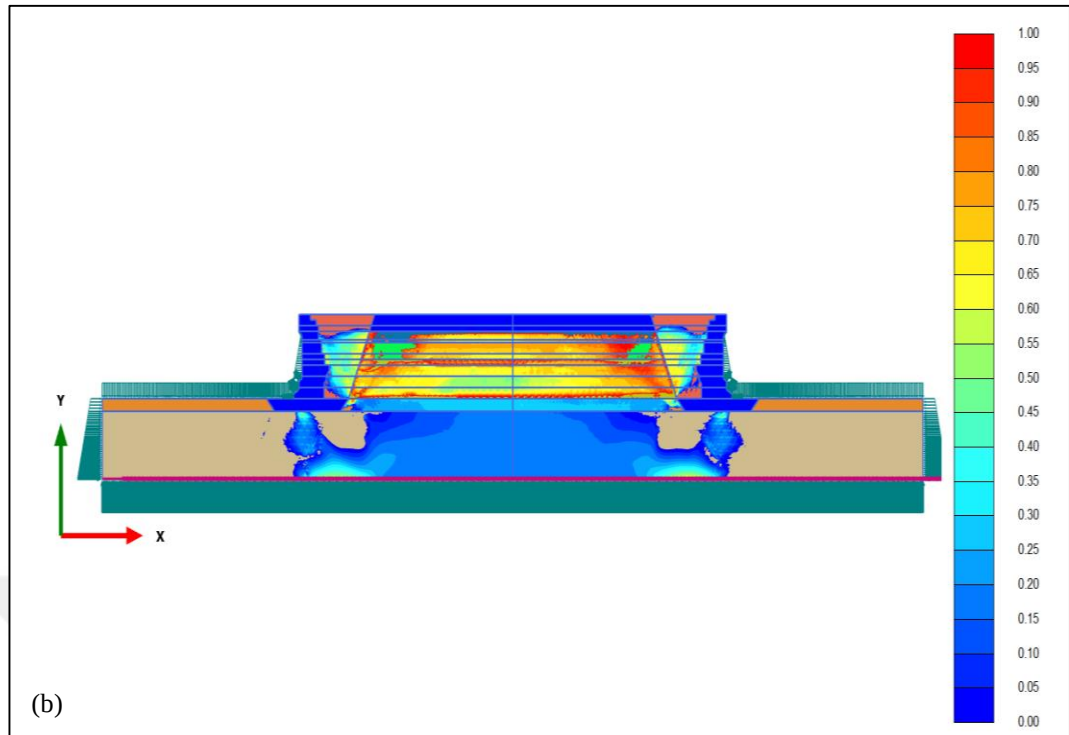


Figure 4.22 continues

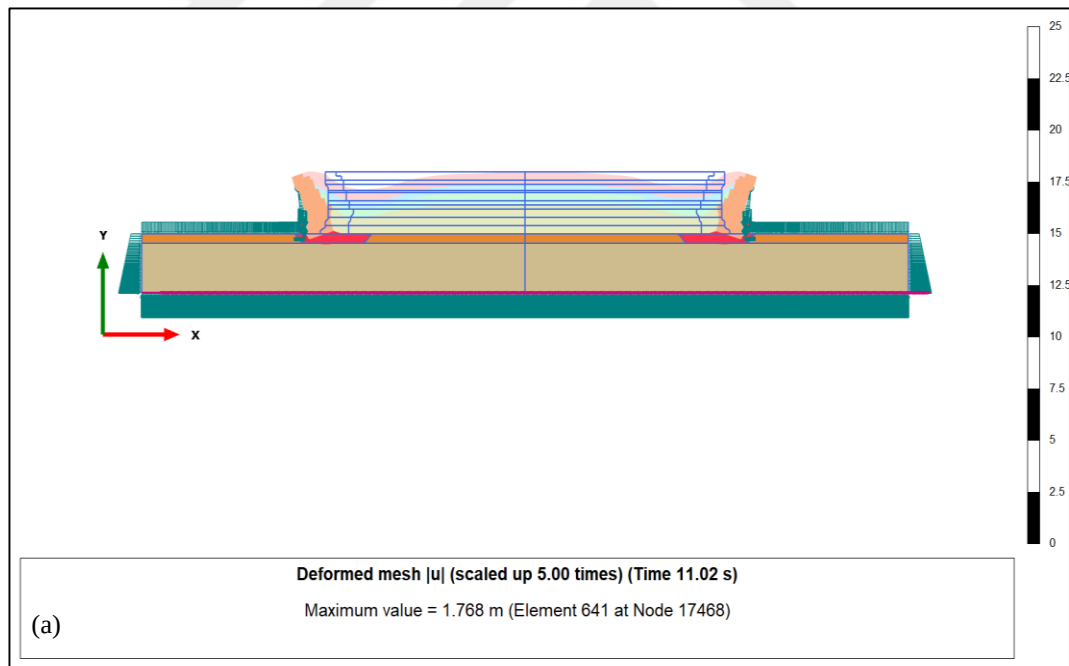


Figure 4.23 The cumulative displacement and quay wall displacement in terms of “deformed mesh” form in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

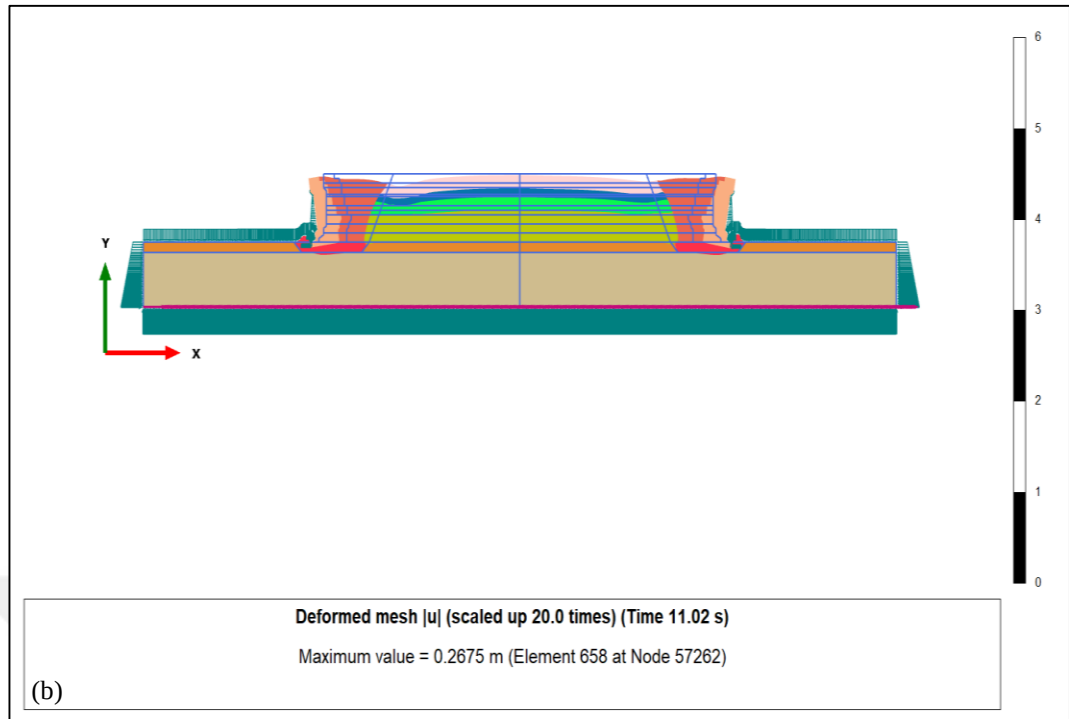


Figure 4.23 continues

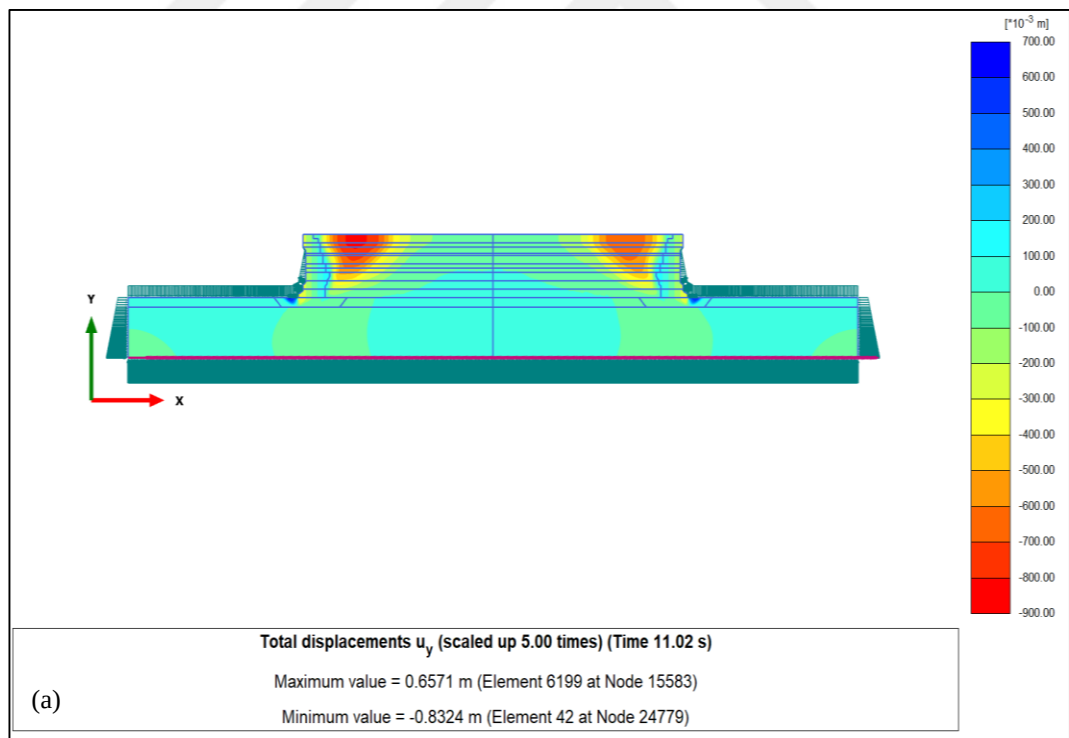


Figure 4.24 The vertical displacement distribution in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

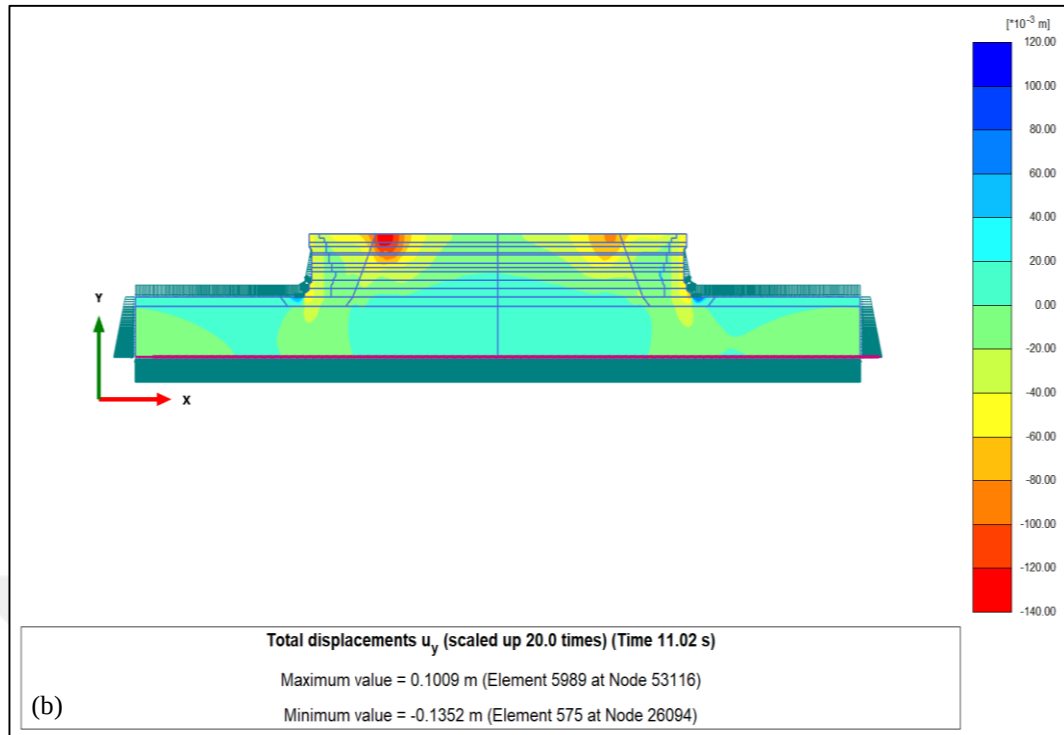


Figure 4.24 continues

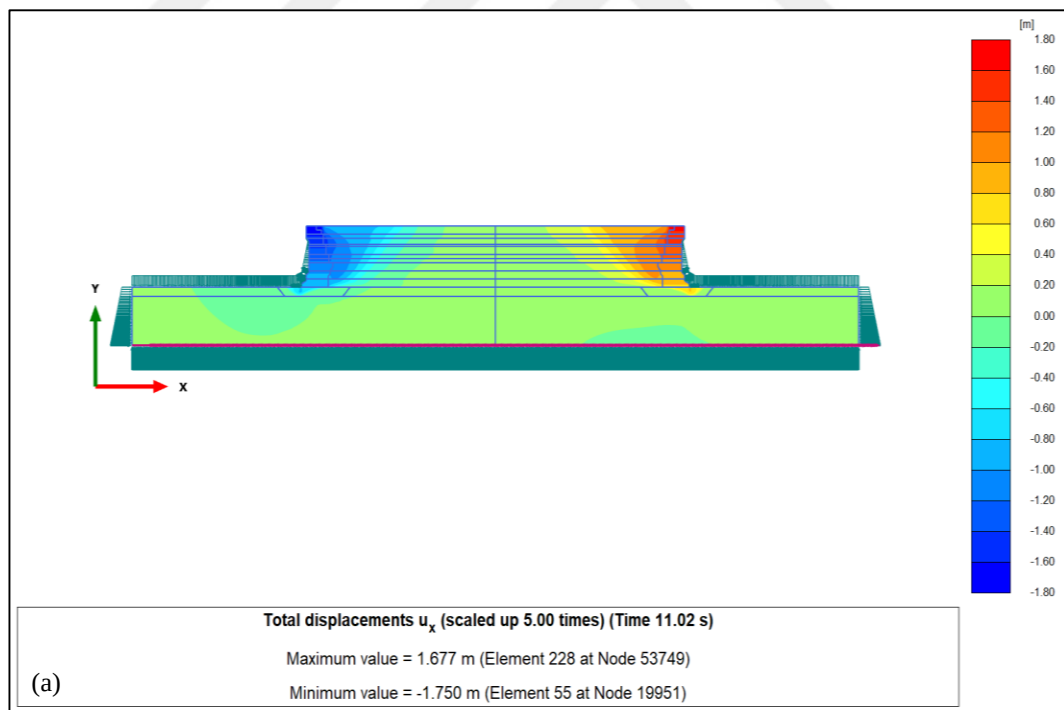


Figure 4.25 The lateral displacement distribution in Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

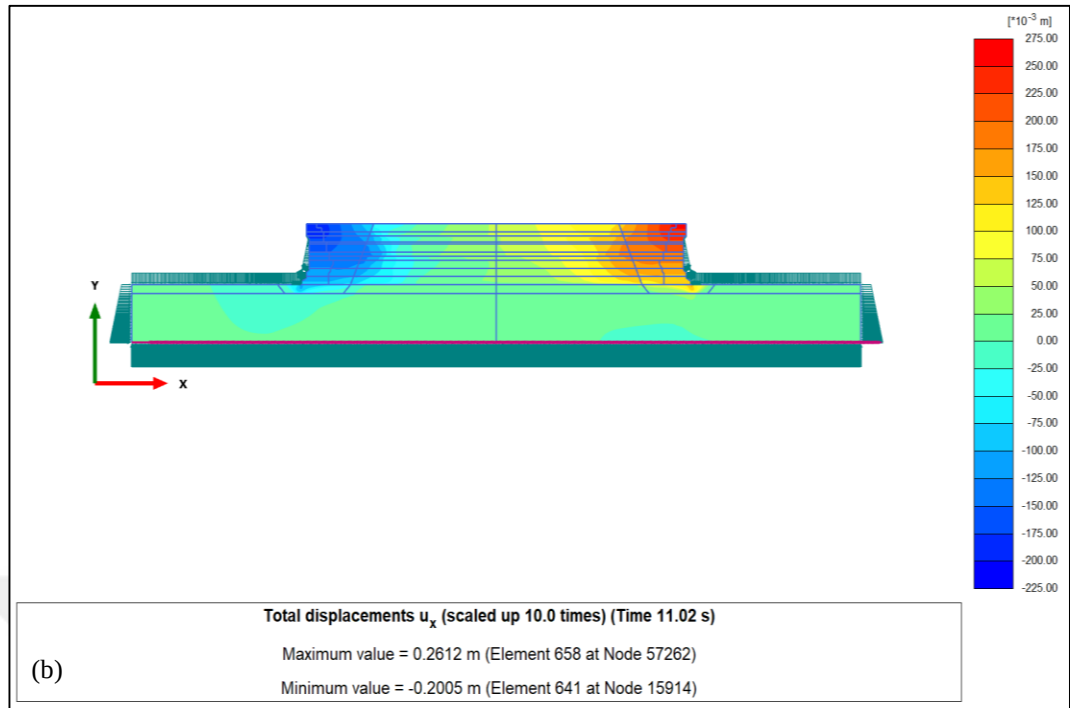


Figure 4.25 continues

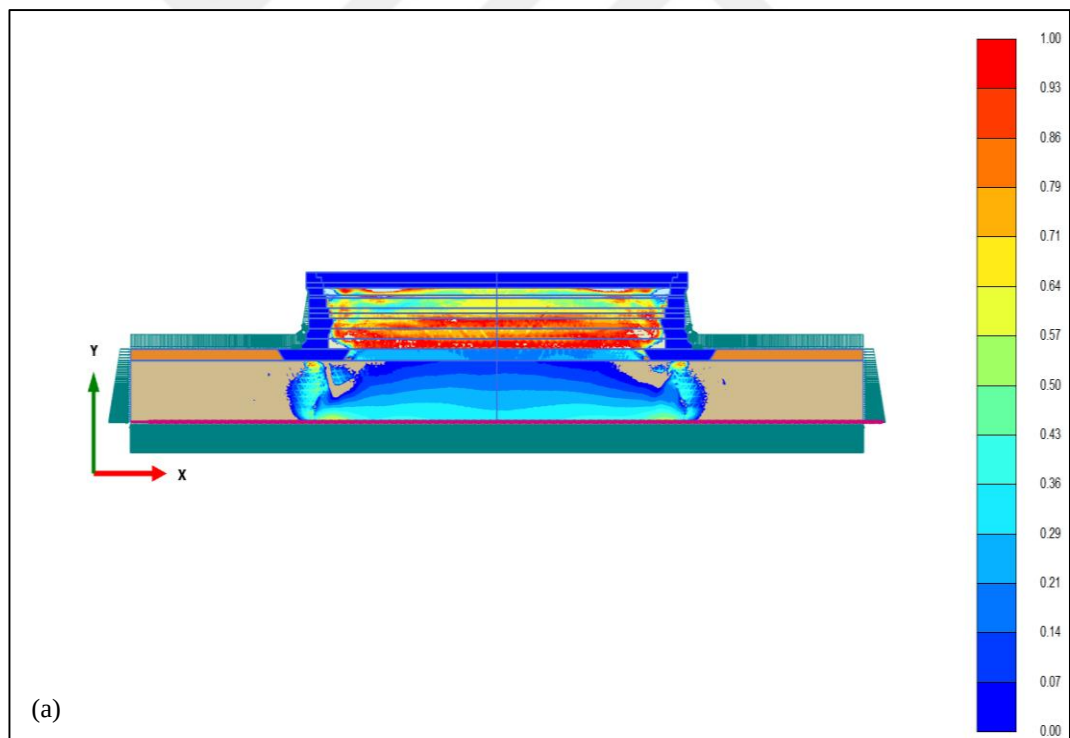


Figure 4.26 The distribution of excess pore water pressure ratio in dynamic finite element in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

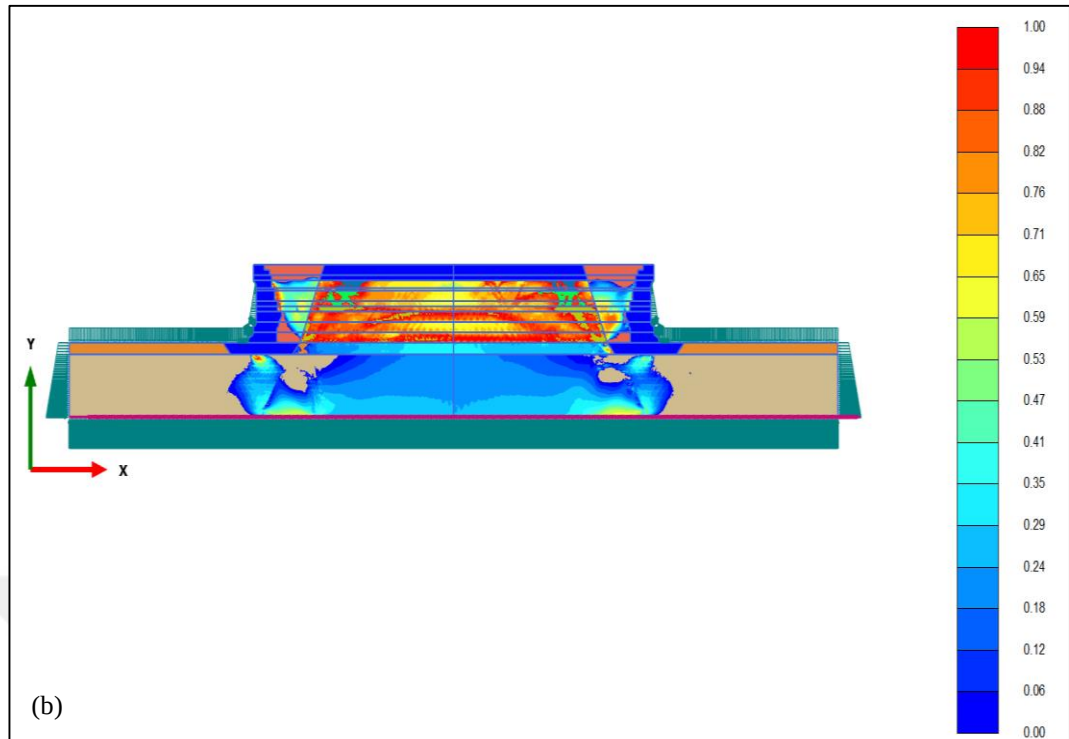


Figure 4.26 continues

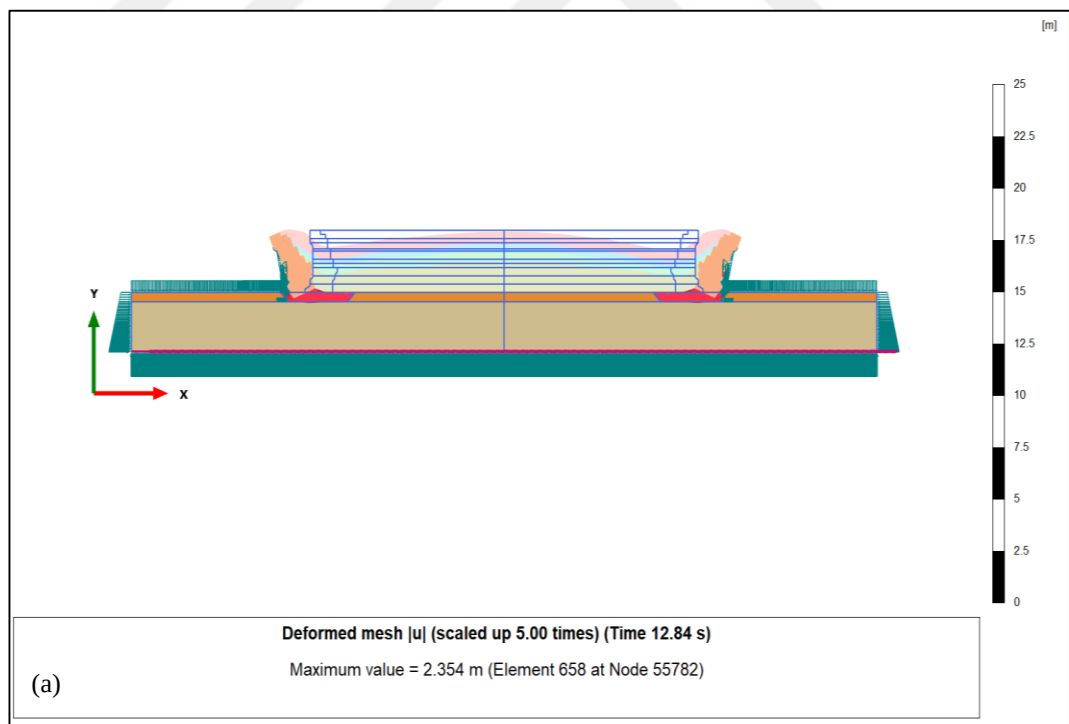


Figure 4.27 The cumulative displacement and quay wall displacement in terms of “deformed mesh” form in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

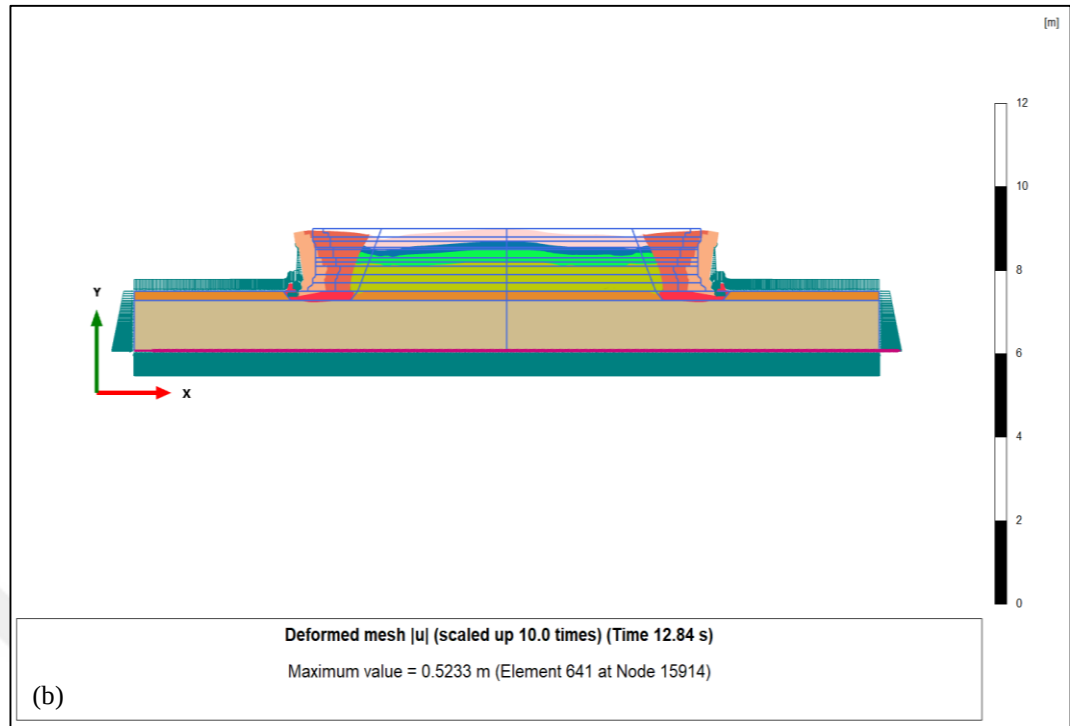


Figure 4.27 continues

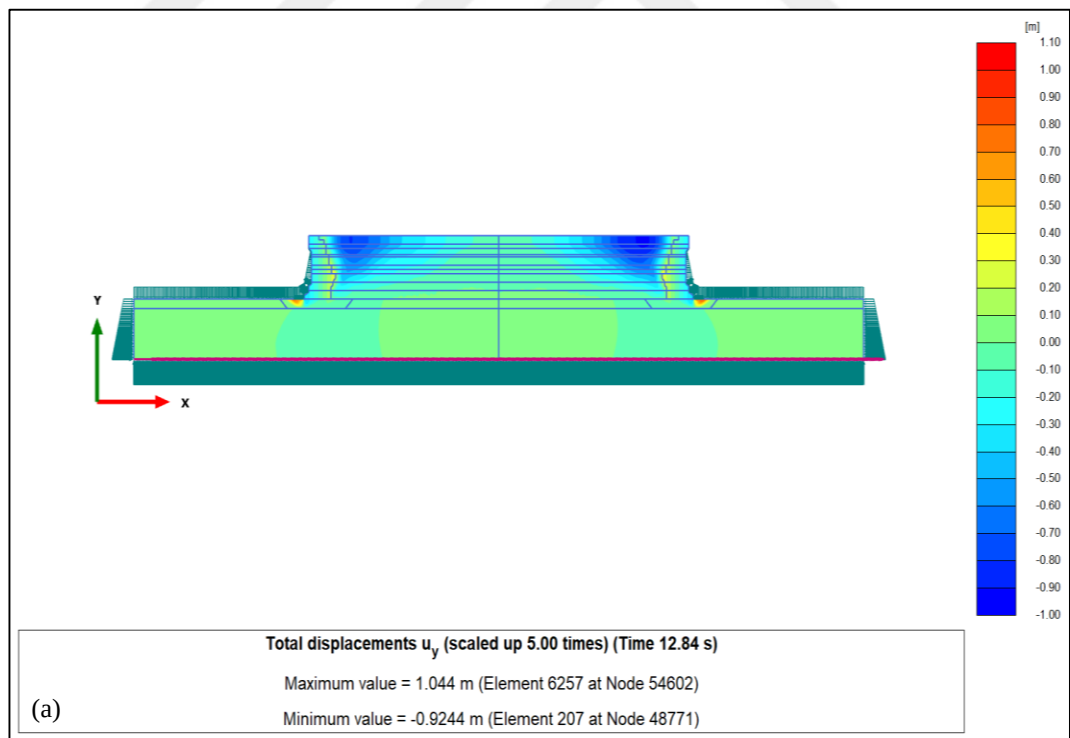


Figure 4.28 The vertical displacement distribution in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

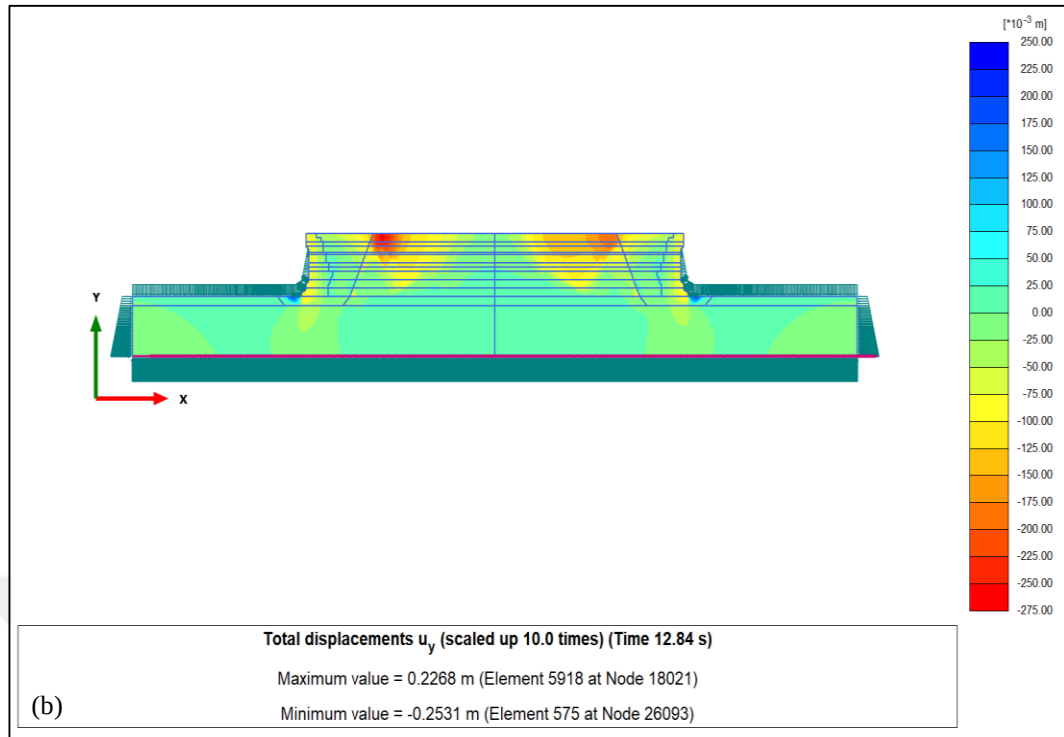


Figure 4.28 continues

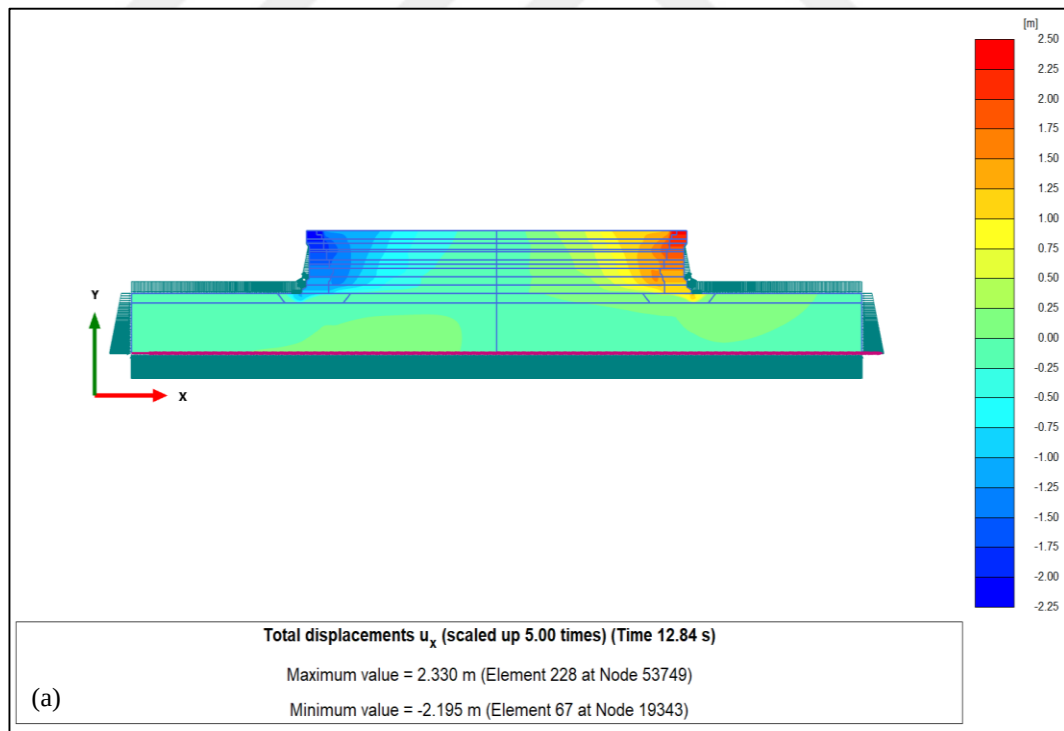


Figure 4.29 The lateral displacement distribution in Düzce Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

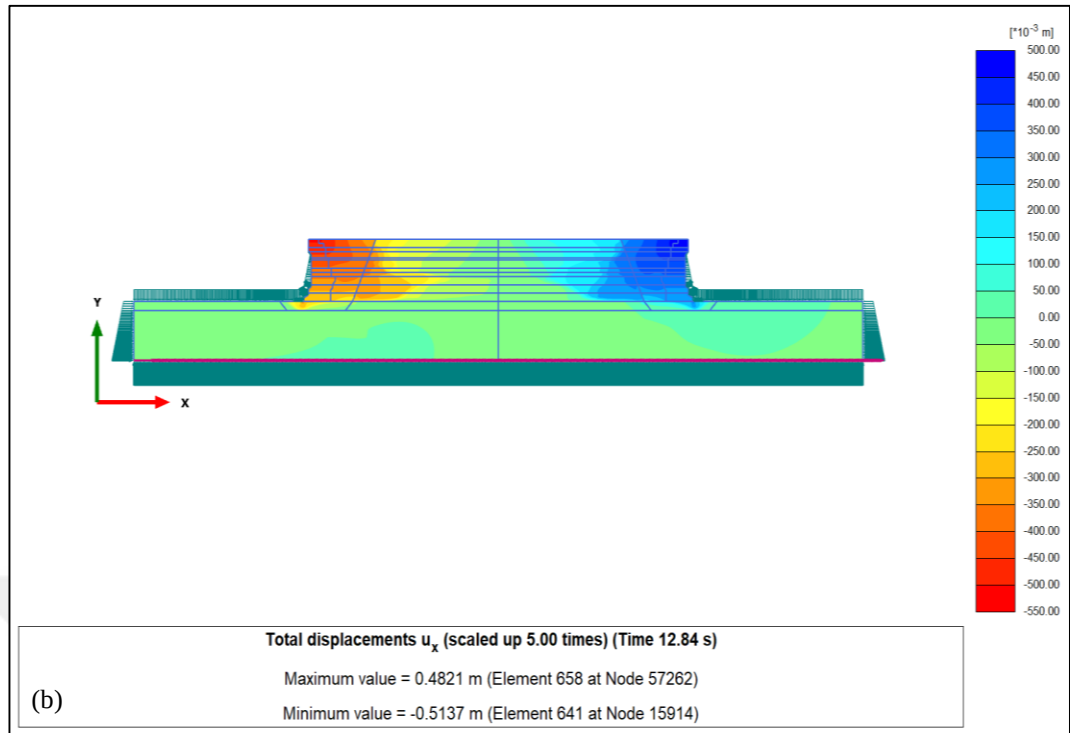


Figure 4.29 continues

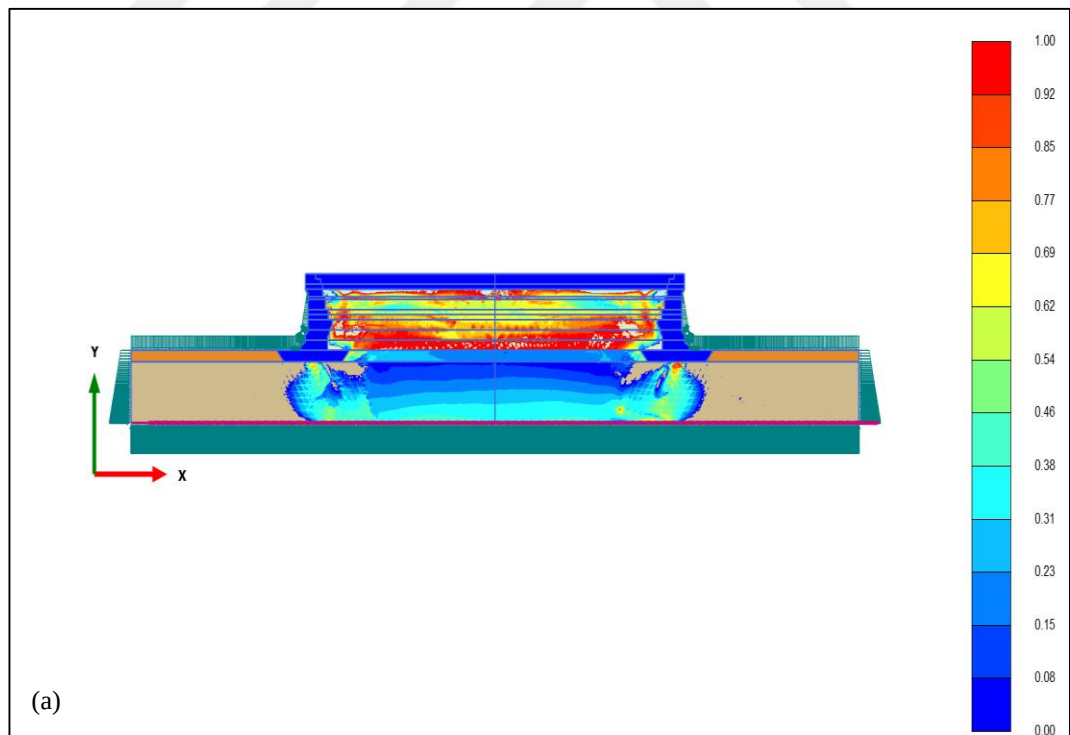


Figure 4.30 The distribution of excess pore water pressure ratio in dynamic finite element in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

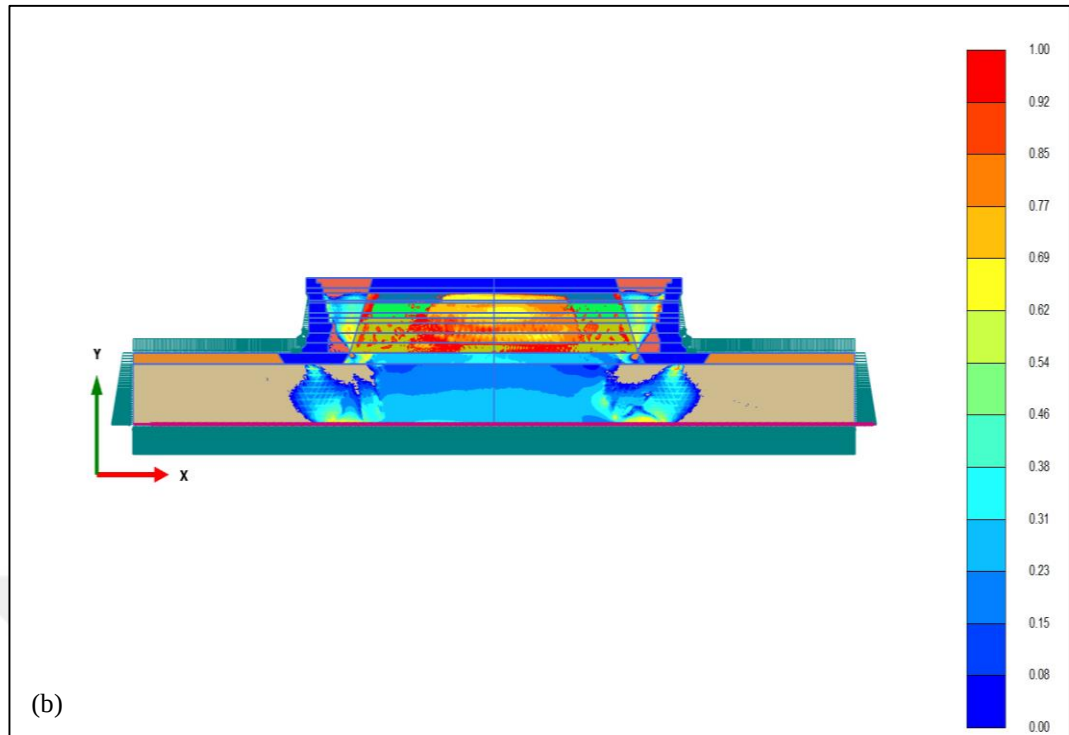


Figure 4.30 continues

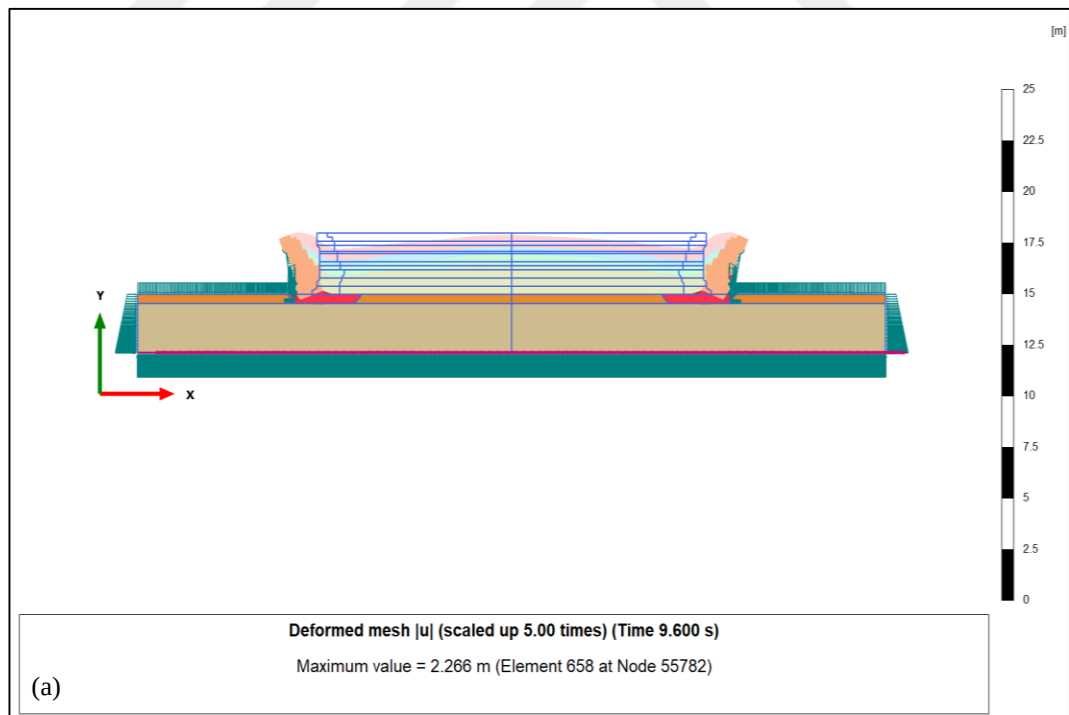


Figure 4.31 The cumulative displacement and quay wall displacement in terms of “deformed mesh” form in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

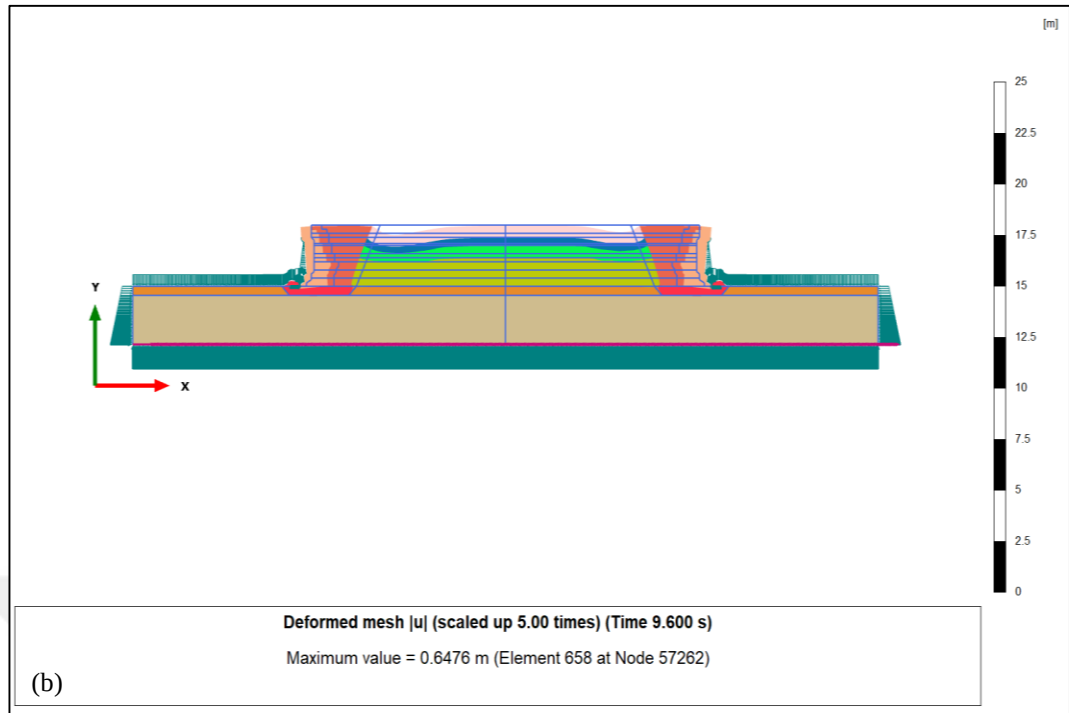


Figure 4.31 continues

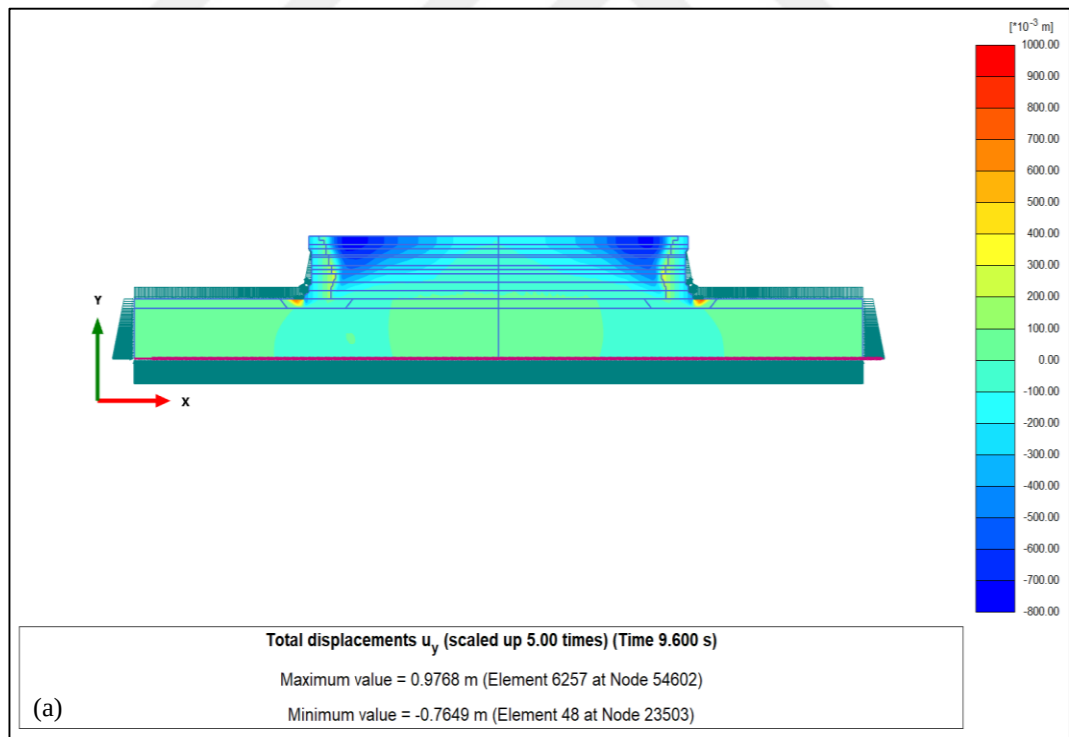


Figure 4.32 The vertical displacement distribution in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

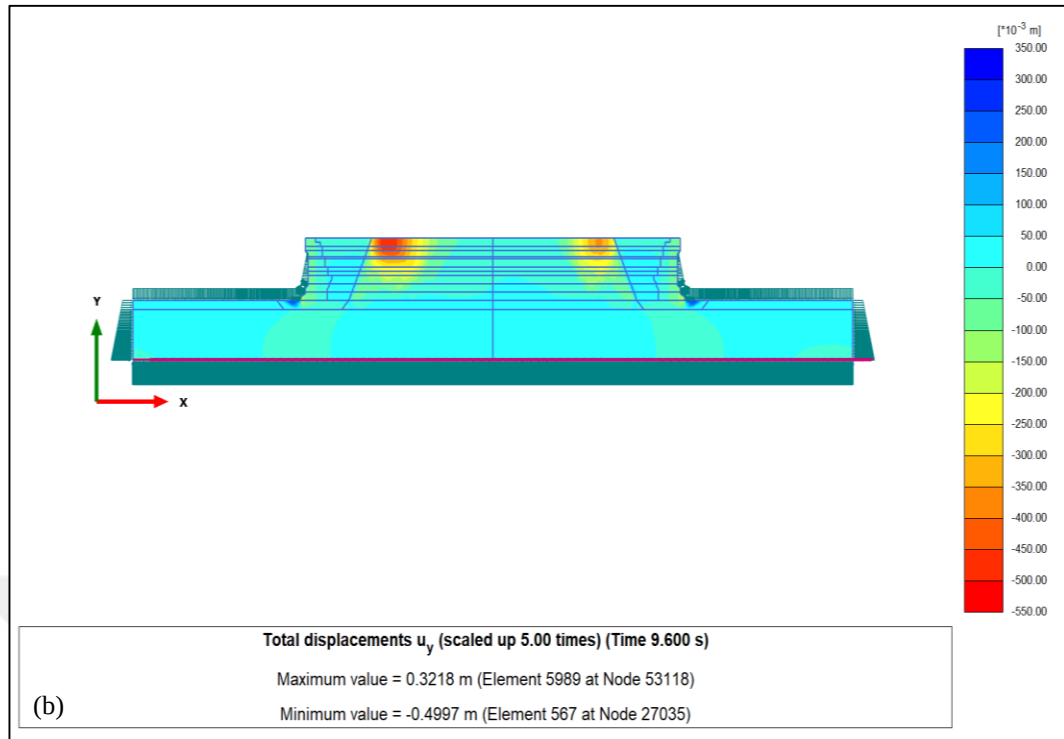


Figure 4.32 continues

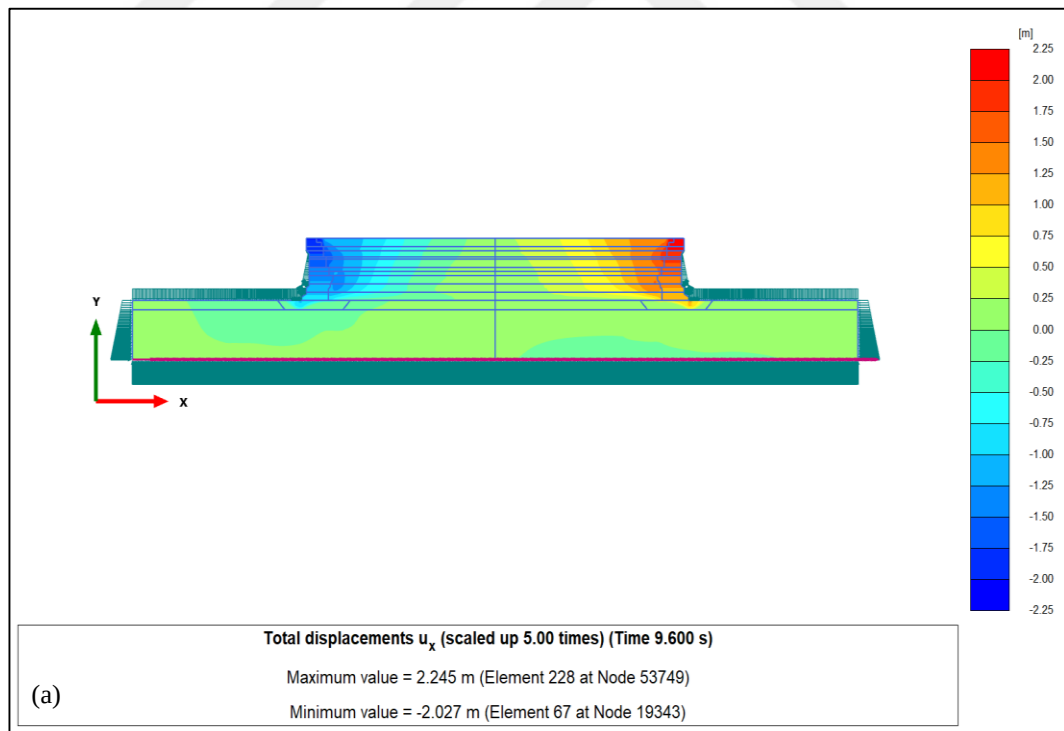


Figure 4.33 The vertical displacement distribution in Kobe Earthquake scaled according to Kocaeli Earthquake (a) stratified soil profile; (b) non-liquefiable backfill

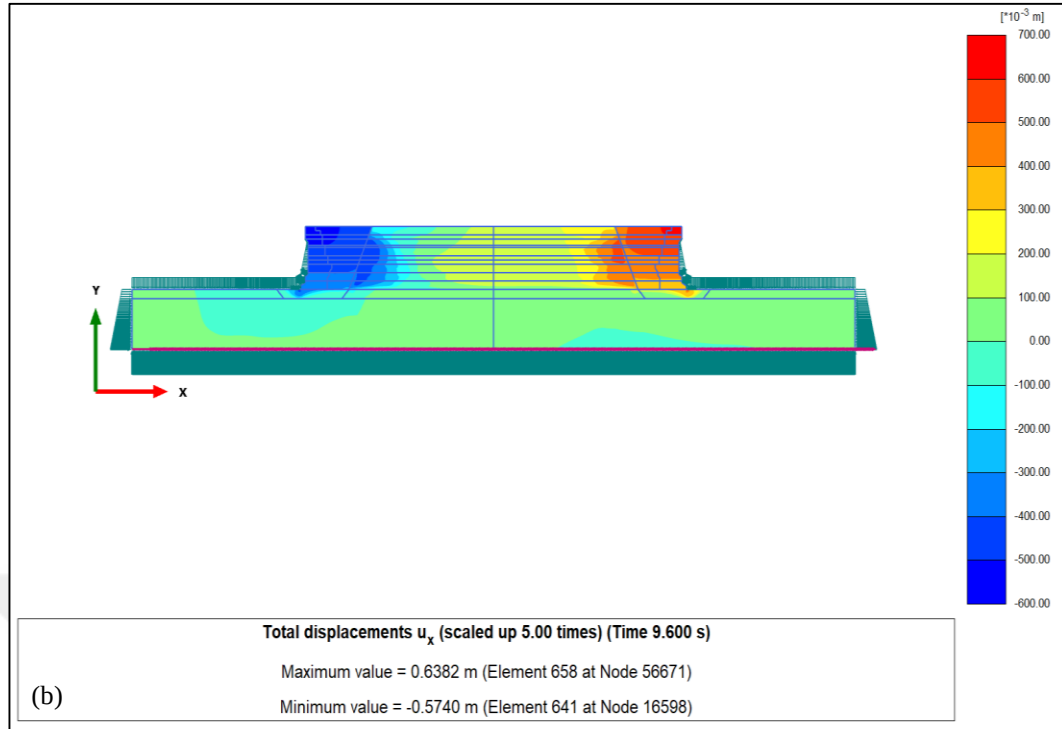


Figure 4.33 continues

4.5 Studied Method to Decrease Excess Pore Water Pressure Generation

The back analyses of the original Quay Wall No.6 shed light to certain aspects of gravity type quay wall performance during earthquakes such as wall-soil interaction and frequency content of the earthquake excitation. Since it was found that wall inertia may be quite effective on liquefaction behavior of the backfill, a technique was devised to either prevent or decrease excess pore water pressure accumulation by providing open channels on the concrete blocks of the wall (Figure 4.34).

The aim of this suggestion to wall configuration is to decrease the excess pore water pressure which occur behind the block by providing pore water pass thru and from the blocks towards the ground surface. By leaving a blank channel among the blocks, high permeability material can be defined. The inverse of the permeability coefficient of coarse-grained soil being 100 m/s is assigned to the open channel since this approach was deemed to be the only way in defining open channels inside rigid concrete blocks. The parameters regarding the assigned material model to the open channels are given in Table 4.6.

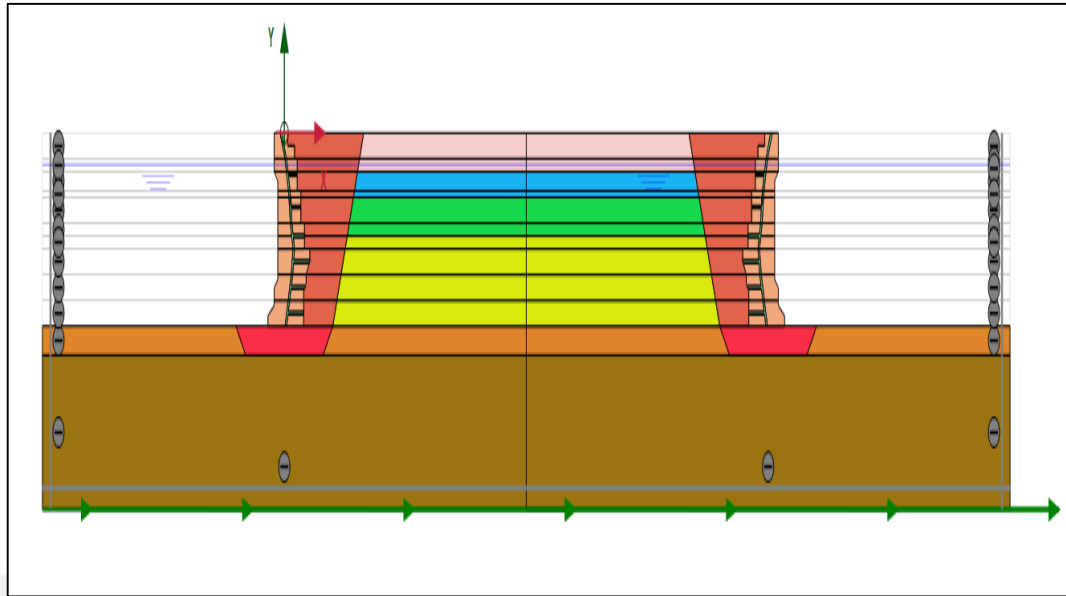


Figure 4.34 The new model geometry

Table 4.6 The assigned material parameter to open flow channels

	Linear Elastic
	Drained
γ_{unsat} (kN/m ³)	25
γ_{sat} (kN/m ³)	25
ν	0.2
E (kN/m ²)	30000000
Data set	User defined
Model	Linear
k_x (m/s)	100
k_y (m/s)	100

The coarseness factor value was used to improve the finite element mesh quality. Several combinations of the factors were tried to obtain a reasonable FEM mesh suitable for dynamic finite element analysis. One should note that too intense mesh may inhibit completion of earthquake analyses due to increased computational time. The coarseness factor values which are assigned to each soil cluster are shown in Table 4.7:

Table 4.7 The coarseness factor for polygon/cluster

The coarseness factor	Polygon
0.2	Concrete block
0.3	Subbase filling
0.3	Soil
0.75	The bottom layer

The analysis in the new model is repeated for three earthquake accelerograms in the stratified soil profile with non-liquefiable backfill. When the change in excess pore water pressure ratio in time is examined, it is seen that there is a same results with undrained model (Figure 4.35a~4.35c).

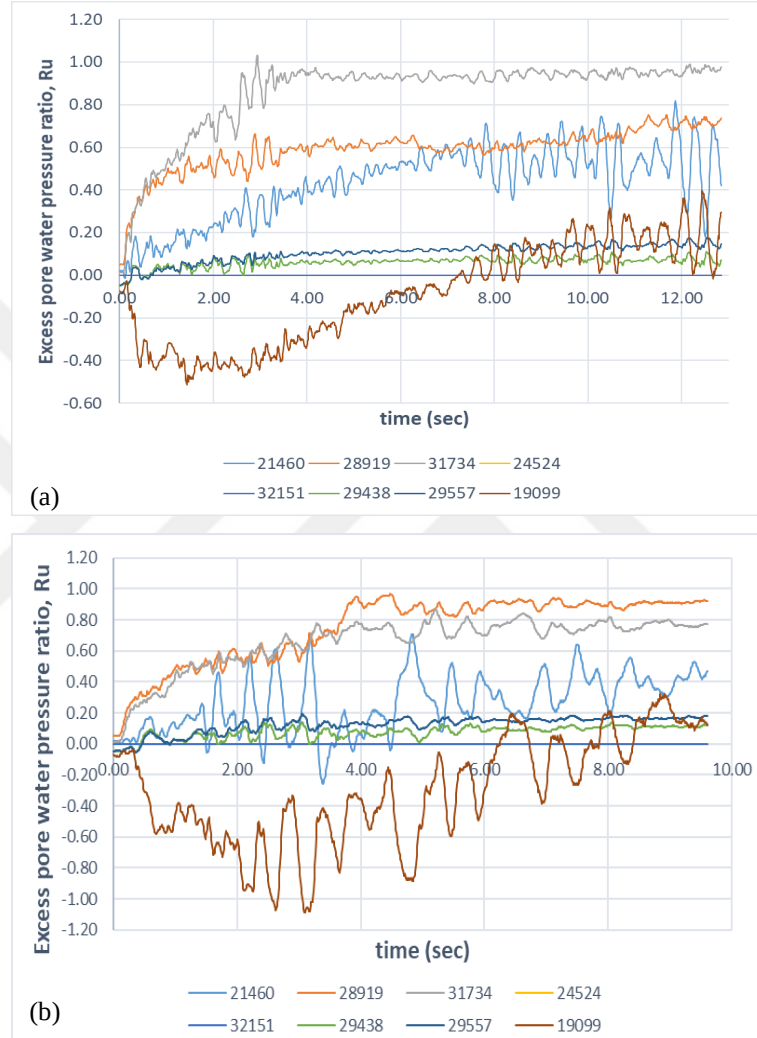


Figure 4.35 The change of excess pore water pressure ratio in stratified soil profile with non-liquefiable backfill in drained model (a) Düzce Earthquake scaled according to Kocaeli Earthquake; (b) Kobe Earthquake scaled according to Kocaeli Earthquake; (c) Kocaeli Earthquake

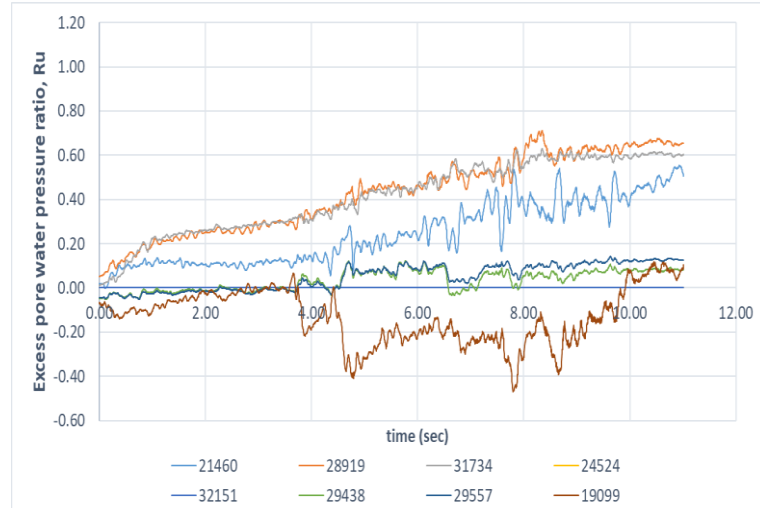


Figure 4.35 continues

When the change of excess pore water pressure ratio is compared (i.e. Figure 4.20 versus 4.35), it is observed that the excess pore water pressure ratio pressure ratio changes in small quantities or remains the same. The reason behind why there is small change or same, can be explained as the chosen of “tied degrees of freedom” for the vertical model boundaries. The detailed graphics which compare the liquefaction of the original wall and the new wall with open channels inside are displayed for Kocaeli Earthquake in Figures 4.37. These graphics show variation of pore water ratio with depth for each cross section given in Figure 4.36. The analyses also provided similar results with undrained model. The results did not exhibit a positive trend and excess pore pressure ratio did not decrease with the presence of open drainage channels on concrete blocks. On the other hand, it should be noted that the non-liquefiable backfill is positive influence behind the wall (Figure 4.38). As moving away from quay wall, the excess pore water pressure ratio changes in small quantities or remains the same.

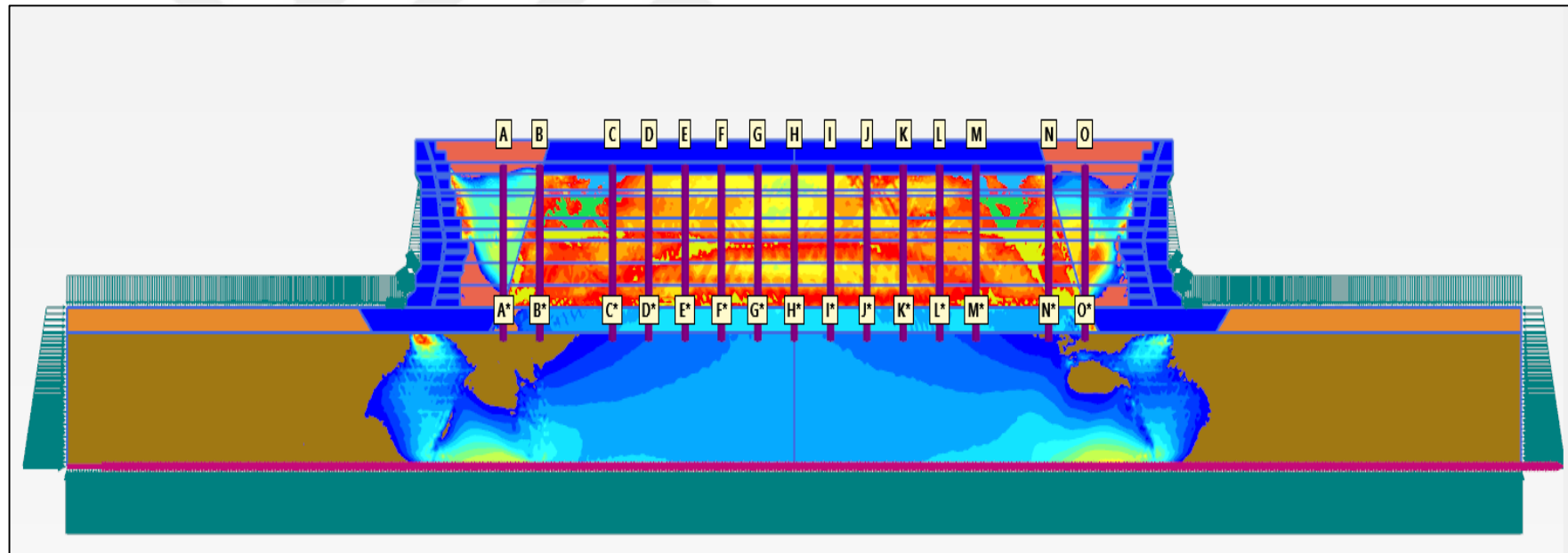


Figure 4.36 The cross-sections for reviewing liquefaction behavior

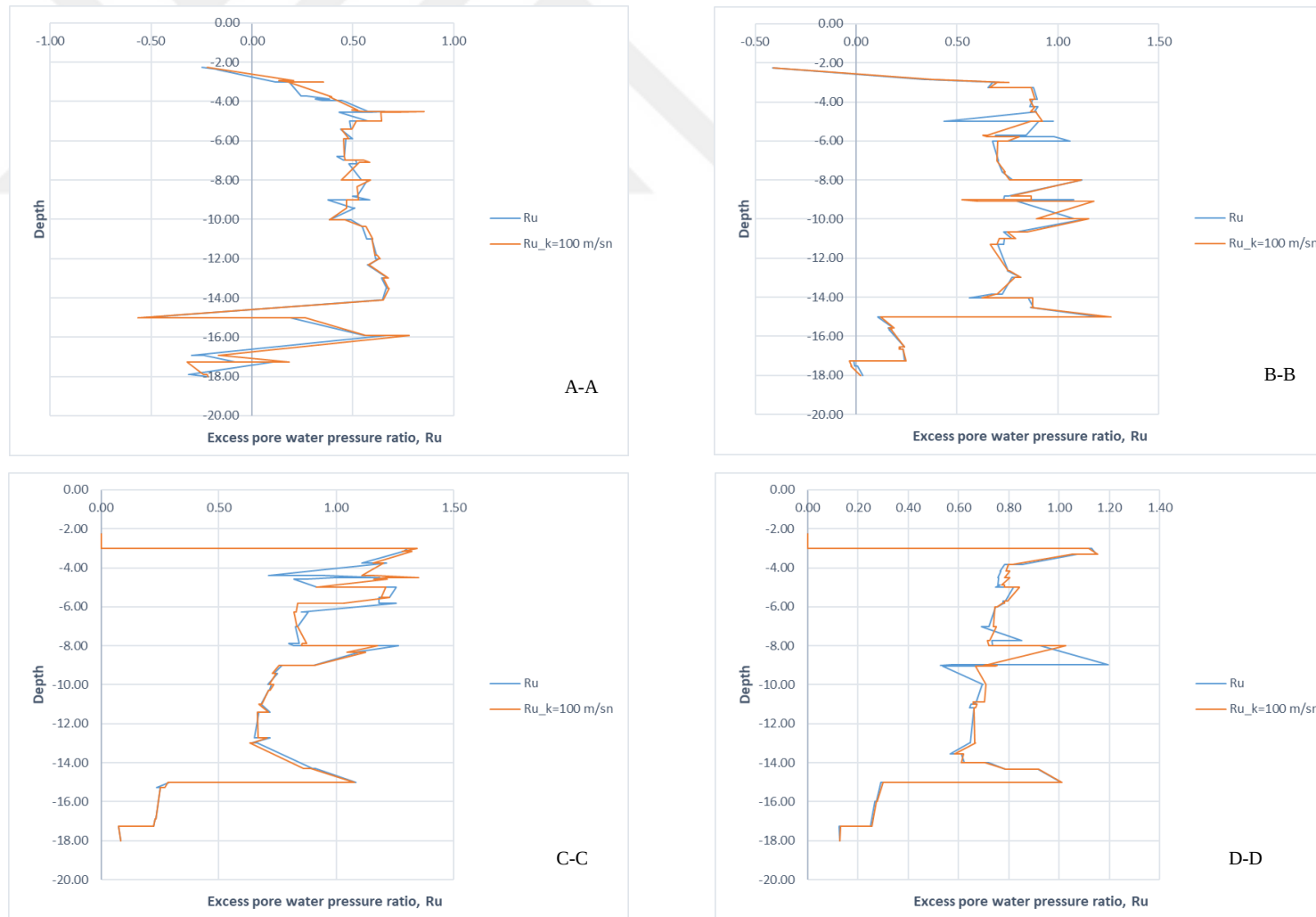


Figure 4.37 Comparison of excess pore water ratio between the original wall and new model for Kocaeli Earthquake

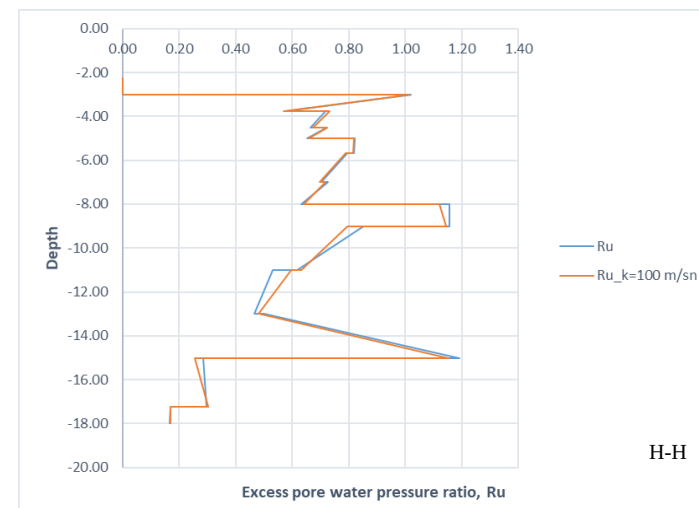
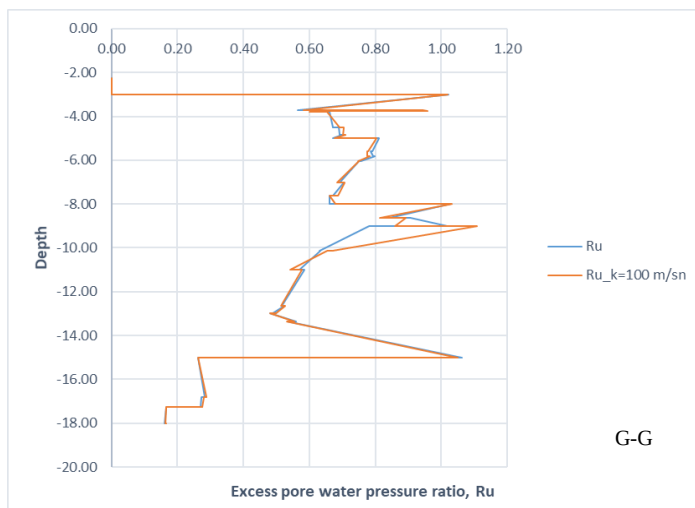
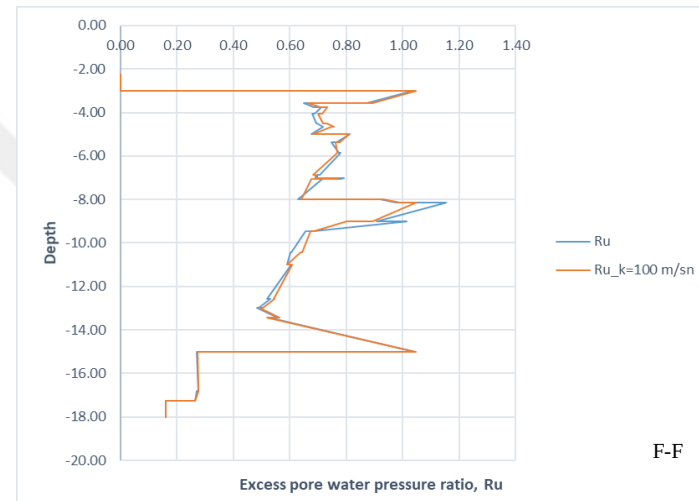
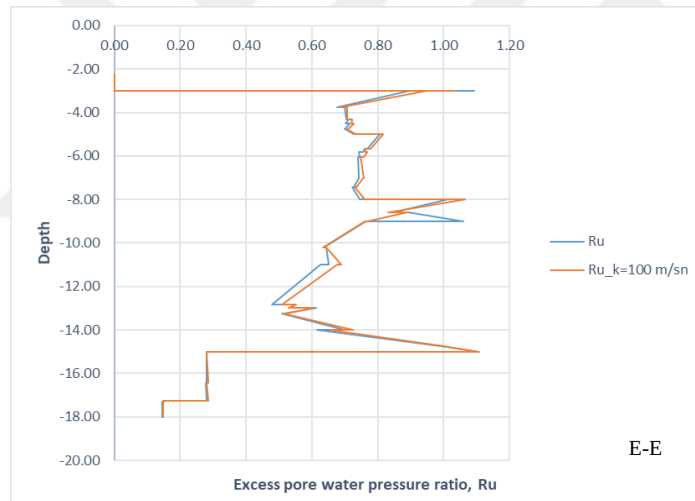


Figure 4.37 continues

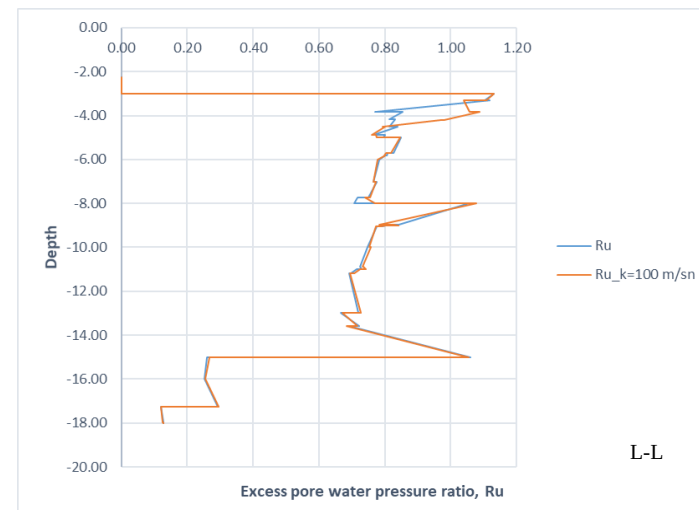
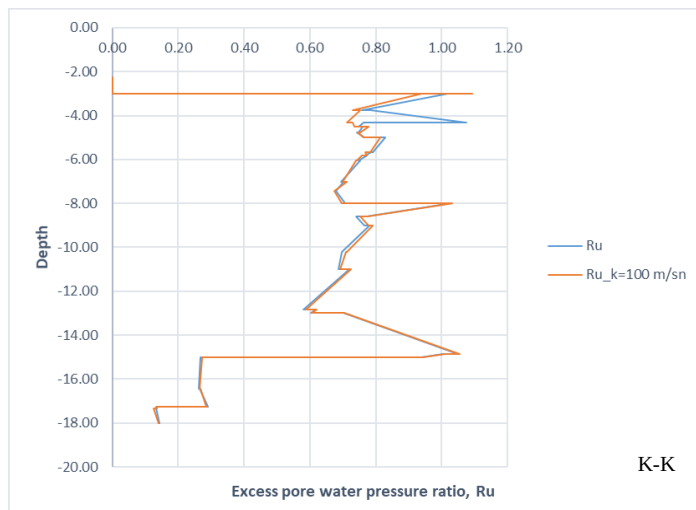
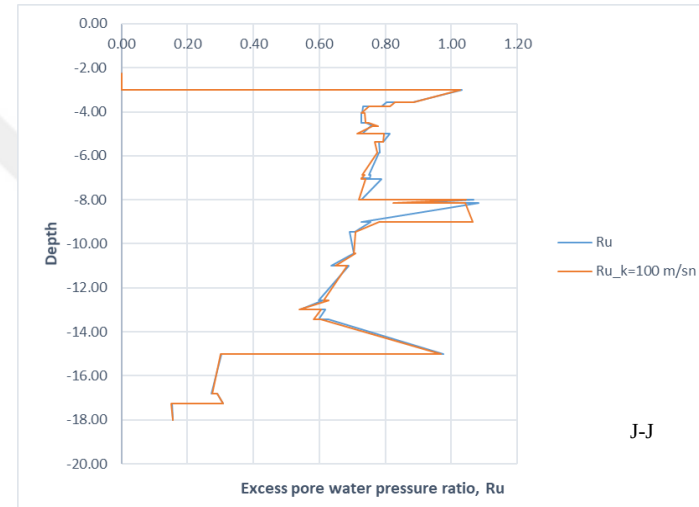
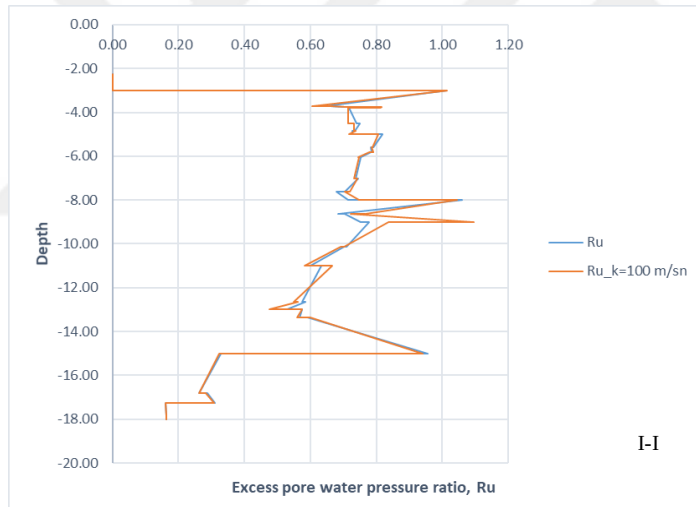


Figure 4.37 continues

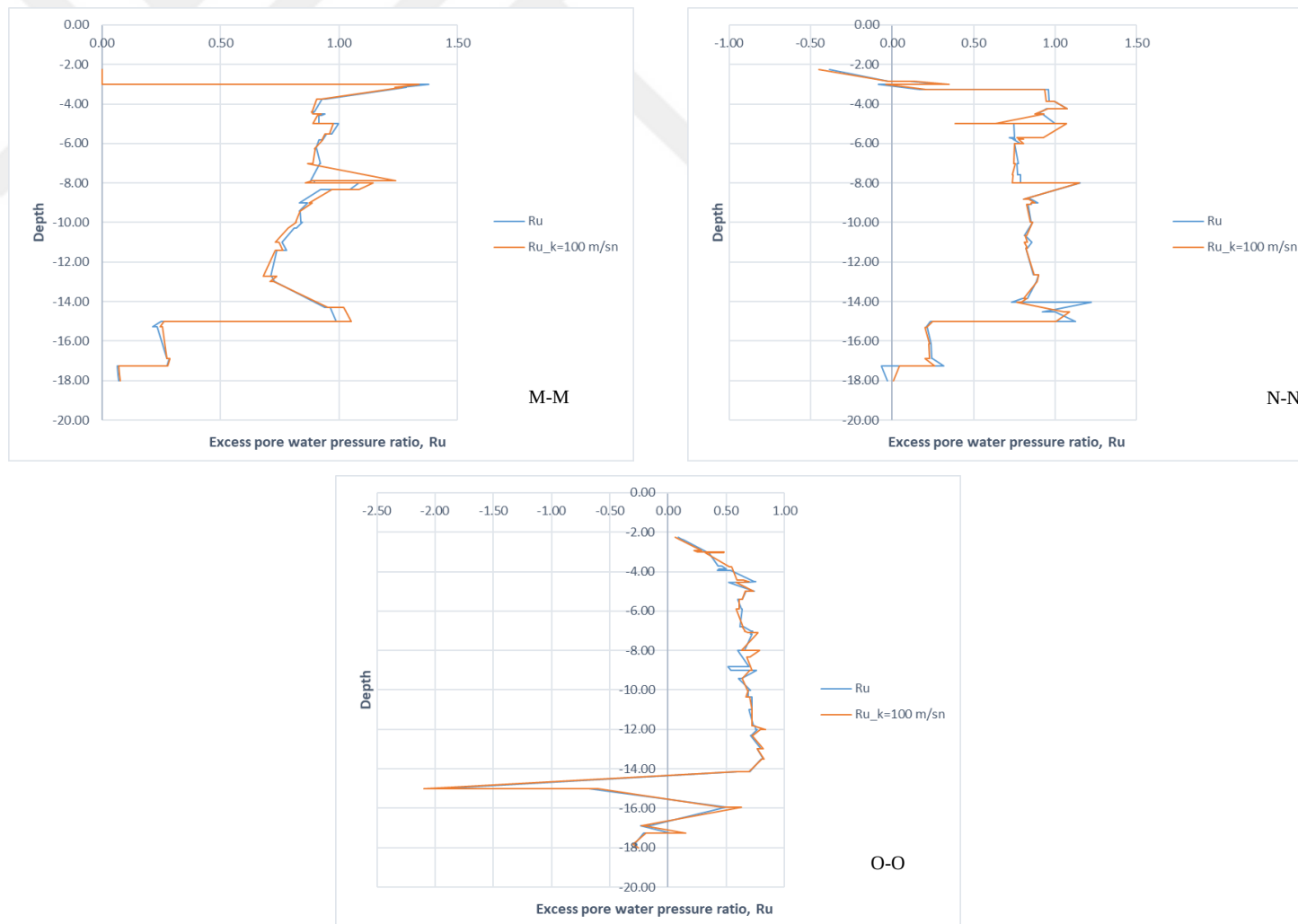


Figure 4.37 continues

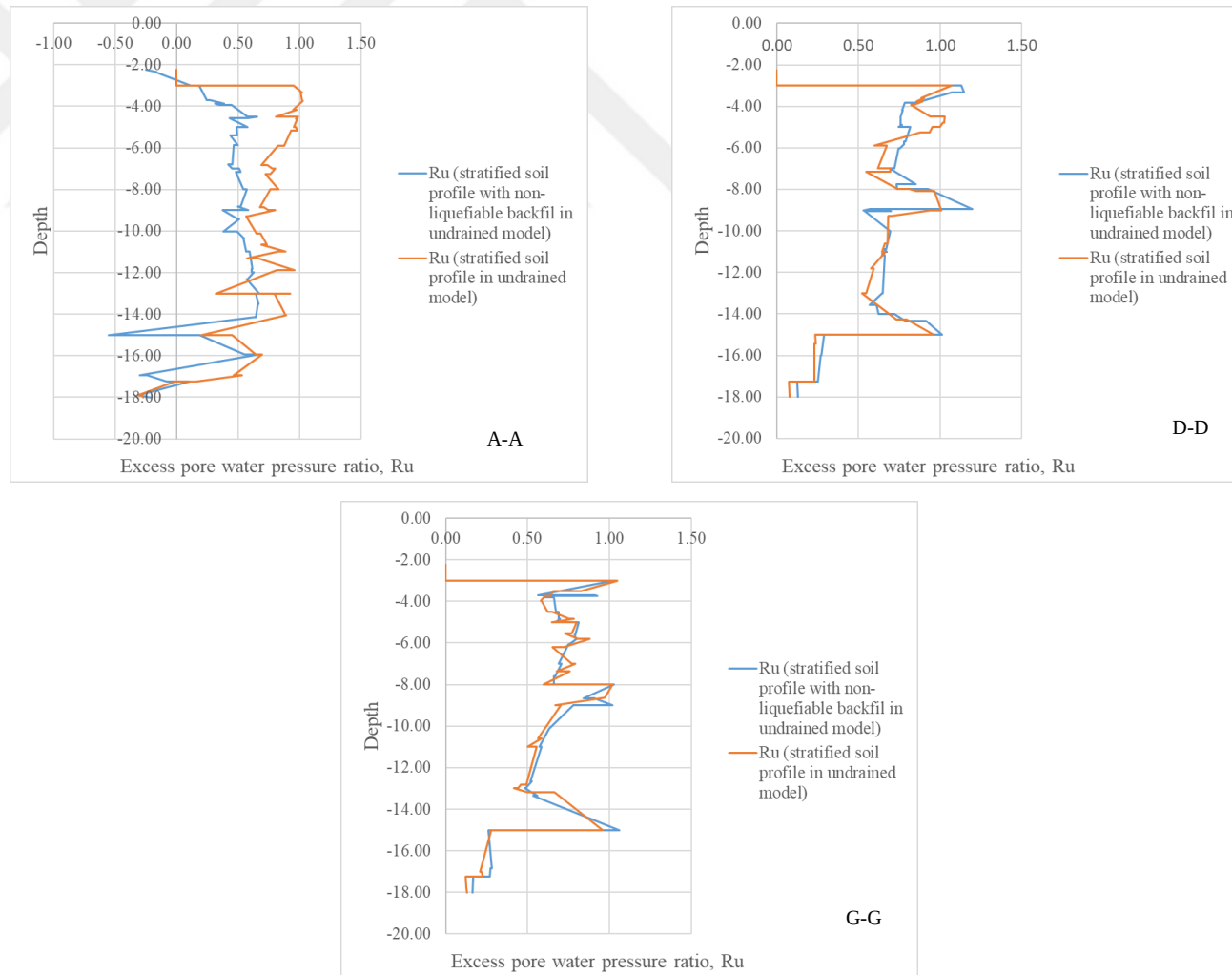


Figure 4.38 The excess pore water pressure ratio is compared between stratified soil profile with non-liquefiable backfill and stratified soil profile

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

In this study, backward analyses of No.6 Gravity Quay Wall in Derince Port, where damages were observed as a result of liquefaction occurred in 17th August 1999 Marmara Earthquake, were performed using dynamic finite element method. The behavior of liquefied soil was accounted for by means of UBC3D-PLM material model. Three separate earthquake records were applied to the model so that the effect of frequency content and peak ground acceleration on soil and wall response could be examined.

The results revealed that the peculiar characteristics of the soil were not the sole governing factor on the liquefaction behavior behind the quay wall. The frequency content of the earthquake and gravity quay wall-soil interaction were also found to be effective. Accumulation of excess pore water pressure happened to be almost equal at the points where response was recorded excluding Point #:29783 which is right behind the wall at almost its mid-height. At that location r_u reached 0.8 for Düzce Earthquake scaled with respect to the PGA of Kocaeli Earthquake. It can be concluded that frequency content played a considerable role on excess pore water pressure development. Although peak acceleration in three earthquakes is same, excess pore water pressure development in Düzce Earthquake is higher than Kobe and Kocaeli Earthquakes. This can be explained with the existence of higher energy harmonics in Düzce Earthquake. One may conclude that higher period harmonics of the earthquake captured already softening liquefiable soil keeping it within the resonance region throughout the analysis. Therefore, the Düzce Earthquake record resulted in widespread liquefaction behind the wall. Also, the small governing period of the quay wall caused larger wall displacements under the action of Kobe Earthquake involving harmonics with quite smaller periods as compared with other earthquakes. Dynamic analyses results showed that response of the liquefiable backfill should be discussed considering soil-structure interaction and frequency content of the earthquake records.

A separate FE model was also realized in order to check effectiveness of a new way of liquefaction prevention technique. For this purpose, open channels were set on quay wall blocks to provide drainage paths for excess pore water pressure that would accumulate in the liquefiable sand so that liquefaction onset could be completely avoided or minimized as much as possible. The methodology followed for the realization of the FE model has already been given in the fourth chapter of this thesis. When the results obtained in both models are compared, it is observed that the excess pore water pressure ratio changes in small quantities or remains the same. The reason behind why the results did not exhibit a positive trend and excess pore pressure ratio did not decrease with the presence of open drainage channels on concrete blocks can be explained as the chosen of “tied degrees of freedom” for the vertical model boundary. The positive influence of drainage channels, however, is observed on FE analyses performed using free field boundary modelling method. The results of those analyses can be seen in Appendix III.

The FE runs that were made within the scope of this thesis consisted of three different backfill types behind the wall. Although the wall geometry was exactly the same in three models, the results varied significantly and only the results of the model where the backfill right behind the wall consists of non-liquefiable rock chunks did match the post-earthquake displacement profile measured in the field demonstrating positive influence of such a backfill. The non-liquefiable backfill prevented liquefaction behind the wall. The comparison of analysis results between stratified soil profile and stratified soil profile with non-liquefiable backfill showed that the excess pore water pressure ratio behind the wall was prevented, but as moving away from the quay wall excess pore water pressure ratio changed in small quantities or remained the same. Also, the results showed that negative pore water pressure occurred behind the wall in the non-liquefiable backfill. This can be explained as dilatation of dense backfill under earthquake loading. The saturated dense soil exhibits expansion under dynamic load, so the effective stress decreases near ground surface since effective stresses are lower making dilatation of dense soil easier. In other respect, the positive

trend was also observed on displacements thanks to non-liquefiable backfill. It is understood that the choice of backfill material behind the wall is essential.

The maximum horizontal displacement measured in the field following the 1999 Marmara Earthquake for the Gravity Quay Wall No.6 is 30 cm at the far edge and the displacement at the mid-span of the wall can be interpolated as 15 cm. The dynamic horizontal displacement calculated in the Kocaeli Earthquake analysis was on the order of 27 cm being in accordance with the field performance of the wall. Considering the damage levels as specified in the State Harbors and Airports Technical Earthquake Code one may evaluate the wall's performance as controlled damage (CD).

5.2 Recommendations

In this thesis, back analyses of a gravity type quay wall were made using both conventional and numerical methods. Numerical analyses revealed one more time that pseudo-static method to check the factor of safety against sliding and over turning is far away from representing performance of the wall in terms of displacements and stiffness decrease due to excess pore water pressure accumulation. Therefore, further analyses are necessary in order to clarify and possibly modify the code requirements by considering wall-soil interaction where the frequency content of the earthquake excitation may be significant. One may notice in Appendix I and II that design checks for the PGA recorded in Kocaeli Earthquake and L2 level earthquake according to Turkish Seismic Design Code of Ports and Harbours resulted in failure for most wall blocks whereas FE analyses for non-liquefiable backfill case showed major deformations without collapse.

REFERENCES

- Algermissen, S.T. (1985). Preliminary report of investigations of the Central Chile Earthquake of March 3, 1985. *U.S. Geological Survey*, 1-182.
- Beatty, M.H. & Byrne, P.M. (2011). *UBCSAND constitutive model version 904aR*. Retrieved May, 12, 2019 from http://www.itasca-udm.com/media/download/UBCSand/UBCSAND_UDM_Documentation.pdf
- Borg, R.C. (2007). *Seismic performance, analysis and design of wharf structures: a comparison of worldwide typologies*. M.Sc. Thesis, Rose School, Italia.
- Brinkgreve, R.B.J., Kumarswamy, S., Swolfs, W.M., Zampich, L. & Manoj, N.R. (2019). *Plaxis material models manual*. Retrieved January 2019, from <https://www.plaxis.com/support/manuals/plaxis-2d-manuals/>
- Chen, C.H. & Hwang, G.S. (2000). Preliminary analysis for quay wall movement in Taichung Harbor during the September 21, 1999, Chi-Chi Earthquake. *Earthquake Engineering and Engineering Seismology*, 2(1), 43-54.
- Daftari, A. & Kudla W. (2014). Prediction of soil liquefaction by using UBC3D-PLM model in Plaxis. *World Academy of Science, Engineering and Technology International Journal of Civil and Environmental Engineering*, 8(2), 106-111.
- Ergin, A. & Yüksel, Y. (2006). Deprem ve tsunami yükleri altında kıyı yapıları tasarımına yeni bir yaklaşım: Davranışa Dayalı Tasarım. *Sigma, Mühendislik ve Fen Bilimleri Dergisi*, 2006/2, 1-13
- Galavi, V., Petalas, A., & Brinkgreve, R.B.J. (2014). Finite element modelling of seismic liquefaction in soils. *Geotechnical Engineering Journal of the SEAGS & AGSSEA*, 44(3), 55-64

- Groot, M.B, Bolton M.D., Foray, P., Meijers, P., Palmer, A.C., Sandven R., et al. (2016). Physics of liquefaction phenomena around marine structures. *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 132(4), 227-243.
- Hamamoto, T., Nagano, T. & Yamao, N. (2016). *Analytical study of the sheet pile quay damaged by 2005 Fukuoka Earthquake*. Retrieved April 28, 2019, from https://iirrr.ucalgary.ca/files/iirrr/A5-4_.pdf
- Iai, S & Ichii, K. (1998). Performance based design for port structure. *Wind and Seismic Effects, Proceeding of the 30th Joint Meeting*, 84-96.
- Iai, S. (2011). Overview of backwards analysis in geotechnical engineering. *International Symposium on Backwards Problem in Geotechnical Engineering and Monitoring of Geo-Construction*, (2011), 29-34
- Inagaki, H., Iai, S., Sugano, T., Yamazaki, H. & Inatomi, T. (1996). Performance of caisson type quay walls at Kobe Port. *Japanese Geotechnical Society, Special Issue of Soils and Foundations*, 119-136.
- Karakuş, H. (2007). *New seismic design approaches for block type quay walls*. M.Sc. Thesis, METU, Ankara.
- Kazama, M. & Yanagisawa, E. (2000). Earthquake motion intensity from liquefaction viewpoint. *12th World Conference on Earthquake Engineering*, 2, 1-8.
- Kokusho, T. (Ed.) (2009). *Earthquake geotechnical case histories for performance-based design*. (1st ed.). New York: CRC Press.
- Makra, A. 2013. *Evaluation of the UBC3D-PLM constitutive model for prediction of earthquake induced liquefaction on embankment dams*. M.Sc. Thesis, Delft University of Technology, Delft.

- Miura, K., Kohama, E., Inoue, E., Ohtsuka, N., Sasajima, T., Hayashi, T., et al. (2000). Behavior of gravity quay wall during earthquake regarding dynamic interaction between caisson and backfill during liquefaction. *12th World Conference on Earthquake Engineering*, 1-8.
- Na, U.J., Chaudri, S.R. & Shinozuka M. (2008). Probabilistic assessment for seismic performance of port structures. *Soil Dynamic Earthquake Engineering*, 28(2), 147-158
- Petalas, A. & Galavi, V. (2013). *Plaxis liquefaction model UBC3D-PLM*. Plaxis Report, The Netherlands, Delft.
- PIANC (2001). *Seismic design guidelines for port structure*. Tokyo: International Navigation Association, A.A. Balkema Publishers.
- Pitilakis, K. & Moutsakis, A. (1989). Seismic analysis and behavior of gravity retaining walls-the case of Kalamata Harbor quay wall. *Soils and Foundations* 29(1), 1-17.
- Sümer, B.M., Ansal, A., Çetin, K.O., Damgaard, J., Günbak, A.R., Hansen, N.O., et al. (2007). Earthquake-Induced Liquefaction around Marine Structures. *Journal of Waterway, Port, Coastal, and Ocean Engineering*, 133(1), 55-82.
- Takahashi, H., Oohashi, H. & Endoh, H. (2012). Earthquake damage investigation of gravity-type quay walls in Ibaraki Port. *International Symposium on Engineering Lessons Learned from the 2011 Great Japan Earthquake*, 687-696.
- Tiznado, J.C. & Roa, F.R. (2010). Seismic lateral movement prediction for gravity retaining walls on granular soils. *Soil Dynamics and Earthquake Engineering* 31, 391–400.

Tsegaye, A. B. (2010). *Liquefaction model (UBC3D)*. Plaxis Report, The Netherlands, Delft.

Youd, T.L. & Idriss, I. M. (2001). Liquefaction resistance of soils: summary report from the 1996 Nceer and 1998 Nceer/Nsf workshops on evaluation of liquefaction resistance of soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 297-313

Yüksel, Y., Özmen, H., Çevik, E., Özgüven, O., Çelikoğlu, Y., Bostan, T., et al. (2000). Doğu Marmara Depreminin körfez bölgesi deniz yapıları üzerindeki etkileri. *III. Ulusal Kıyı Mühendisliği Sempozyumu*, 67-91.

Yüksel, Y., Alpar, B., Yalçiner, A.C., Çevik, E., Özgüven, O. & Çelikoğlu, Y. (2002). Effects of the Eastern Marmara Earthquake on marine structures and coastal areas, *Proceedings of The Institution of Civil Engineers. Water & Maritime Engineers* 156, Issue WM2, 147-163.

Yüksel, Y. & Güler, I. (2005). Deniz yapılarında deprem etkileri ve 1999 Gölçük Depremi örneği. *TMH-Türkiye Mühendislik Haberleri*, 2005/4, 74-79.

Yüksel Proje (2007). Kıyı ve Liman Yapıları, Demiryolları ve Hava Meydanları İnşaatlarına İlişkin Deprem Teknik Yönetmeliği. Demiryolları, Limanlar ve Hava Meydanları (DLH) Genel Müdürlüğü.

Yüksel, Y. & Çevik, E., (2010). *Liman Mühendisliği*, İkinci Baskı. İstanbul: Beta Yayıncılık.

APPENDICES

APPENDIX-I: Conventional Analysis for L2 Earthquake Level

Design Parameter

Table A.1 Design Parameter for Analysis

ϕ	δ^*	β	θ	H	H _{sub}	q _s	q _{EA}	Bollard	H _c
(°)	(°)	(°)	(°)	(m)	(m)	(kN/m ²)	(kN/m ²)	(kN)	(m)
38	12.67	0	0	15.00	12.50	30	15	800	0.8

γ_{conc}	$\gamma_{\text{dry_layer\#1}}$	$\gamma_{\text{sat_layer\#1}}$	$\gamma_{\text{dry_layer\#2}}$	$\gamma_{\text{sat_layer\#2}}$	γ_{sub}	$\gamma_{\text{conc,sub}}$	γ_{w}
(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)
24	16	18	16	19	9	14	10

*The friction angle between wall and soil was accepted as $\phi/3$ for the all calculations.

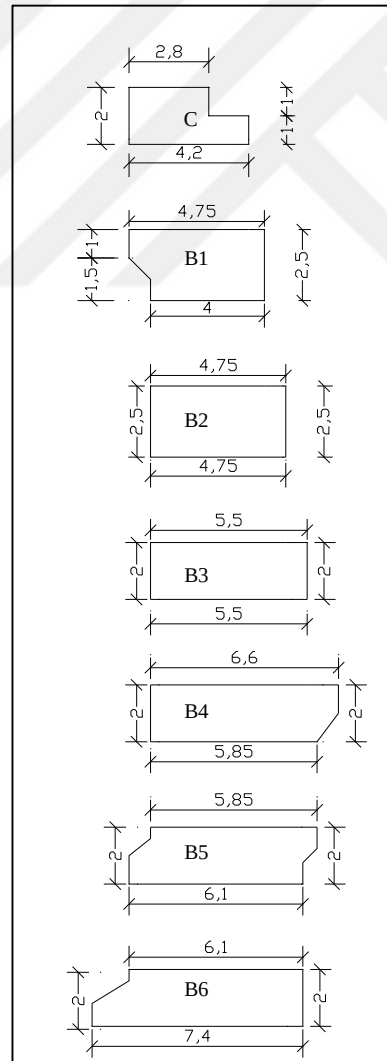


Figure A.1 Dimension of block type quay walls (unit: m)

Seismic Coefficient

Derince Port coordinates are 29.5 longitude and 40.4 latitude. Spectral acceleration for the short period is determined by TSDC-CDR (2007), as below for 10% probability of exceedance in 50 years

$$S_s = 1.32 \text{ \%10 exceedance probability (L2)}$$

- The short period soil coefficient (F_a) for B soil classification is obtained by using Table A.2.

Table A.2 The short period soil coefficient (F_a), (TSDC-CRA, 2007)

Soil Class	Spectral acceleration (g) ^a for T=0.2 sec				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.0$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	_b	_b	_b	_b	_b

^a Interpolation is made to find S_s .
^b Special geotechnical analysis and dynamic soil attitude analysis is made.

$$F_a = 1 \text{ for } S_s = 1.50$$

- The spectral acceleration value is obtained below formulation.

$$S_{ms} = F_a \times S_s$$

$$S_{ms} = 1.32 \times 1 = 1.32$$

- Effective seismic coefficient for probability of exceedance %10;

For L2 earthquake motion;

$$A_{20} = 0.4 \times 1.32 = 0.528 \text{ (a}_{\max}/g)$$

- Horizontal seismic coefficient for L2 earthquake motion;

$$k_h = (1/3) \times 0.528^{(1/3)} = 0.27$$

- The modified seismic coefficient, k'_h ;

$$k'_h = \frac{\sum_{j=1}^{ND} (\gamma_j h_j) + \sum_{j=ND+1}^N (\gamma_{sj} h_j) + q_0}{\sum_{j=1}^{ND} (\gamma_j h_j) + \sum_{j=ND+1}^N (\gamma_{bj} h_j) + q_0} \times k_h$$

$$k'_h = \frac{16 \times 2.5 + 19 \times 2 + 19 \times 2.5 + 19 \times 2 + 19 \times 2 + 19 \times 2 + 19 \times 2 + 15}{16 \times 2.5 + 9 \times 2 + 9 \times 2.5 + 9 \times 2 + 9 \times 2 + 9 \times 2 + 9 \times 2 + 15} \times 0.27$$

$$k'_h = 0.47$$

- Seismic inertia angle, ψ ;

$$\psi = \arctan(k_h)$$

$$\psi = \arctan(0.27) = 15.10^\circ \quad \text{for dry part of soil}$$

$$\psi = \arctan(0.47) = 25.17^\circ \quad \text{for saturated part of soil}$$

Static Horizontal Forces

- Active earth pressure coefficient, K_A ;

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2(\theta) \cos(\theta + \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\cos(\delta + \theta) \cos(\beta - \theta)}} \right]^2}$$

$$K_A = \frac{\cos^2(38 - 0)}{\cos(0) \cos^2(0) \cos(0 + 12.67) \left[1 + \sqrt{\frac{\sin(38 + 12.67) \sin(38 - 0)}{\cos(12.67 + 0) \cos(0 - 0)}} \right]^2}$$

$$K_A = 0.221$$

- Active earth pressure, P_A ;

$$P_A = \frac{1}{2} K_A \gamma H^2$$

$$P_A = \frac{1}{2} 0.221 \times 9 \times 15^2 \Rightarrow P_A = 223.76 \text{ kN/m}$$

- Surcharge;

$$P_{i_{ss}} = q \times K_A$$

$$F_{i_{ss}} = P_{i_{ss}} \times h_i$$

Table A.3 The results of static earth pressures and static earth forces for L2 earthquake level

	K_A	γ (kN/m ³)	H (m)	P_a (kN/m ²)	F (kN/m)
Crown wall	0.221	16.00	2.00	7.072	7.07
Crown wall_1	0.221	16.00	2.50	8.84	3.98
Block 1	0.221	9.00	2.00	12.82	21.66
Block 2	0.221	9.00	2.50	17.79	38.26
Block 3	0.221	9.00	2.00	21.77	39.56
Block 4	0.221	9.00	2.00	25.75	47.52
Block 5	0.221	9.00	2.00	29.72	55.47
Block 6	0.221	9.00	2.00	33.70	63.43

Table A.4 The results of static surcharge pressures and static surcharge forces for L2 earthquake level

	K_A	q (kN/m ²)	H (m)	P_s (kN/m ²)	F (kN/m)
Crown wall	0.221	30.00	2.00	6.63	13.26
Crown wall_1	0.221	30.00	2.50	6.63	3.32
Block 1	0.221	30.00	2.00	6.63	13.26
Block 2	0.221	30.00	2.50	6.63	16.58
Block 3	0.221	30.00	2.00	6.63	13.26
Block 4	0.221	30.00	2.00	6.63	13.26
Block 5	0.221	30.00	2.00	6.63	13.26
Block 6	0.221	30.00	2.00	6.63	13.26

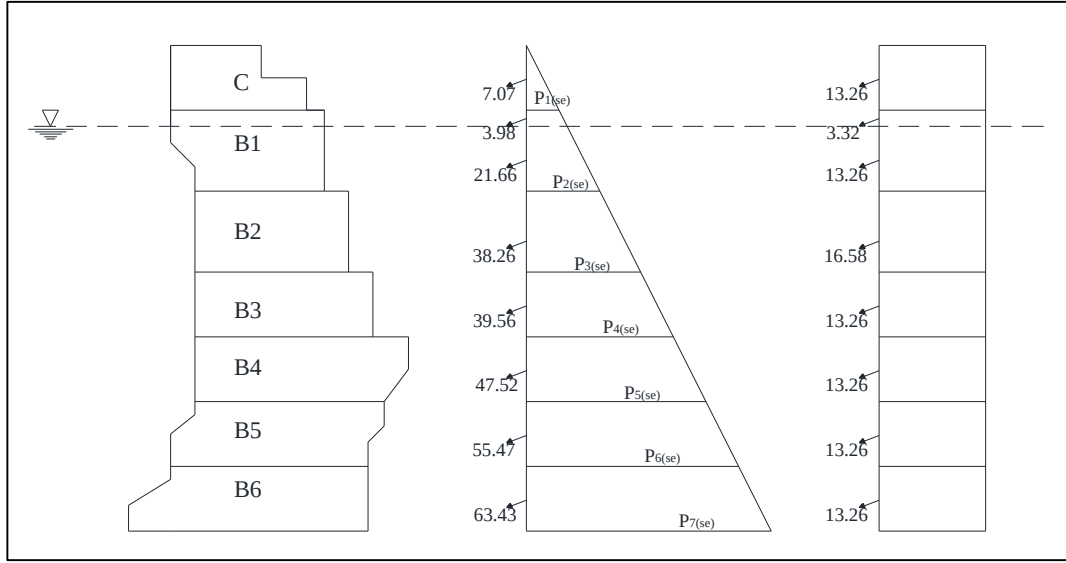


Figure A.2 The soil effect and surcharge effect on block type quay wall for static case for L2 earthquake motion

Dynamic Horizontal Forces

- Dynamic active earth pressure coefficient, K_{AE} ;

$$K_{AE, T} = \frac{\cos^2(\phi - \psi - \theta)}{\cos(\psi) \cos^2(\theta) \cos(\theta + \delta + \psi) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)}} \right]^2}$$

For dry part of soil;

$$K_{AE, T} = \frac{\cos^2(38 - 15.10 - 0)}{\cos(15.10) \cos^2(0) \cos(0 + 12.67 + 15.10) \left[1 + \sqrt{\frac{\sin(38 + 12.67) \sin(38 - 0 - 15.10)}{\cos(12.67 + 0 + 15.1) \cos(0 - 0)}} \right]^2}$$

$$K_{AE, T} = 0.396$$

$$K_{AE, d} = 0.396 - 0.221 = 0.175$$

For saturated part of soil;

$$K_{AE, T} = \frac{\cos^2(38 - 25.17 - 0)}{\cos(25.17) \cos^2(0) \cos(0 + 12.67 + 25.17) \left[1 + \sqrt{\frac{\sin(38 + 12.67) \sin(38 - 25.17)}{\cos(12.67 + 0 + 25.17) \cos(0 - 0)}} \right]^2}$$

$$K_{AE, T} = 0.619$$

$$K_{AE, d} = 0.619 - 0.221 = 0.398$$

- Total active thrust, P_{AE} ;

$$P_{AE} = \frac{1}{2} K_A \gamma H^2$$

$$P_{AE} = \left(\frac{1}{2} 0.396 \times 16 \times 2.5^2 \right) + \left(\frac{1}{2} 0.619 \times 9 \times 12.5^2 \right) \Rightarrow P_A = 455.03 \text{ kN/m}$$

- Dynamic component of total thrust, ΔP_{AE} ;

$$\Delta P_{AE} = 455.03 - 397.8 = 57.23 \text{ kN/m}$$

Table A.5 The results of dynamic forces for L2 earthquake level

	K_A	γ (kN/m ³)	H (m)	P_d (kN/m ²)	F (kN/m)
Crown wall	0.175	16.00	2.00	5.60	5.60
Crown wall_1	0.175	16.00	2.50	7.00	3.15
Block 1	0.398	9.00	2.00	14.16	21.16
Block 2	0.398	9.00	2.50	23.12	46.60
Block 3	0.398	9.00	2.00	30.28	53.40
Block 4	0.398	9.00	2.00	37.45	67.73
Block 5	0.398	9.00	2.00	44.61	82.06
Block 6	0.398	9.00	2.00	51.78	96.39

- Dynamic surcharge pressures, $P_{i(ds)}$, and dynamic earth forces, $F_{i(ds)}$;

Table A.6 The results of dynamic surcharge forces for L2 earthquake level

	K_A	q (kN/m ²)	H (m)	P_{ds} (kN/m ²)	F (kN/m)
Crown wall	0.175	15.00	2.00	2.63	5.25
Crown wall_1	0.175	15.00	2.50	2.63	1.31
Block 1	0.398	15.00	2.00	5.97	11.94
Block 2	0.398	15.00	2.50	5.97	14.93
Block 3	0.398	15.00	2.00	5.97	11.94
Block 4	0.398	15.00	2.00	5.97	11.94
Block 5	0.398	15.00	2.00	5.97	11.94
Block 6	0.398	15.00	2.00	5.97	11.94

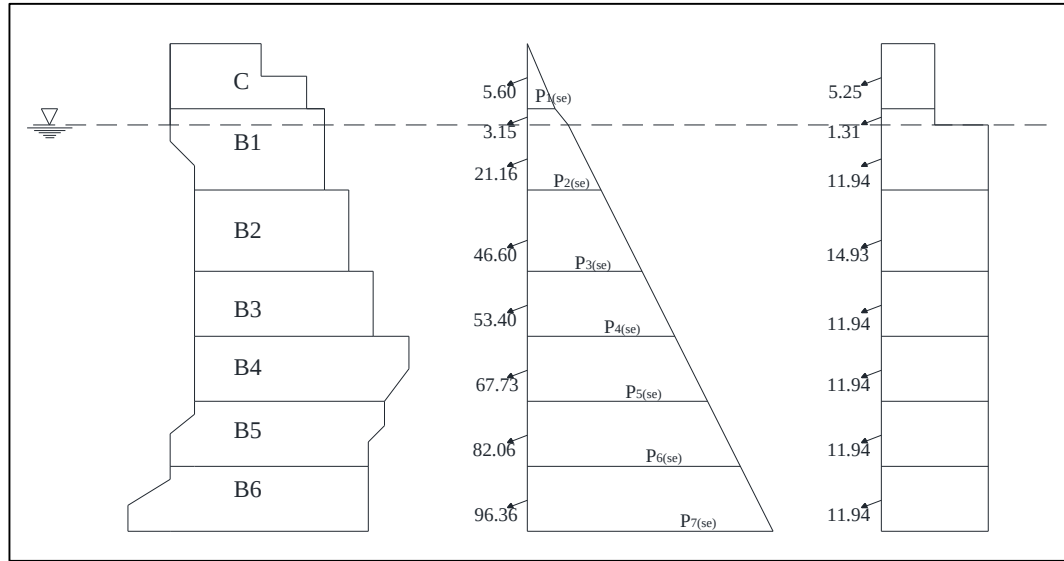


Figure A.3 The soil effect and surcharge effect on block type quay wall for dynamic case for L2 earthquake motion

- Dynamic water pressure, P_{dwi} , and dynamic water forces, F_{dwi} ;

$$P_{dw} = (7/8) k_h \gamma_w \sqrt{H_w h}$$

$$F_{dw} = \left(\frac{p_{dwi} + p_{dw(i-1)}}{2} \right) h_i$$

Table A.7 The results of dynamic water pressures and forces for L2 earthquake level

	H_w (m)	H (m)	k_h	γ_w (kN/m ³)	P_{dw} (kN/m ²)	F_{dw} (kN/m)
Block 1	12.5	2	0.27	10	11.79	11.79
Block 2	12.5	4.5	0.27	10	17.68	36.83
Block 3	12.5	6.5	0.27	10	21.25	38.93
Block 4	12.5	8.5	0.27	10	24.30	45.55
Block 5	12.5	10.5	0.27	10	27.01	51.31
Block 6	12.5	12.5	0.27	10	29.47	56.47

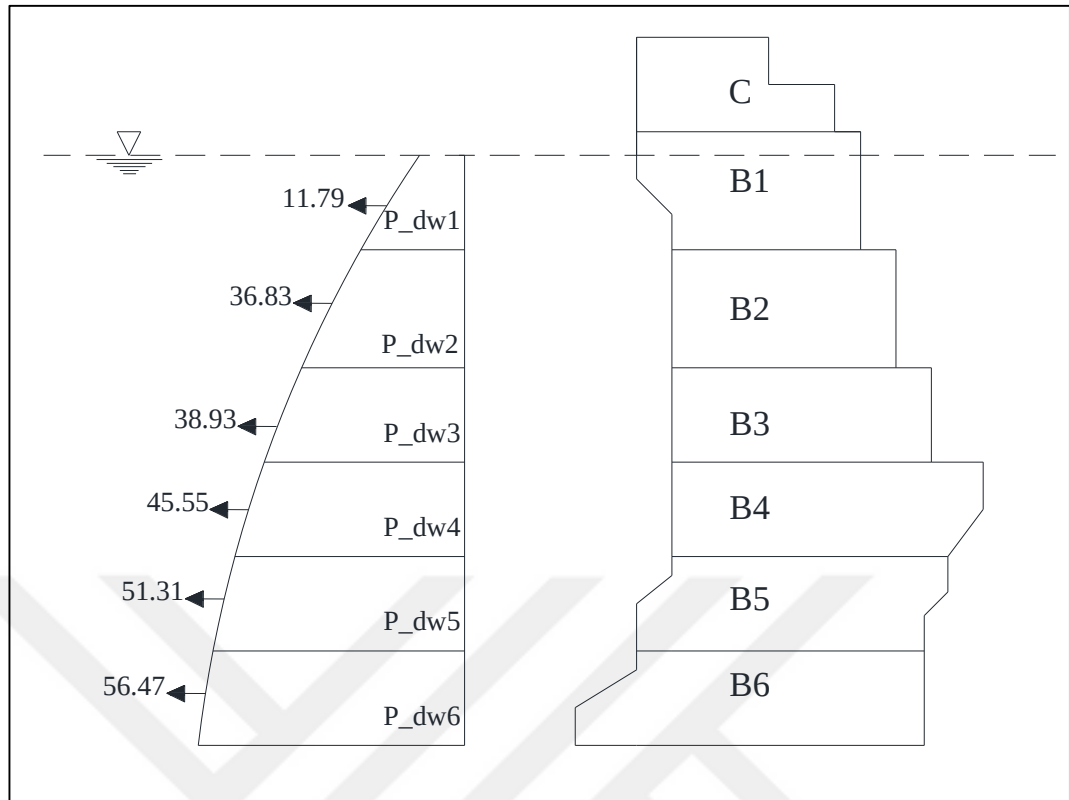


Figure A.4 The hydrodynamic forces on the block type quay wall for L2 earthquake motion

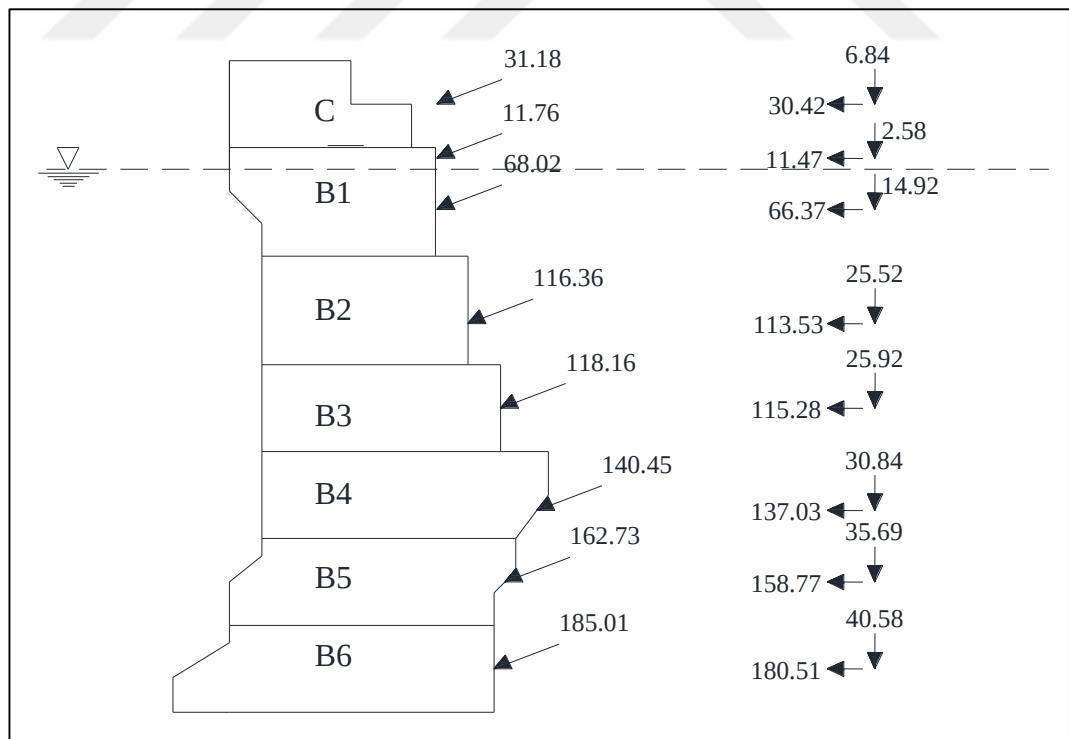
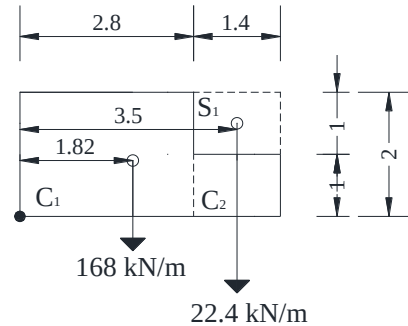


Figure A.5 Total forces on the quay wall for L2 earthquake motion

Vertical Forces

- **Crown wall and soil**



$$W = (x) \times (y) \times (\gamma_c)$$

$$W_{c1} = 2.8 \times 2 \times 24 = 134.4 \text{ kN/m}$$

$$W_{c2} = 1.4 \times 1 \times 24 = 33.6 \text{ kN/m}$$

$$W_{ct} = 128.8 + 32.2 = 168 \text{ kN/m}$$

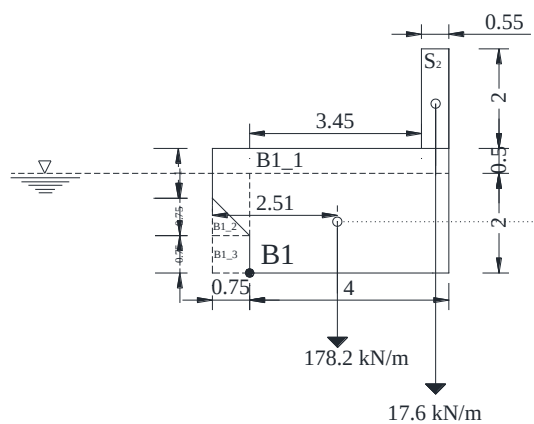
$$W_{s1} = 1.4 \times 1 \times 16 = 22.4 \text{ kN/m}$$

The centre of gravity;

$$e_{xc} = \frac{134.4 \times 1.4 + 33.6 \times (2.8 + 1.4/2)}{134.4 + 33.6} = 1.82 \text{ m}$$

$$e_{xs1} = 1.4/2 + 2.8 = 3.50 \text{ m}$$

- **Block 1 and soil**



$$W_{B1_1} = 4.75 \times 0.5 \times 24 = 57 \text{ kN/m}$$

$$W_{B1_2} = (0.75 \times 0.75)/2 \times 14 = 3.94 \text{ kN/m}$$

$$W_{B1_3} = 0.75 \times 0.75 \times 14 = 7.88 \text{ kN/m}$$

$$W_{B1_4} = 2 \times 4.75 \times 14 = 133 \text{ kN/m}$$

$$W_{B1T} = 133 + 57 - 7.88 - 3.94 = 178.2 \text{ kN/m}$$

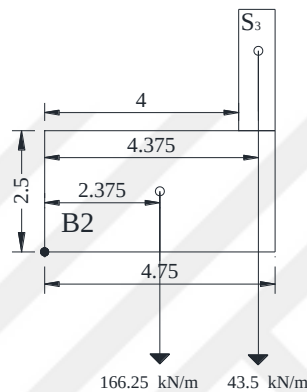
$$W_{s2} = 0.55 \times 2 \times 16 = 17.6 \text{ kN/m}$$

The center of gravity;

$$e_{xb1} = \frac{57 \times 2.375 + 133 \times 2.375 - 3.94 \times 0.50 - 7.88 \times 0.375}{178.2} = 2.51 \text{ m}$$

$$e_{xs2} = 0.55/2 + 3.45 = 3.725 \text{ m}$$

- **Block 2 and soil;**



$$W_{B2} = 4.75 \times 2.5 \times 14 = 166.25 \text{ kN/m}$$

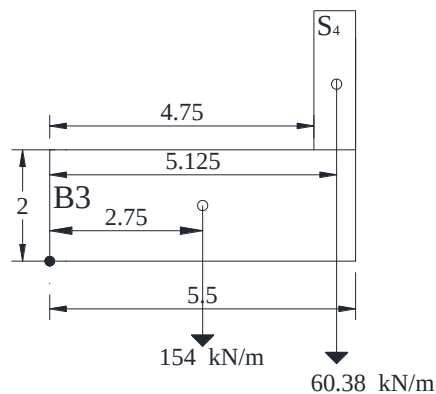
$$W_{S3} = 0.75 \times 2.5 \times 16 + 2 \times 9 \times 0.75 = 43.5 \text{ kN/m}$$

The centre of gravity;

$$e_{xB2} = 4.75/2 = 2.375 \text{ m}$$

$$e_{xS3} = 0.75/2 + 4 = 4.375 \text{ m}$$

- **Block 3 and soil;**



$$W_{B3} = 5.50 \times 2 \times 14 = 154 \text{ kN/m}$$

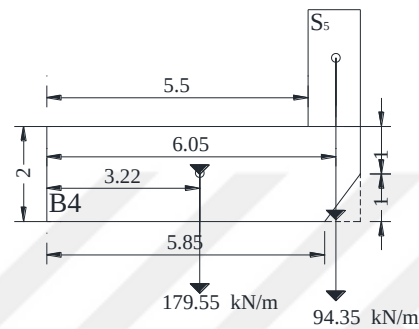
$$W_{S4} = 0.75 \times 2.5 \times 16 + 0.75 \times 4.5 \times 9 = 60.38 \text{ kN/m}$$

The centre of gravity;

$$e_{xB3} = 5.5/2 = 2.75 \text{ m}$$

$$e_{xS4} = 0.75/2 + 4.75 = 5.125 \text{ m}$$

- Block 4 and soil**



$$W_{B4_1} = 6.6 \times 2 \times 14 = 184.8 \text{ kN/m}$$

$$W_{B4_2} = (0.75 \times 1)/2 \times 14 = 5.25 \text{ kN/m}$$

$$W_{B4T} = 184.8 - 5.25 = 179.55 \text{ kN/m}$$

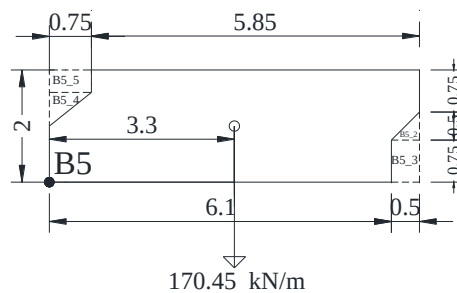
$$W_{S5} = 1.1 \times 9 \times 6.5 + 0.75 \times 2.5 \times 16 = 94.35 \text{ kN/m}$$

The centre of gravity;

$$e_{xB4} = \frac{184.8 \times 3.3 - 5.25 \times (5.85 + 0.75/3)}{179.55} = 3.22 \text{ m}$$

$$e_{xS5} = 1.1/2 + 5 = 6.05 \text{ m}$$

- Block 5;**



$$W_{B5_1} = 6.6 \times 2 \times 14 = 184.8 \text{ kN/m}$$

$$W_{B5_2} = ((0.5 \times 0.5)/2) \times 14 = 1.75 \text{ kN/m}$$

$$W_{B5_3} = 0.75 \times 0.5 \times 14 = 5.25 \text{ kN/m}$$

$$W_{B5_4} = ((0.6 \times 0.75)/2) \times 14 = 3.15 \text{ kN/m}$$

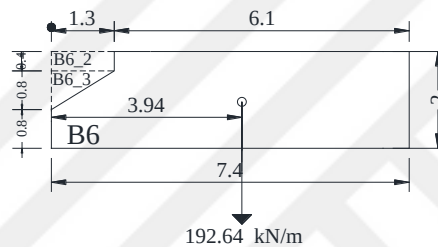
$$W_{B5_5} = 0.4 \times 0.75 \times 14 = 4.2 \text{ kN/m}$$

$$W_{B5_T} = 184.8 - 1.75 - 5.25 - 3.15 - 4.2 = 170.45 \text{ kN/m}$$

The centre of gravity;

$$e_{xB5} = \frac{184.8 \times 3.3 - 1.75 \times (6.1 + 0.5/3) - 5.25 \times (6.1 + 0.5/2) - 3.15 \times 0.25 - 4.2 \times 0.375}{170.45} = 3.30 \text{ m}$$

- Block 6;**



$$W_{B6_1} = 7.4 \times 2 \times 14 = 207.2 \text{ kN/m}$$

$$W_{B6_2} = 1.3 \times 0.4 \times 14 = 7.28 \text{ kN/m}$$

$$W_{b63} = (1.3 \times 0.8)/2 \times 14 = 7.28 \text{ kN/m}$$

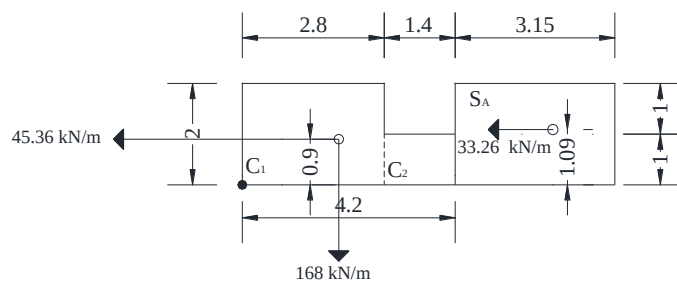
$$W_{b6T} = 207.2 - 7.28 - 7.28 = 192.64 \text{ kN/m}$$

The centre of gravity;

$$e_{xb6} = \frac{207.2 \times 3.7 - 7.28 \times 0.65 - 7.28 \times 0.65}{192.64} = 3.94 \text{ m}$$

Inertia Forces

- Crown wall**



$$W_{C_T} = 168 \text{ kN/m} ; k_h = 0.27$$

$$F_{hc} = 168 \times 0.27 = 45.36 \text{ kN/m}$$

Soil;

$$F_{S_1} = 1.4 \times 1 \times 16 \times 0.27 = 6.05 \text{ kN/m}$$

$$F_{S_A} = 3.15 \times 2 \times 16 \times 0.27 = 27.22 \text{ kN/m}$$

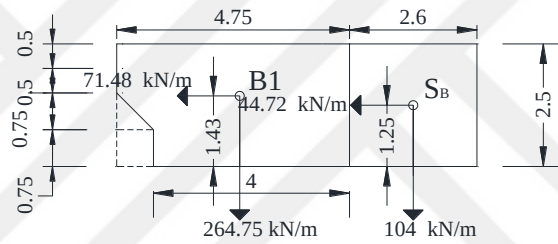
Total block forces: 168 kN/m

$$e_{yc} = \frac{134.4 \times 1 + 33.6 \times 0.5}{168} = 0.90 \text{ m}$$

Total soil forces: 33.26 kN/m

$$e_{ys1} = \frac{6.05 \times (1 + 1/2) + 27.22 \times 1}{33.26} = 1.1 \text{ m}$$

• **Block 1**



$$W_{B1_1} = 4.75 \times 0.5 \times 24 = 57 \text{ kN/m}$$

$$W_{B1_2} = (0.75 \times 0.75)/2 \times 24 = 6.75 \text{ kN/m}$$

$$W_{B1_3} = 0.75 \times 0.75 \times 24 = 13.5 \text{ kN/m}$$

$$W_{B1_4} = 2 \times 4.75 \times 24 = 228 \text{ kN/m}$$

$$W_{B1T} = 228 + 57 - 6.75 - 13.5 = 264.75 \text{ kN/m}$$

$$F_{h_B1} = 264.75 \times 0.27 = 71.48 \text{ kN/m}$$

$$e_{yB1} = \frac{228 \times 1 + 57 \times 2.25 - 6.75 \times 1 - 13.5 \times 0.375}{264.75} = 1.30 \text{ m}$$

Soil;

$$W_{SB_1} = 2.6 \times 0.5 \times 16 = 20.8 \text{ kN/m}$$

$$W_{SB_2} = 2.60 \times 2 \times 16 = 83.2 \text{ kN/m}$$

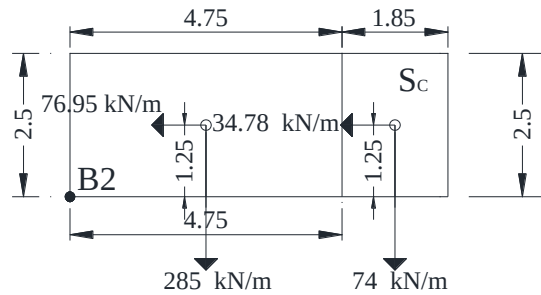
$$F_{hSB_1} = 20.8 \times 0.27 = 5.62 \text{ kN/m}$$

$$F_{hSB_2} = 83.2 \times 0.47 = 39.10 \text{ kN/m}$$

$$F_{hSB} = 39.10 + 5.62 = 44.72 \text{ kN/m}$$

$$e_{ySB} = 1.25 \text{ m}$$

- **Block 2**



$$W_{B2} = 4.75 \times 2.5 \times 24 = 285 \text{ kN/m}$$

$$F_{h_B2} = 285 \times 0.27 = 76.95 \text{ kN/m}$$

$$e_{yB2} = 1.25 \text{ m}$$

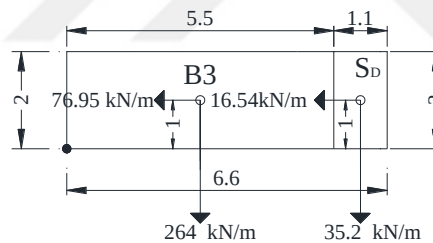
Soil;

$$W_{SC} = 1.85 \times 2.5 \times 16 = 74 \text{ kN/m}$$

$$F_{hSC} = 74 \times 0.47 = 34.78 \text{ kN/m}$$

$$e_{ySC} = 1.25 \text{ m}$$

- **Block 3**



$$W_{B3} = 5.5 \times 2 \times 24 = 264 \text{ kN/m}$$

$$F_{h_B3} = 264 \times 0.27 = 71.28 \text{ kN/m}$$

$$e_{yB3} = 1 \text{ m}$$

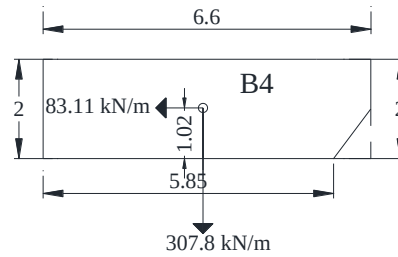
Soil;

$$W_{SD} = 1.1 \times 2 \times 16 = 35.2 \text{ kN/m}$$

$$F_{hSD} = 35.2 \times 0.47 = 16.54 \text{ kN/m}$$

$$e_{ySD} = 1 \text{ m}$$

- **Block 4**



$$W_{B4_1} = 6.6 \times 2 \times 24 = 316.8 \text{ kN/m}$$

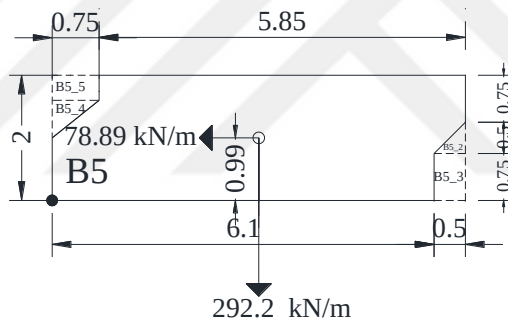
$$W_{B4_2} = (0.75 \times 1)/2 \times 24 = 9 \text{ kN/m}$$

$$W_{B4T} = 316.8 - 9 = 307.8 \text{ kN/m}$$

$$F_{h_B4} = 307.8 \times 0.27 = 83.11 \text{ kN/m}$$

$$e_{yB4} = \frac{316.8 \times 1 - 9 \times (1/3)}{307.8} = 1.02 \text{ m}$$

- **Block 5**



$$W_{B5_1} = 6.6 \times 2 \times 24 = 316.8 \text{ kN/m}$$

$$W_{B5_2} = ((0.5 \times 0.5)/2) \times 24 = 3 \text{ kN/m}$$

$$W_{B5_3} = 0.5 \times 0.75 \times 24 = 9 \text{ kN/m}$$

$$W_{B5_4} = ((0.75 \times 0.6)/2) \times 24 = 5.4 \text{ kN/m}$$

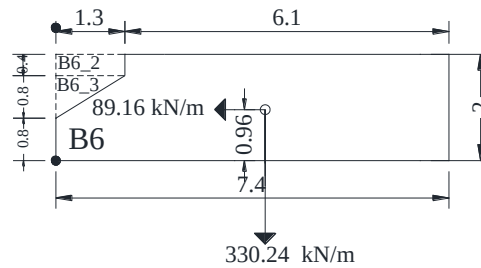
$$W_{B5_5} = 0.75 \times 0.4 \times 24 = 7.2 \text{ kN/m}$$

$$W_{B5_T} = 316.8 - 3 - 9 - 5.4 - 7.2 = 292.2 \text{ kN/m}$$

$$F_{h_B5} = 292.2 \times 0.27 = 78.89 \text{ kN/m}$$

$$e_{yB5} = \frac{316.8 \times 3.3 - 3 \times (0.75 + 0.5/3) - 9 \times (0.75/2) - 5.4 \times (1 + 0.6/3) - 7.2 \times 1.8}{292.2} = 0.99 \text{ m}$$

- **Block 6**



$$W_{B6_1} = 7.4 \times 2 \times 24 = 355.2 \text{ kN/m}$$

$$W_{B6_2} = 1.3 \times 0.4 \times 24 = 12.48 \text{ kN/m}$$

$$W_{B6_3} = ((1.3 \times 0.8)/2) \times 24 = 12.48 \text{ kN/m}$$

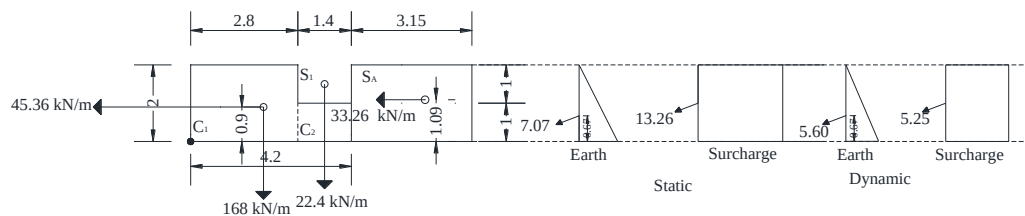
$$W_{B6_T} = 355.2 - 12.48 - 12.48 = 330.24 \text{ kN/m}$$

$$F_{h_B6} = 330.24 \times 0.27 = 89.16 \text{ kN/m}$$

$$e_{yB6} = \frac{355.2 \times 1 - 12.48 \times 1.8 - 12.48 \times (0.8 + 0.8/3)}{330.24} = 0.96 \text{ m}$$

Stability Analysis

- **Crown wall**



$$\Sigma \text{Horizontal} = 45.36 + 24.75 + 33.26 + 7.07 \times \cos(12.67) + 13.26 \times 0.5 \times \cos(12.67) + 5.60 \times \cos(12.67) + 5.25 \times \cos(12.67) = 127.33 \text{ kN/m}$$

$$\Sigma \text{Vertical} = 168 + 22.4 + 6.84 = 197.24 \text{ kN/m}$$

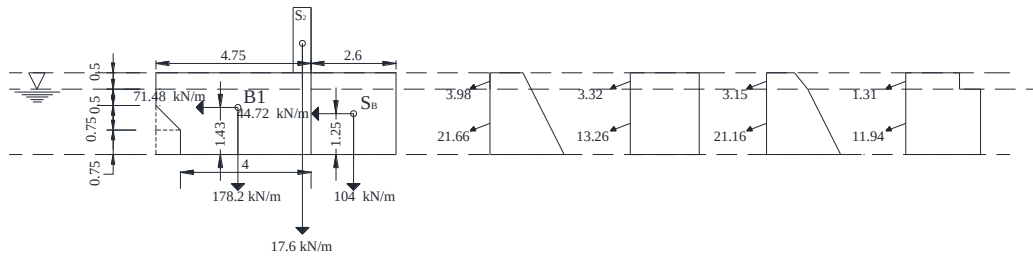
$$FS_S = \frac{197.24 \times 0.5}{127.33} = 0.77 < 1.0 \text{ (x)}$$

$$\text{Overturning moments} = 45.36 \times 0.9 + 33.26 \times 1.09 + 6.9 \times 2/3 + 12.94 \times 1 + 1.5 \times (5.60 \times 2/3) + 1.5 \times (5.12 \times 1) + 24.75 \times 2.375 = 166.58 \text{ kNm/m}$$

$$\text{Resisting moments} = 168 \times 1.82 + 22.4 \times 3.5 + 6.84 \times 4.2 = 412.88 \text{ kNm/m}$$

$$FS_O = \frac{412.88}{166.58} = 2.48 > 1.1 \text{ (OK)}$$

• **Block 1**



$$\begin{aligned}\Sigma \text{Horizontal} &= 127.33 + 71.48 + 44.72 + 3.98 \times \cos(12.67) + 21.66 \times \cos(12.67/2) \\ &\quad + 0.5 \times 3.32 \times \cos(12.67) + 0.5 \times 13.26 \times \cos(12.67/2) + 3. \times \cos(12.67) \\ &\quad + 21.26 \times \cos(12.67/2) + 1.31 \times \cos(12.67) + 11.94 \times \cos(12.67/2) + 11.79 \\ &= 326.19 \text{ kN/m}\end{aligned}$$

$$\Sigma \text{Vertical} = 197.24 + 178.2 + 17.6 + 10.08 = 403.11 \text{ kN/m}$$

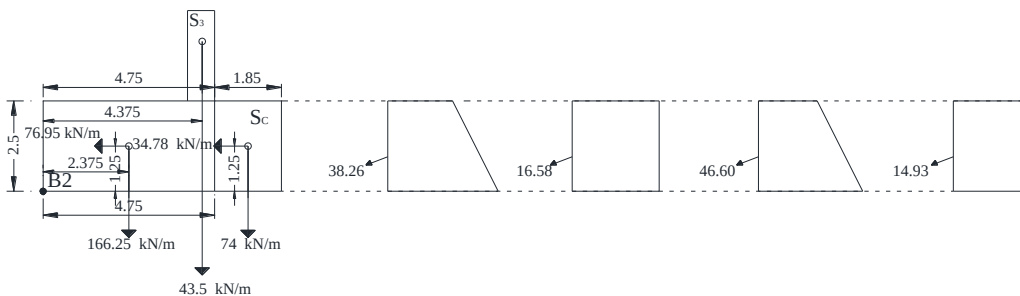
$$FS_S = \frac{403.11 \times 0.5}{326.19} = 0.62 < 1.0 \text{ (x)}$$

$$\begin{aligned}\text{Overturning moments} &= 166.58 + 127.33 \times 2.5 + 71.48 \times 1.3 + 44.72 \times 1.25 + 3.98 \times 2.24 + \\ &\quad 21.66 \times 0.93 + 3.32 \times 2.25 + 13.26 \times 1 + 3.63 \times 2.24 + 1.5 \times 21.26 \times 0.9 \\ &\quad + 1.51 \times 1.31 + 1.5 \times 11.94 \times 1 + 11.79 \times 2/3 = 746.76 \text{ kNm/m}\end{aligned}$$

$$\begin{aligned}\text{Resisting moments} &= 197.24 \times 0.65 + 33.6 \times 2.75 + 22.4 \times 2.75 + 6.84 \times 3.45 + 178.2 \times 1.76 \\ &\quad + 17.6 \times 3.725 + 10.08 \times 4.2 = 684.45 \text{ kNm/m}\end{aligned}$$

$$FS_O = \frac{684.45}{746.76} = 0.92 < 1.1 \text{ (x)}$$

• **Block 2**



$$\begin{aligned}\Sigma \text{Horizontal} &= 326.19 + 76.95 + 34.78 + 38.26 \times \cos(12.67/2) + 0.5 \times 16.58 \times \cos(12.67/2) \\ &\quad + 46.6 \times \cos(12.67/2) + 14.93 \times \cos(12.67/2) + 36.83 = 582.17 \text{ kN/m}\end{aligned}$$

$$\Sigma \text{Vertical} = 403.11 + 166.25 + 43.5 + 12.84 = 625.69 \text{ kN/m}$$

$$FS_S = \frac{625.69 \times 0.5}{582.17} = 0.54 < 1.0 \text{ (x)}$$

$$\begin{aligned}\text{Overturning moments} &= 746.76 + 326.19 \times 2.5 + 76.95 \times 1.25 + 34.78 \times 1.25 + 38.03 \times 1.18 \\ &\quad + 16.47 \times 1.25 + 1.5 \times 46.32 \times 1.15 + 1.5 \times 14.83 \times 1.25 + 36.83 \times 1.17 \\ &= 1918.16 \text{ kNm/m}\end{aligned}$$

$$\text{Resisting moments} = 684.45 + 166.25 \times 2.38 + 43.5 \times 4.38 + 12.84 \times 4.75 = 1330.57 \text{ kNm/m}$$

$$FS_O = \frac{1330.57}{1918.16} = 0.69 < 1.1 \text{ (x)}$$

• Block 3



$$\begin{aligned}\Sigma \text{Horizontal} &= 582.17 + 71.28 + 16.54 + 39.56 \times \cos(12.67/2) + 0.5 \times 13.26 \times \cos(12.67/2) \\ &\quad + 53.40 \times \cos(12.67/2) + 11.94 \times \cos(12.67/2) + 38.93 = 819.77 \text{ kN/m}\end{aligned}$$

$$\Sigma \text{Vertical} = 625.69 + 154 + 60.38 + 13.03 = 853.10 \text{ kN/m}$$

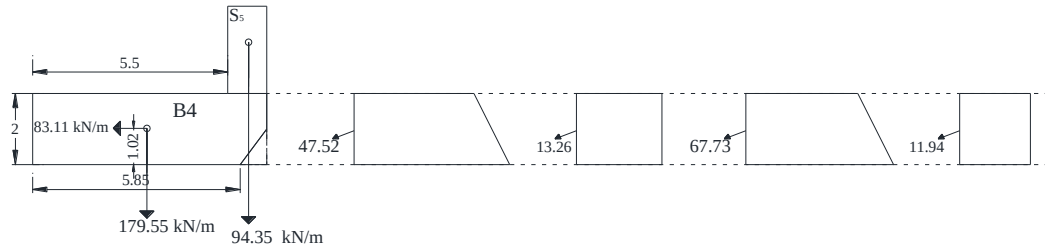
$$FS_S = \frac{853.1 \times 0.5}{819.77} = 0.52 < 1.0 \text{ (x)}$$

$$\begin{aligned}\text{Overturning moments} &= 1918.16 + 582.17 \times 2 + 71.28 \times 1 + 16.54 \times 1 + 39.32 \times 0.97 \\ &\quad + 13.18 \times 1 + 1.5 \times 53.08 \times 0.96 + 1.5 \times 11.87 \times 1 + 38.93 \times 1 \\ &= 3353.63 \text{ kNm/m}\end{aligned}$$

$$\text{Resisting moments} = 1330.57 + 154 \times 2.75 + 60.38 \times 5.125 + 13.03 \times 5.5 = 2135.19 \text{ kNm/m}$$

$$FS_O = \frac{2135.19}{3353.63} = 0.64 < 1.1 \text{ (x)}$$

• **Block 4**



$$\Sigma \text{Horizontal} = 819.77 + 83.11 + 47.52 \times \cos(12.67/2) + 0.5 \times 13.26 \times \cos(12.67/2) + 67.73 \times \cos(12.67/2) + 11.94 \times \cos(12.67/2) + 45.55 = 1081.43 \text{ kN/m}$$

$$\Sigma \text{Vertical} = 853.10 + 179.55 + 94.35 + 15.49 = 1142.50 \text{ kN/m}$$

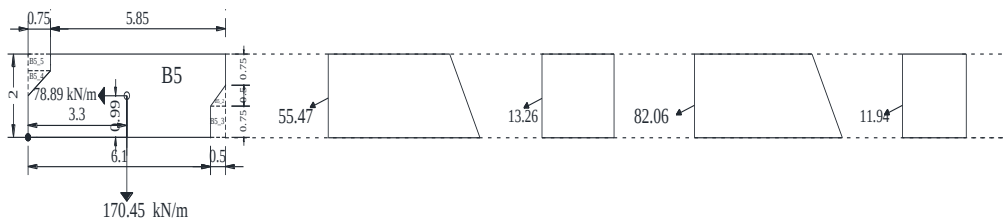
$$FS_s = \frac{1142.5 \times 0.5}{1081.43} = 0.53 < 1.0 \text{ (x)}$$

$$\text{Overturning moments} = 3353.63 + 819.77 \times 2 + 83.11 \times 1.02 + 47.23 \times 0.97 + 13.18 \times 1 + 1.5 \times 67.32 \times 0.97 + 1.5 \times 11.87 \times 1 + 45.55 \times 0.98 = 5297.3 \text{ kNm/m}$$

$$\text{Resisting moments} = 2135.19 + 179.55 \times 3.22 + 94.35 \times 6.05 + 15.49 \times 6.6 = 3386.41 \text{ kNm/m}$$

$$FS_o = \frac{3386.41}{5297.28} = 0.64 < 1.1 \text{ (x)}$$

• **Block 5**



$$\Sigma \text{Horizontal} = 1081.43 + 78.89 + 55.47 \times \cos(12.67/2) + 0.5 \times 13.26 \times \cos(12.67/2) + 82.06 \times \cos(12.67/2) + 11.94 \times \cos(12.67/2) + 51.31 = 1366.77 \text{ kN/m}$$

$$\Sigma \text{Vertical} = 1142.50 + 170.45 + 17.95 = 1330.9 \text{ kN/m}$$

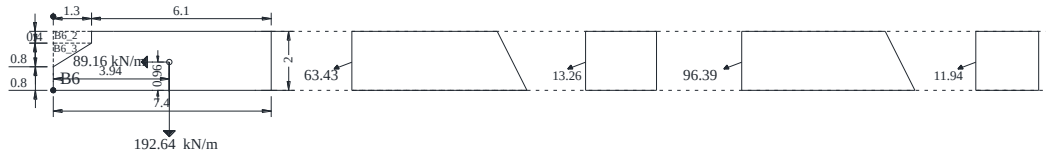
$$FS_s = \frac{1330.9 \times 0.5}{1366.77} = 0.49 < 1.0 \text{ (x)}$$

$$\text{Overturning moments} = 5297.28 + 1081.43 \times 2 + 78.89 \times 0.99 + 55.13 \times 0.98 + 13.18 \times 1 + 1.5 \times 81.56 \times 0.97 + 1.5 \times 11.87 \times 1 + 51.31 \times 0.99 = 7792.9 \text{ kNm/m}$$

$$\text{Resisting moments} = 3386.41 + 1142.50 \times 0.75 + 170.45 \times 3.3 + 17.95 \times 6.33 = 4919.39 \text{ kNm/m}$$

$$FS_O = \frac{4919.39}{7792.94} = 0.63 < 1.1 \text{ (x)}$$

- **Block 6**



$$\Sigma \text{Horizontal} = 1366.77 + 89.16 + 63.43 \times \cos(12.67/2) + 0.5 \times 12.36 \times \cos(12.67/2) + 96.39 \times \cos(12.67/2) + 11.94 \times \cos(12.67/2) + 56.47 = 1689.71 \text{ kN/m}$$

$$\Sigma \text{Vertical} = 1330.9 + 192.64 + 20.41 = 1543.95 \text{ kN/m}$$

$$FS_s = \frac{1543.95 \times 0.6}{1689.71} = 0.55 < 1.0 \text{ (x)}$$

$$\begin{aligned} \text{Overturning moments} &= 7792.94 + 1366.77 \times 2 + 89.16 \times 0.96 + 63.04 \times 0.98 + 13.18 \times 1 \\ &\quad + 1.5 \times 95.80 \times 0.97 + 1.5 \times 14.73 \times 1 + 56.47 \times 0.98 \\ &= 10899.33 \text{ kNm/m} \end{aligned}$$

$$\begin{aligned} \text{Resisting moments} &= 4919.39 + 1330.9 \times 1.3 + 192.64 \times 3.94 + 20.41 \times 7.4 \\ &= 7559.59 \text{ kNm/m} \end{aligned}$$

$$FS_O = \frac{7559.59}{10899.33} = 0.69 < 1.1 \text{ (x)}$$

Table A.8 Factor of safety values against sliding and overturning for L2 earthquake level

	F_{Ss}	F_{So}
Crown Wall	0.77	2.48
Block 1	0.62	0.92
Block 2	0.54	0.69
Block 3	0.52	0.64
Block 4	0.53	0.64
Block 5	0.49	0.63
Block 6	0.55	0.69

Table A.9 Factor of Safety against Sliding and Overturning according to Performance Level (TDSC-CRA, 2007)

Factor of Safety	Minimum Damage	Controlled Damage
Sliding	>1.2	>1.0
Overturning	>1.3	>1.1

Table A.10 The horizontal and vertical forces for sliding factor for L2 earthquake level

	Static Forces						Dynamic Forces						Hydrodynamic Force	Horizontal Forces
	F_s (kN/m)	$F_{s,x}$ (kN/m)	$F_{s,y}$ (kN/m)	F_{ss} (kN/m)	$F_{ss,x}$ (kN/m)	$F_{ss,y}$ (kN/m)	F_{de} (kN/m)	$F_{de,x}$ (kN/m)	$F_{de,y}$ (kN/m)	F_{ds} (kN/m)	$F_{ds,x}$ (kN/m)	$F_{ds,y}$ (kN/m)	F_{dw} (kN/m)	Total Thrust (TT)
CW	7.07	6.90	1.55	13.26	12.94	2.91	5.60	5.46	1.23	5.25	5.12	1.15	0	127.33
CW1	3.98	3.88	0.87	3.32	3.23	0.73	3.15	3.07	0.69	1.31	1.28	0.29	0.00	326.19
B1	21.66	21.53	2.39	13.26	13.18	1.46	21.16	21.03	2.33	11.94	11.87	1.32	11.79	
B2	38.26	38.03	4.22	16.58	16.47	1.83	46.60	46.32	5.14	14.93	14.83	1.65	36.83	582.17
B3	39.56	39.32	4.36	13.26	13.18	1.46	53.40	53.08	5.89	11.94	11.87	1.32	38.93	819.77
B4	47.52	47.23	5.24	13.26	13.18	1.46	67.73	67.32	7.47	11.94	11.87	1.32	45.55	1081.43
B5	55.47	55.13	6.12	13.26	13.18	1.46	82.06	81.56	9.05	11.94	11.87	1.32	51.31	1366.77
B6	63.43	63.04	7.00	13.26	13.18	1.46	96.39	95.80	10.63	11.94	11.87	1.32	56.47	1689.71

	Inertia Forces	Blocks Weight	Soil Effect		Vertical Forces
	F_h (kN/m)	W (kN/m)	F_h (kN/m)	W (kN/m)	Total Thrust (TT)
CW	45.36	168.00	33.26	22.40	197.24
CW1	71.48	178.19	44.72	17.6	403.11
B1					
B2	76.95	166.25	34.78	43.5	625.69
B3	71.28	154.00	16.54	60.375	853.10
B4	83.11	179.55	0.00	94.35	1142.50
B5	78.89	170.45	0.00	0	1330.90
B6	89.16	192.64	0.00	0	1543.95

Table A.11 The horizontal and vertical forces and epicenter point for overturning factor for L2 earthquake level

	Static Forces								Dynamic Forces							
	F_s (kN/m)	$F_{s,x}$ (kN/m)	$F_{s,y}$ (kN/m)	e_y	F_{ss} (kN/m)	$F_{ss,x}$ (kN/m)	$F_{ss,y}$ (kN/m)	e_y	F_{de} (kN/m)	$F_{de,x}$ (kN/m)	$F_{de,y}$ (kN/m)	e_y	F_{ds} (kN/m)	$F_{ds,x}$ (kN/m)	$F_{ds,y}$ (kN/m)	e_y
CW	7.07	6.90	1.55	0.67	13.26	12.94	2.91	1.00	5.60	5.46	1.23	0.67	5.25	5.12	1.15	1.00
CW1	3.98	3.88	0.87	2.24	3.32	3.23	0.73	2.25	3.15	3.07	0.69	2.24	1.31	1.28	0.29	2.25
B1	21.66	21.53	2.39	0.93	13.26	13.18	1.46	1.00	21.16	21.03	2.33	0.90	11.94	11.87	1.32	1.00
B2	38.26	38.03	4.22	1.18	16.58	16.47	1.83	1.25	46.60	46.32	5.14	1.15	14.93	14.83	1.65	1.25
B3	39.56	39.32	4.36	0.97	13.26	13.18	1.46	1.00	53.40	53.08	5.89	0.96	11.94	11.87	1.32	1.00
B4	47.52	47.23	5.24	0.97	13.26	13.18	1.46	1.00	67.73	67.32	7.47	0.97	11.94	11.87	1.32	1.00
B5	55.47	55.13	6.12	0.98	13.26	13.18	1.46	1.00	82.06	81.56	9.05	0.97	11.94	11.87	1.32	1.00
B6	63.43	63.04	7.00	0.98	13.26	13.18	1.46	1.00	96.39	95.80	10.63	0.97	11.94	11.87	1.32	1.00

	Hydrodynamic Force		Inertia Forces		Soil Effect	
	F_{dw} (kN/m)	e_y	F_h (kN/m)	e_y	F_{sh} (kN/m)	e_y
CW	0.00	0.00	45.36	0.90	33.26	1.09
CW1	11.79	0.67	71.48	1.30	44.72	1.25
B1						
B2	36.83	1.17	76.95	1.25	34.78	1.25
B3	38.93	0.97	71.28	1.00	16.54	1.00
B4	45.55	0.98	83.11	1.02	0.00	0.00
B5	51.31	0.99	78.89	0.99	0.00	0.00
B6	56.47	0.98	89.16	0.96	0.00	0.00

Table A.12 The factor of safety against sliding for L2 earthquake level

	Horizontal Forces	Vertical Forces	F_{ss}
Crown Wall	127.33	197.24	0.77
Block 1	326.19	403.11	0.62
Block 2	582.17	625.69	0.54
Block 3	819.77	853.10	0.52
Block 4	1081.43	1142.50	0.53
Block 5	1366.77	1330.90	0.49
Block 6	1689.71	1543.95	0.55

Table A.13 The factor of safety against overturning for L2 earthquake level

	Overturning Moment	Resistant Moment	F_{so}
Crown Wall	166.578	412.88	2.48
Block 1	746.756	684.45	0.92
Block 2	1918.162	1330.57	0.69
Block 3	3353.633	2135.19	0.64
Block 4	5297.277	3386.41	0.64
Block 5	7792.939	4919.39	0.63
Block 6	10899.332	7559.59	0.69

APPENDIX–II: Conventional Analysis for Peak Ground Acceleration Obtained from Kocaeli Station During 1999 Marmara Earthquake

Design Parameter

Table A.14 Design Parameter for Analysis

ϕ	δ	β	θ	H	H_{sub}	q_s	q_{EA}	Bollard	H_c
(°)	(°)	(°)	(°)	(m)	(m)	(kN/m ²)	(kN/m ²)	(kN)	(m)
38	12.67	0	0	15.00	12.50	30	15	800	0.8

γ_{conc}	$\gamma_{dry_layer\#1}$	$\gamma_{sat_layer\#1}$	$\gamma_{dry_layer\#2}$	$\gamma_{sat_layer\#2}$	γ_{sub}	$\gamma_{conc,sub}$	γ_w
(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)	(kN/m ³)
24	16	18	16	19	9	14	10

*The friction angle between wall and soil was accepted as $\phi/3$ for the all calculations.

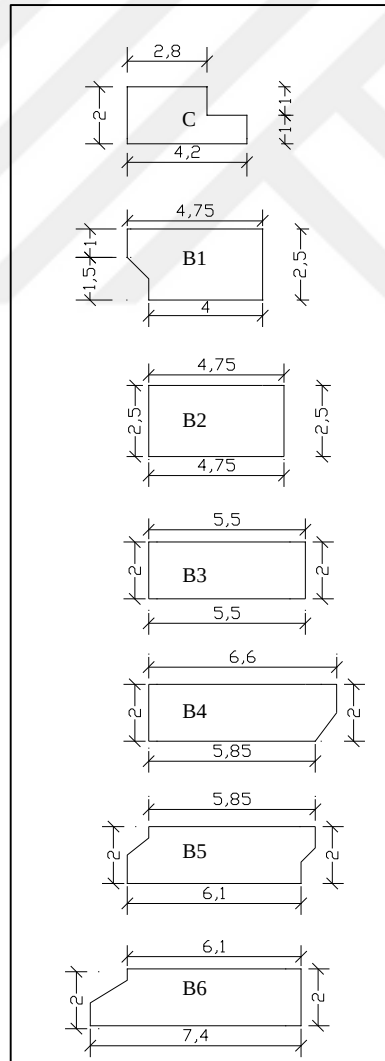


Figure A.6 Dimension of block type quay walls (unit: m)

Seismic Coefficient

In this calculation, seismic coefficient is used as 0.22g. This value is obtained from Kocaeli station which is recorded during 1999 Marmara Earthquake.

For L2 earthquake motion;

$$A_{20} = 0.22 \text{ (a}_{\max}/g)$$

- Horizontal seismic coefficient for L2 earthquake motion;

$$k_h = (1/3) \times 0.22^{(1/3)} = 0.201$$

- The modified seismic coefficient, k'_h ;

$$k'_h = \frac{\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^N(\gamma_{sj} h_j) + q_0}{\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^N(\gamma_{bj} h_j) + q_0} \times k_h$$

$$k'_h = \frac{16 \times 2.5 + 19 \times 2 + 19 \times 2.5 + 19 \times 2 + 19 \times 2 + 19 \times 2 + 19 \times 2 + 15}{16 \times 2.5 + 9 \times 2 + 9 \times 2.5 + 9 \times 2 + 9 \times 2 + 9 \times 2 + 9 \times 2 + 15} \times 0.201$$

$$k'_h = 0.351$$

- Seismic inertia angle, ψ ;

$$\psi = \arctan(k_h)$$

$$\psi = \arctan(0.201) = 11.38^\circ \quad \text{for dry part of soil}$$

$$\psi = \arctan(0.351) = 19.36^\circ \quad \text{for saturated part of soil}$$

Static Horizontal Forces

- Active earth pressure coefficient, K_A ;

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2(\theta) \cos(\theta + \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta)}{\cos(\delta + \theta) \cos(\beta - \theta)}} \right]^2}$$

$$K_A = \frac{\cos^2(38 - 0)}{\cos(0) \cos^2(0) \cos(0 + 12.67) \left[1 + \sqrt{\frac{\sin(38 + 12.67) \sin(38 - 0)}{\cos(12.67 + 0) \cos(0 - 0)}} \right]^2}$$

$$K_A = 0.221$$

- Active earth pressure, P_A ;

$$P_A = \frac{1}{2} K_A \gamma H^2$$

$$P_A = \frac{1}{2} 0.221 \times 16 \times 15^2 \Rightarrow P_A = 397.8 \text{ kN/m}$$

- Surcharge;

$$P_{i_ss} = q \times K_A$$

$$F_{i_ss} = P_{i_ss} \times h_i$$

Table A.15 The results of static earth pressures and static earth forces for Kocaeli Earthquake

	K_A	γ (kN/m ³)	H (m)	P_a (kN/m ²)	F (kN/m)
Crown wall	0.221	16.00	2.00	7.072	7.07
Crown wall_1	0.221	16.00	2.50	8.84	3.98
Block 1	0.221	9.00	2.00	12.82	21.66
Block 2	0.221	9.00	2.50	17.79	38.26
Block 3	0.221	9.00	2.00	21.77	39.56
Block 4	0.221	9.00	2.00	25.75	47.52
Block 5	0.221	9.00	2.00	29.72	55.47
Block 6	0.221	9.00	2.00	33.70	63.43

Table A.16 The results of static surcharge pressures and static surcharge forces for Kocaeli Earthquake

	K_A	q (kN/m ²)	H (m)	P_s (kN/m ²)	F (kN/m)
Crown wall	0.221	30.00	2.00	6.63	13.26
Crown wall_1	0.221	30.00	2.50	6.63	3.32
Block 1	0.221	30.00	2.00	6.63	13.26
Block 2	0.221	30.00	2.50	6.63	16.58
Block 3	0.221	30.00	2.00	6.63	13.26
Block 4	0.221	30.00	2.00	6.63	13.26
Block 5	0.221	30.00	2.00	6.63	13.26
Block 6	0.221	30.00	2.00	6.63	13.26

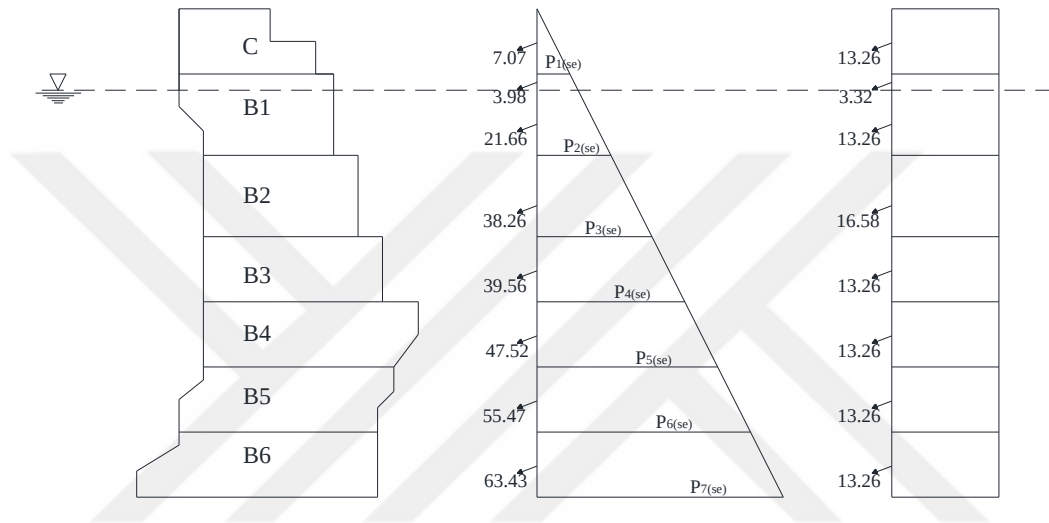


Figure A.7 The soil effect and surcharge effect on block type quay wall for static case for Kocaeli Earthquake

Dynamic Horizontal Forces

- Dynamic active earth pressure coefficient, K_{AE} ;

$$K_{AE, T} = \frac{\cos^2(\phi - \psi - \theta)}{\cos(\psi) \cos^2(\theta) \cos(\theta + \delta + \psi) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)}} \right]^2}$$

For dry part of soil;

$$K_{AE, T} = \frac{\cos^2(38 - 11.38 - 0)}{\cos(11.38) \cos^2(0) \cos(0 + 12.67 + 11.38) \left[1 + \sqrt{\frac{\sin(38 + 12.67) \sin(38 - 0 - 11.38)}{\cos(12.67 + 0 + 11.38) \cos(0 - 0)}} \right]^2}$$

$$K_{AE, T} = 0.342$$

$$K_{AE, d} = 0.342 - 0.221 = 0.121$$

For saturated part of soil;

$$K_{AE,T} = \frac{\cos^2(38-19.36-0)}{\cos(19.36) \cos^2(0) \cos(0+12.67+19.36) \left[1 + \sqrt{\frac{\sin(38+12.67)\sin(38-19.36)}{\cos(12.67+0+19.36)\cos(0-0)}} \right]^2}$$

$$K_{AE,T} = 0.473$$

$$K_{AE,d} = 0.473 - 0.221 = 0.252$$

- Total active thrust, P_{AE} ;

$$P_{AE} = \frac{1}{2} K_A \gamma H^2$$

$$P_{AE} = \left(\frac{1}{2} 0.342 \times 16 \times 2.5^2 \right) + \left(\frac{1}{2} 0.473 \times 9 \times 12.5^2 \right) \Rightarrow P_A = 499.61 \text{ kN/m}$$

- Dynamic component of total thrust, ΔP_{AE} ;

$$\Delta P_{AE} = 499.61 - 397.8 = 101.81 \text{ kN/m}$$

Table A.17 The results of dynamic forces for Kocaeli Earthquake

	K_A	γ (kN/m ³)	H (m)	P_d (kN/m ²)	F (kN/m)
Crown wall	0.121	16.00	2.00	3.87	6.56
Crown wall_1	0.121	16.00	2.50	4.84	2.18
Block 1	0.252	9.00	2.00	9.38	14.22
Block 2	0.252	9.00	2.50	15.05	30.53
Block 3	0.252	9.00	2.00	19.58	34.63
Block 4	0.252	9.00	2.00	24.12	43.70
Block 5	0.252	9.00	2.00	28.65	52.77
Block 6	0.252	9.00	2.00	33.19	61.84

- Dynamic surcharge pressures, $P_{i(ds)}$, and dynamic earth forces, $F_{i(ds)}$;

Table A.18 The results of dynamic surcharge forces for Kocaeli Earthquake

	K_A	q (kN/m ²)	H (m)	P_{ds} (kN/m ²)	F (kN/m)
Crown wall	0.121	15.00	2.00	1.82	3.63
Crown wall_1	0.121	15.00	2.50	1.82	0.91
Block 1	0.252	15.00	2.00	3.78	7.56
Block 2	0.252	15.00	2.50	3.78	9.45
Block 3	0.252	15.00	2.00	3.78	7.56
Block 4	0.252	15.00	2.00	3.78	7.56
Block 5	0.252	15.00	2.00	3.78	7.56
Block 6	0.252	15.00	2.00	3.78	7.56

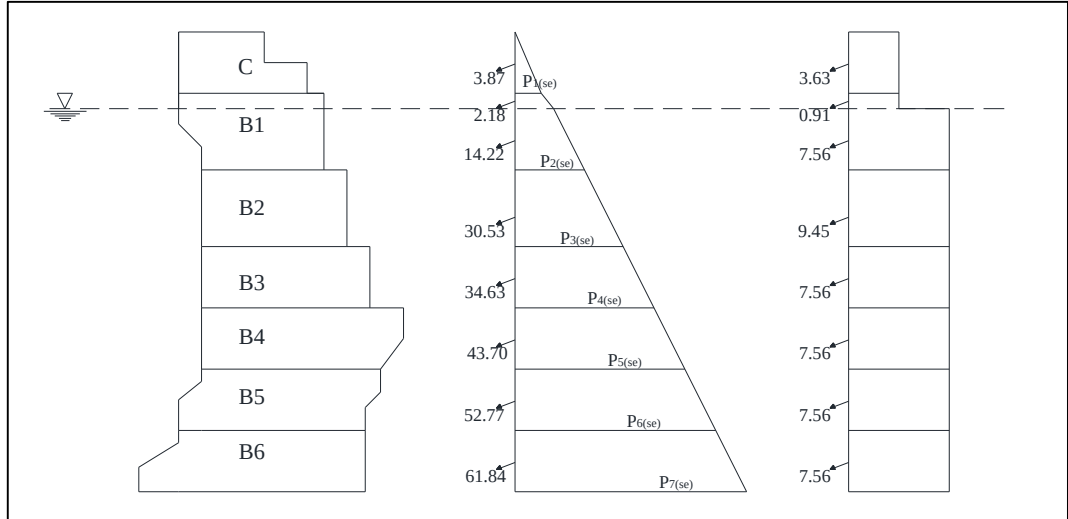


Figure A.8 The soil effect and surcharge effect on block type quay wall for dynamic case for Kocaeli Earthquake

- Dynamic water pressure, P_{dwi} , and dyanmic water forces, F_{dwi} ;

$$P_{dw} = (7/8) k_h \gamma_w \sqrt{H_w h}$$

$$F_{dw} = \left(\frac{p_{dwi} + p_{dw(i-1)}}{2} \right) h_i$$

Table A.19 The results of dynamic water pressures and forces for Kocaeli Earthquake

	H_w (m)	H (m)	k_h	γ_w (kN/m ³)	P_{dw} (kN/m ²)	F_{dw} (kN/m)
Block 1	12.5	2.00	0.20	10.00	8.80	8.80
Block 2	12.5	4.50	0.20	10.00	13.21	27.51
Block 3	12.5	6.50	0.20	10.00	15.87	29.08
Block 4	12.5	8.50	0.20	10.00	18.15	34.02
Block 5	12.5	10.50	0.20	10.00	20.17	38.32
Block 6	12.5	12.50	0.20	10.00	22.01	42.18

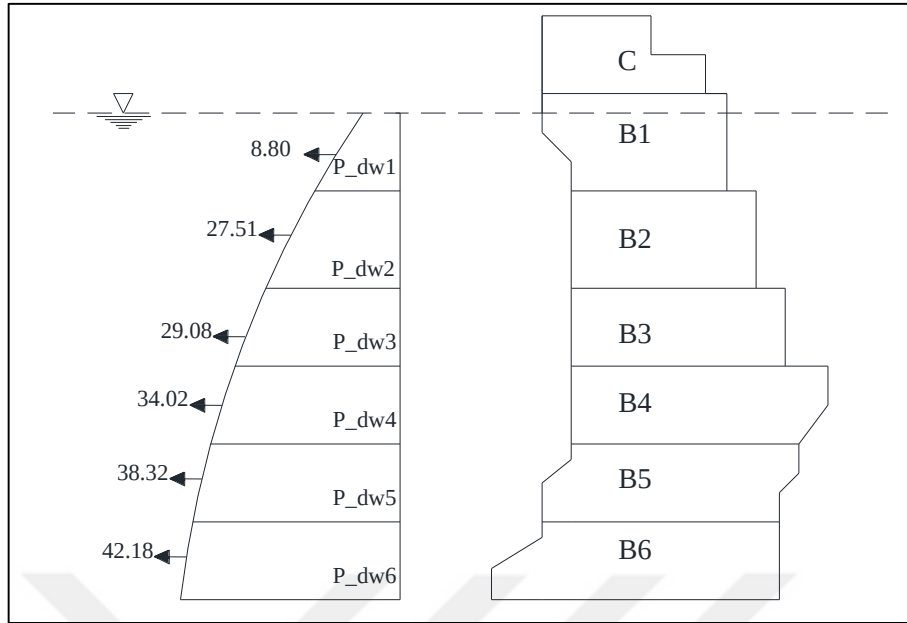


Figure A.9 The hydrodynamic forces on the block type quay wall for Kocaeli Earthquake

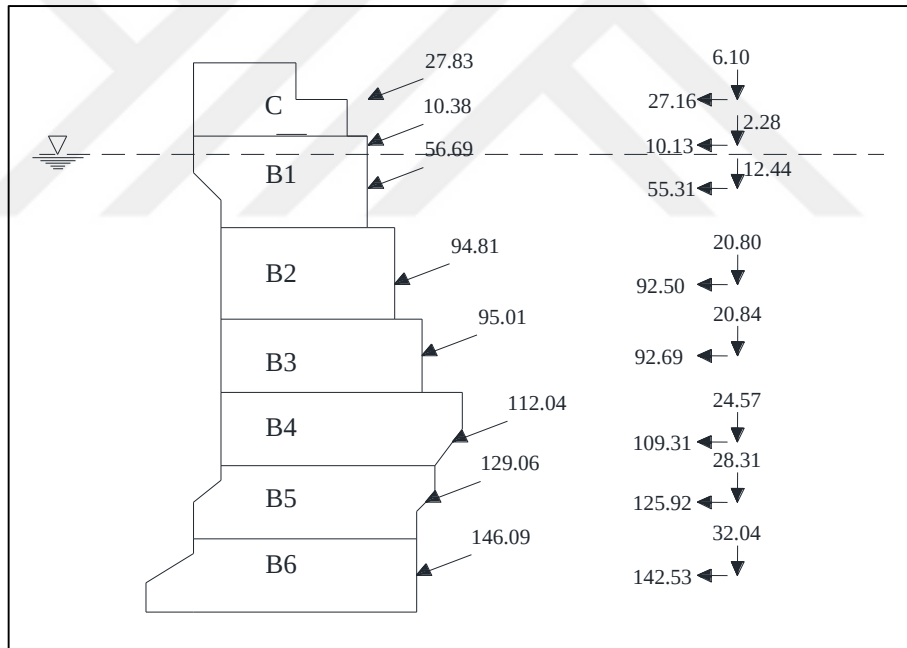
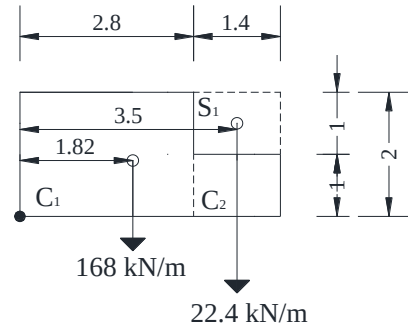


Figure A.10 Total forces on the quay wall for Kocaeli Earthquake

Vertical Forces

- Crown wall and soil**



$$W = (x) \times (y) \times (\gamma_c)$$

$$W_{c1} = 2.8 \times 2 \times 24 = 134.4 \text{ kN/m}$$

$$W_{c2} = 1.4 \times 1 \times 24 = 33.6 \text{ kN/m}$$

$$W_{ct} = 128.8 + 32.2 = 168 \text{ kN/m}$$

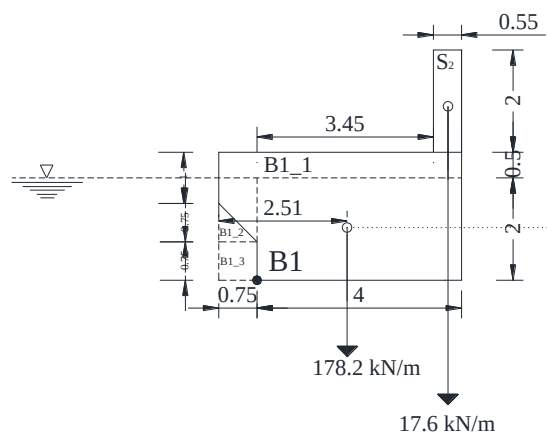
$$W_{s1} = 1.4 \times 1 \times 16 = 22.4 \text{ kN/m}$$

The centre of gravity;

$$e_{xc} = \frac{134.4 \times 1.4 + 33.6 \times (2.8 + 1.4/2)}{134.4 + 33.6} = 1.82 \text{ m}$$

$$e_{xs1} = 1.4/2 + 2.8 = 3.50 \text{ m}$$

- Block 1 and soil**



$$W_{B1_1} = 4.75 \times 0.5 \times 24 = 57 \text{ kN/m}$$

$$W_{B1_2} = (0.75 \times 0.75)/2 \times 14 = 3.94 \text{ kN/m}$$

$$W_{B1_3} = 0.75 \times 0.75 \times 14 = 7.88 \text{ kN/m}$$

$$W_{B1_4} = 2 \times 4.75 \times 14 = 133 \text{ kN/m}$$

$$W_{B1T} = 133 + 57 - 7.88 - 3.94 = 178.2 \text{ kN/m}$$

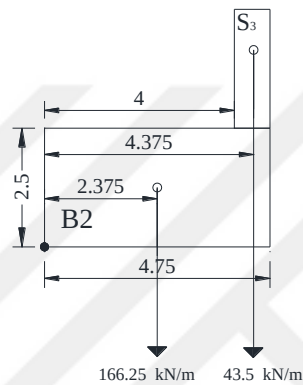
$$W_{s2} = 0.55 \times 2 \times 16 = 17.6 \text{ kN/m}$$

The center of gravity;

$$e_{xb1} = \frac{57 \times 2.375 + 133 \times 2.375 - 3.94 \times 0.50 - 7.88 \times 0.375}{178.2} = 2.51 \text{ m}$$

$$e_{xs2} = 0.55/2 + 3.45 = 3.725 \text{ m}$$

- **Block 2 and soil;**



$$W_{B2} = 4.75 \times 2.5 \times 14 = 166.25 \text{ kN/m}$$

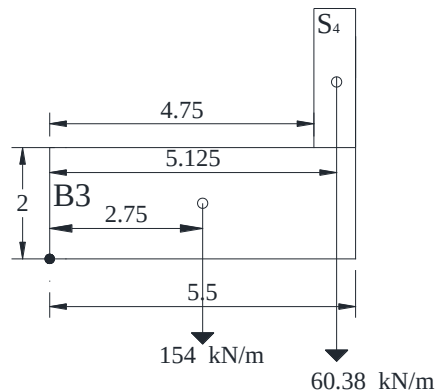
$$W_{S3} = 0.75 \times 2.5 \times 16 + 2 \times 9 \times 0.75 = 43.5 \text{ kN/m}$$

The centre of gravity;

$$e_{xB2} = 4.75/2 = 2.375 \text{ m}$$

$$e_{xS3} = 0.75/2 + 4 = 4.375 \text{ m}$$

- **Block 3 and soil;**



$$W_{B3} = 5.50 \times 2 \times 14 = 154 \text{ kN/m}$$

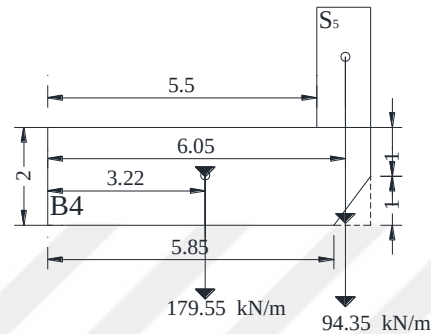
$$W_{S4} = 0.75 \times 2.5 \times 16 + 0.75 \times 4.5 \times 9 = 60.38 \text{ kN/m}$$

The centre of gravity;

$$e_{xB3} = 5.5/2 = 2.75 \text{ m}$$

$$e_{xS4} = 0.75/2 + 4.75 = 5.125 \text{ m}$$

- Block 4 and soil**



$$W_{B4_1} = 6.6 \times 2 \times 14 = 184.8 \text{ kN/m}$$

$$W_{B4_2} = (0.75 \times 1)/2 \times 14 = 5.25 \text{ kN/m}$$

$$W_{B4T} = 184.8 - 5.25 = 179.55 \text{ kN/m}$$

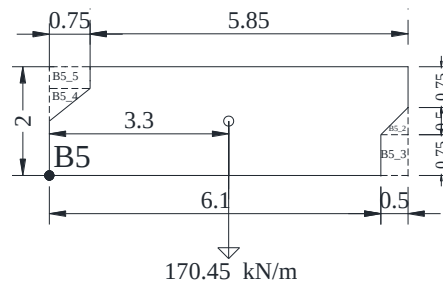
$$W_{S5} = 1.1 \times 9 \times 6.5 + 0.75 \times 2.5 \times 16 = 94.35 \text{ kN/m}$$

The centre of gravity;

$$e_{xB4} = \frac{184.8 \times 3.3 - 5.25 \times (5.85 + 0.75/3)}{179.55} = 3.22 \text{ m}$$

$$e_{xS5} = 1.1/2 + 5 = 6.05 \text{ m}$$

- Block 5;**



$$W_{B5_1} = 6.6 \times 2 \times 14 = 184.8 \text{ kN/m}$$

$$W_{B5_2} = ((0.5 \times 0.5)/2) \times 14 = 1.75 \text{ kN/m}$$

$$W_{B5_3} = 0.75 \times 0.5 \times 14 = 5.25 \text{ kN/m}$$

$$W_{B5_4} = ((0.6 \times 0.75)/2) \times 14 = 3.15 \text{ kN/m}$$

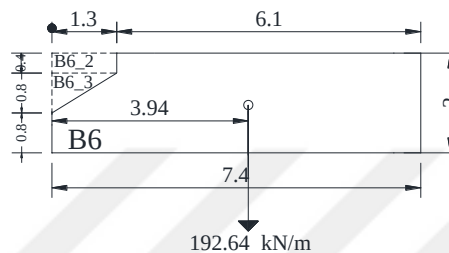
$$W_{B5_5} = 0.4 \times 0.75 \times 14 = 4.2 \text{ kN/m}$$

$$W_{B5_T} = 184.8 - 1.75 - 5.25 - 3.15 - 4.2 = 170.45 \text{ kN/m}$$

The centre of gravity;

$$e_{xB5} = \frac{184.8 \times 3.3 - 1.75 \times (6.1 + 0.5/3) - 5.25 \times (6.1 + 0.5/2) - 3.15 \times 0.25 - 4.2 \times 0.375}{170.45} = 3.30 \text{ m}$$

- Block 6;**



$$W_{B6_1} = 7.4 \times 2 \times 14 = 207.2 \text{ kN/m}$$

$$W_{B6_2} = 1.3 \times 0.4 \times 14 = 7.28 \text{ kN/m}$$

$$W_{b63} = (1.3 \times 0.8)/2 \times 14 = 7.28 \text{ kN/m}$$

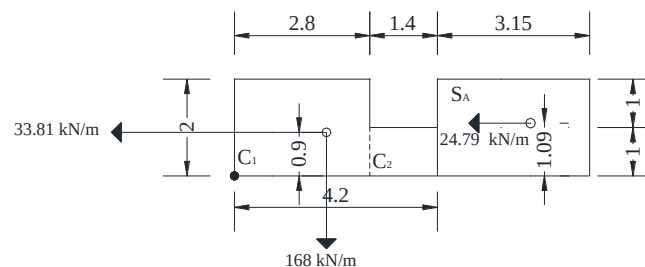
$$W_{b6T} = 207.2 - 7.28 - 7.28 = 192.64 \text{ kN/m}$$

The centre of gravity;

$$e_{xb6} = \frac{207.2 \times 3.7 - 7.28 \times 0.65 - 7.28 \times 0.65}{192.64} = 3.94 \text{ m}$$

Inertia Forces

- Crown wall**



$$W_{C_T} = 168 \text{ kN/m} ; k_h = 0.27$$

$$F_{hc} = 168 \times 0.201 = 33.81 \text{ kN/m}$$

Soil;

$$F_{S_1} = 1.4 \times 1 \times 16 \times 0.201 = 4.51 \text{ kN/m}$$

$$F_{S_A} = 3.15 \times 2 \times 16 \times 0.201 = 20.28 \text{ kN/m}$$

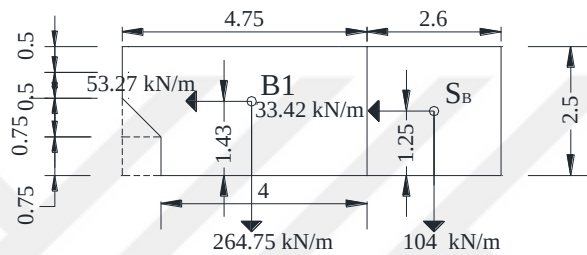
Total block forces: 168 kN/m

$$e_{yc} = \frac{134.4 \times 1 + 33.6 \times 0.5}{168} = 0.90 \text{ m}$$

Total soil forces: 24.79 kN/m

$$e_{ys1} = \frac{4.51 \times (1 + 1/2) + 20.28 \times 1}{24.79} = 1.1 \text{ m}$$

• Block 1



$$W_{B1_1} = 4.75 \times 0.5 \times 24 = 57 \text{ kN/m}$$

$$W_{B1_2} = (0.75 \times 0.75)/2 \times 24 = 6.75 \text{ kN/m}$$

$$W_{B1_3} = 0.75 \times 0.75 \times 24 = 13.5 \text{ kN/m}$$

$$W_{B1_4} = 2 \times 4.75 \times 24 = 228 \text{ kN/m}$$

$$W_{B1T} = 228 + 57 - 6.75 - 13.5 = 264.75 \text{ kN/m}$$

$$F_{h_B1} = 264.75 \times 0.201 = 53.27 \text{ kN/m}$$

$$e_{yB1} = \frac{228 \times 1 + 57 \times 2.25 - 6.75 \times 1 - 13.5 \times 0.375}{264.75} = 1.30 \text{ m}$$

Soil;

$$W_{SB_1} = 2.6 \times 0.5 \times 16 = 20.8 \text{ kN/m}$$

$$W_{SB_2} = 2.60 \times 2 \times 16 = 83.2 \text{ kN/m}$$

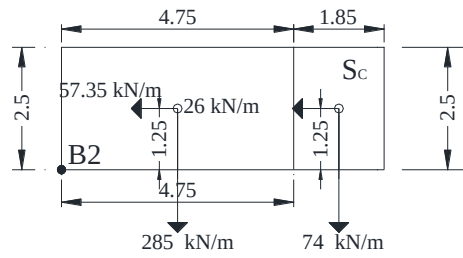
$$F_{hSB_1} = 20.8 \times 0.201 = 4.19 \text{ kN/m}$$

$$F_{hSB_2} = 83.2 \times 0.351 = 29.24 \text{ kN/m}$$

$$F_{hSB} = 29.24 + 4.19 = 33.42 \text{ kN/m}$$

$$e_{ySB} = 1.25 \text{ m}$$

- **Block 2**



$$W_{B2} = 4.75 \times 2.5 \times 24 = 285 \text{ kN/m}$$

$$F_{h_B2} = 285 \times 0.201 = 57.35 \text{ kN/m}$$

$$e_{yB2} = 1.25 \text{ m}$$

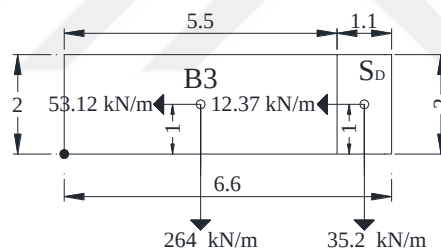
Soil;

$$W_{SC} = 1.85 \times 2.5 \times 16 = 74 \text{ kN/m}$$

$$F_{hSC} = 74 \times 0.351 = 26 \text{ kN/m}$$

$$e_{ySC} = 1.25 \text{ m}$$

- **Block 3**



$$W_{B3} = 5.5 \times 2 \times 24 = 264 \text{ kN/m}$$

$$F_{h_B3} = 264 \times 0.201 = 53.12 \text{ kN/m}$$

$$e_{yB3} = 1 \text{ m}$$

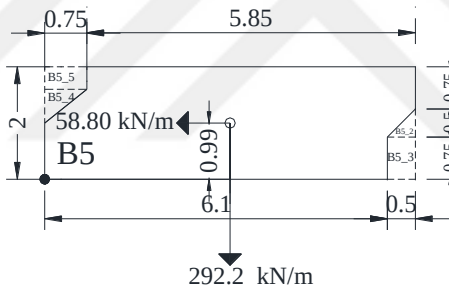
Soil;

$$W_{SD} = 1.1 \times 2 \times 16 = 35.2 \text{ kN/m}$$

$$F_{hSD} = 35.2 \times 0.351 = 12.37 \text{ kN/m}$$

$$e_{ySD} = 1 \text{ m}$$

• **Block 5**

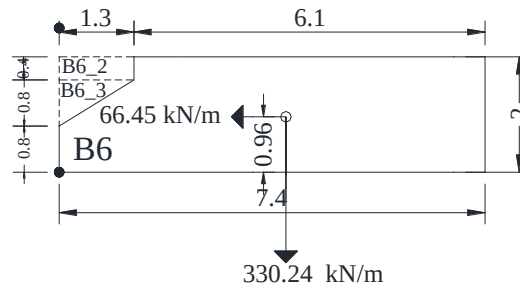


$$e_{yB4} = \frac{316.8 \times 1 - 9 \times (1/3)}{307.8} = 1.02 \text{ m}$$

$$e_{yB5} = \frac{316.8 \times 3.3 - 3 \times (0.75 + 0.5/3) - 9 \times (0.75/2) - 5.4 \times (1 + 0.6/3) - 7.2 \times 1.8}{292.2} = 0.99 \text{ m}$$

$$e_{yB5} = \frac{316.8 \times 3.3 - 3x(0.75 + 0.5/3) - 9x(0.75/2) - 5.4x(1 + 0.6/3) - 7.2x1.8}{292.2} = 0.99 \text{ m}$$

- **Block 6**



$$W_{B6_1} = 7.4 \times 2 \times 24 = 355.2 \text{ kN/m}$$

$$W_{B6_2} = 1.3 \times 0.4 \times 24 = 12.48 \text{ kN/m}$$

$$W_{B6_3} = ((1.3 \times 0.8)/2) \times 24 = 12.48 \text{ kN/m}$$

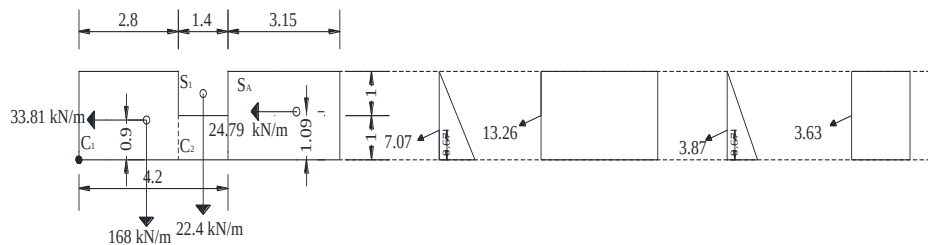
$$W_{B6_T} = 355.2 - 12.48 - 12.48 = 330.24 \text{ kN/m}$$

$$F_{h_B6} = 330.24 \times 0.201 = 66.45 \text{ kN/m}$$

$$e_{yB6} = \frac{355.2 \times 1 - 12.48 \times 1.8 - 12.48 \times (0.8 + 0.8/3)}{330.24} = 0.96 \text{ m}$$

Stability Analysis

- **Crown wall**



$$\Sigma \text{Horizontal} = 33.81 + 24.75 + 24.79 + 7.07 \times \cos(12.67) + 13.26 \times 0.5 \times \cos(12.67) + 3.87 \times \cos(12.67) + 3.63 \times \cos(12.67) = 104.04 \text{ kN/m}$$

$$\Sigma \text{Vertical} = 168 + 22.4 + 6.10 = 196.50 \text{ kN/m}$$

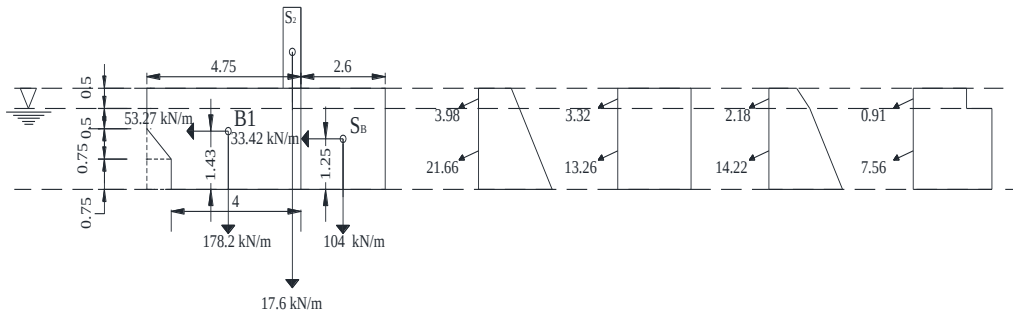
$$FS_s = \frac{196.50 \times 0.5}{104.04} = 0.94 < 1.0 \text{ (x)}$$

$$\text{Overturning moments} = 33.81 \times 0.9 + 24.79 \times 1.09 + 6.90 \times 2/3 + 12.84 \times 1 + 1.5 \times (3.78 \times 2/3) + 1.5 \times (3.54 \times 1) + 24.75 \times 2.375 = 142.88 \text{ kNm/m}$$

$$\text{Resisting moments} = 168 \times 1.82 + 22.4 \times 3.5 + 6.10 \times 4.2 = 409.79 \text{ kNm/m}$$

$$FS_o = \frac{409.79}{142.79} = 2.87 > 1.1 \text{ (OK)}$$

• **Block 1**



$$\begin{aligned}\Sigma \text{Horizontal} &= 104.04 + 53.27 + 33.42 + 3.98 \times \cos(12.67) + 21.66 \times \cos(12.67/2) \\ &\quad + 0.5 \times 3.32 \times \cos(12.67) + 0.5 \times 13.26 \times \cos(12.67/2) + 2.18 \times \cos(12.67) \\ &\quad + 14.22 \times \cos(12.67/2) + 0.91 \times \cos(12.67) + 7.56 \times \cos(12.67/2) + 8.80 \\ &= 257.80 \text{ kN/m}\end{aligned}$$

$$\Sigma \text{Vertical} = 196.50 + 178.2 + 17.6 + 8.53 = 400.82 \text{ kN/m}$$

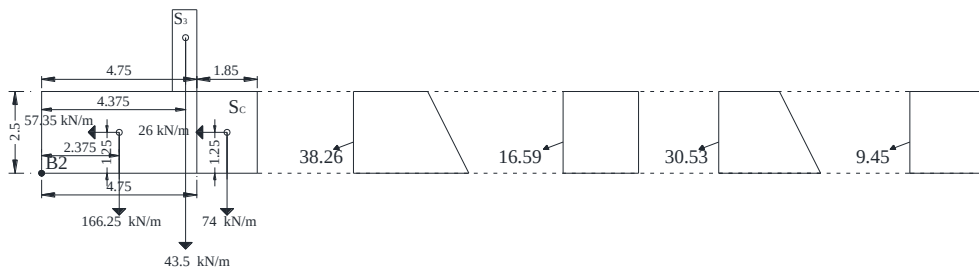
$$FS_s = \frac{400.82 \times 0.5}{257.80} = 0.78 < 1.0 \text{ (x)}$$

$$\begin{aligned}\text{Overturning moments} &= 142.88 + 104.04 \times 2.5 + 53.27 \times 1.3 + 33.42 \times 1.25 + 3.88 \times 2.24 \\ &\quad + 21.53 \times 0.93 + 3.23 \times 2.25 + 13.18 \times 1 + 2.12 \times 2.24 + 1.5 \times 14.13 \times \\ &\quad 0.9 + 0.89 \times 2.25 + 1.5 \times 7.51 \times 1 + 8.80 \times 2/3 = 606.17 \text{ kNm/m}\end{aligned}$$

$$\begin{aligned}\text{Resisting moments} &= 134.4 \times 0.65 + 33.6 \times 2.75 + 22.4 \times 2.75 + 7.19 \times 3.45 + 178.2 \times 1.76 \\ &\quad + 17.6 \times 3.725 + 10.60 \times 4.2 = 675.71 \text{ kNm/m}\end{aligned}$$

$$FS_o = \frac{606.17}{675.71} = 1.11 < 1.1 \text{ (x)}$$

• **Block 2**



$$\begin{aligned}\Sigma \text{Horizontal} &= 257.80 + 57.35 + 26 + 38.26 \times \cos(12.67/2) + 0.5 \times 16.59 \times \cos(12.67/2) \\ &\quad + 30.53 \times \cos(12.67/2) + 9.45 \times \cos(12.67/2) + 11 = 438.16 \text{ kN/m}\end{aligned}$$

$$\Sigma \text{Vertical} = 400.82 + 166.25 + 43.5 + 10.46 = 621.03 \text{ kN/m}$$

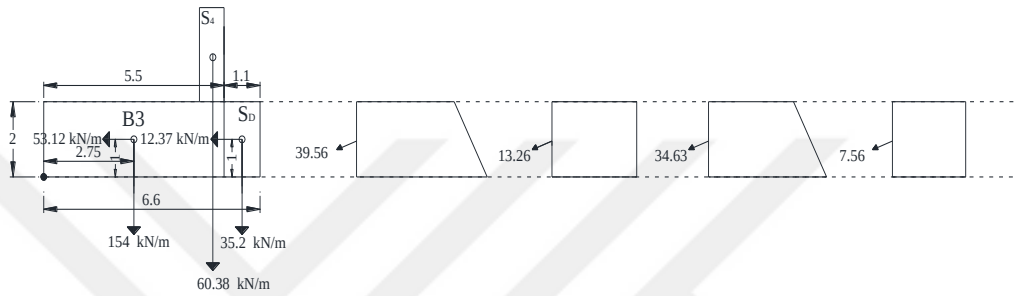
$$FS_s = \frac{621.03 \times 0.5}{438.16} = 0.71 < 1.0 \text{ (x)}$$

$$\begin{aligned}\text{Overturning moments} &= 606.17 + 257.80 \times 2.5 + 57.35 \times 1.25 + 26 \times 1.25 + 38.03 \times 1.18 + \\ &\quad 16.47 \times 1.25 + 1.5 \times 30.34 \times 1.15 + 1.5 \times 9.39 \times 1.25 + 11 \times 1.17 \\ &= 1503.15 \text{ kNm/m}\end{aligned}$$

$$\text{Resisting moments} = 675.71 + 166.25 \times 2.38 + 43.5 \times 4.38 + 10.46 \times 4.75 = 1310.54 \text{ kNm/m}$$

$$FS_O = \frac{1310.54}{1503.15} = 0.87 < 1.1 \text{ (x)}$$

• Block 3



$$\begin{aligned}\Sigma \text{Horizontal} &= 438.16 + 53.12 + 12.37 + 39.56 \times \cos(12.67/2) + 0.5 \times 13.26 \times \cos(12.67/2) \\ &\quad + 34.63 \times \cos(12.67/2) + 7.56 \times \cos(12.67/2) + 29.08 = 620.57 \text{ kN/m}\end{aligned}$$

$$\Sigma \text{Vertical} = 621.03 + 154 + 60.38 + 10.48 = 845.89 \text{ kN/m}$$

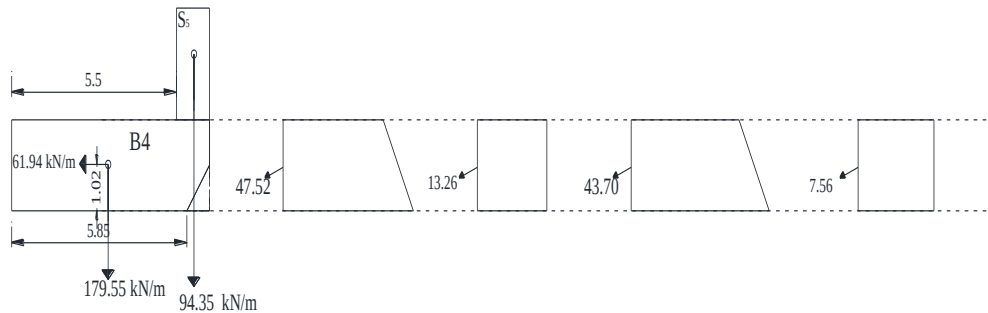
$$FS_S = \frac{845.89 \times 0.5}{620.57} = 0.68 < 1.0 \text{ (x)}$$

$$\begin{aligned}\text{Overturning moments} &= 1503.15 + 438.16 \times 2 + 53.12 \times 1 + 12.37 \times 1 + 39.32 \times 0.97 \\ &\quad + 13.18 \times 1 + 1.5 \times 34.42 \times 0.96 + 1.5 \times 7.51 \times 1 + 29.08 \times 1 \\ &= 2585.32 \text{ kNm/m}\end{aligned}$$

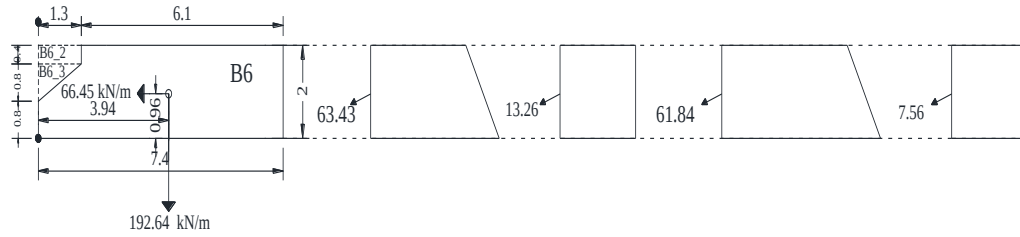
$$\text{Resisting moments} = 1310.54 + 154 \times 2.75 + 60.38 \times 5.125 + 10.48 \times 5.5 = 2101.11 \text{ kNm/m}$$

$$FS_O = \frac{2101.11}{2585.32} = 0.81 < 1.1 \text{ (x)}$$

• Block 4



- **Block 6**



$$\Sigma \text{Horizontal} = 1040.09 + 66.45 + 63.43 \times \cos(12.67/2) + 0.5 \times 13.26 \times \cos(12.67/2) + 61.84 \times \cos(12.67/2) + 7.56 \times \cos(12.67/2) + 42.18 = 1287.33 \text{ kN/m}$$

$$\Sigma \text{Vertical} = 1316.83 + 192.64 + 16.12 = 1525.59 \text{ kN/m}$$

$$F_{Ss} = \frac{1525.59 \times 0.6}{1287.33} = 0.71 < 1.0 \text{ (x)}$$

$$\text{Overturning moments} = 5950.73 + 1040.09 \times 2 + 66.45 \times 0.96 + 63.04 \times 0.98 + 13.18 \times 1 + 1.5 \times 61.47 \times 0.97 + 1.5 \times 7.51 \times 1 + 42.18 \times 0.98 = 8310.9 \text{ kNm/m}$$

$$\text{Resisting moments} = 4833.36 + 1316.83 \times 1.3 + 192.64 \times 3.94 + 16.12 \times 7.4 = 7423.5 \text{ kNm/m}$$

$$F_{So} = \frac{7423.5}{8310.9} = 0.89 < 1.1 \text{ (x)}$$

Table A.20 Factor of safety values against sliding and overturning for Kocaeli Earthquake

	F_{Ss}	F_{So}
Crown Wall	0.94	2.87
Block 1	0.78	1.11
Block 2	0.71	0.87
Block 3	0.68	0.81
Block 4	0.69	0.82
Block 5	0.63	0.81
Block 6	0.71	0.89

Table A.21 Factor of Safety against Sliding and Overturning according to Performance Level (TDSC-CRA, 2007)

Factor of Safety	Minimum Damage	Controlled Damage
Sliding	>1.2	>1.0
Overturning	>1.3	>1.1

Table A.22 The horizontal and vertical forces for sliding factor for Kocaeli Earthquake

	Static Forces						Dynamic Forces						Hydrodynamic Force	Horizontal Forces
	F_s (kN/m)	$F_{s,x}$ (kN/m)	$F_{s,y}$ (kN/m)	F_{ss} (kN/m)	$F_{ss,x}$ (kN/m)	$F_{ss,y}$ (kN/m)	F_{de} (kN/m)	$F_{de,x}$ (kN/m)	$F_{de,y}$ (kN/m)	F_{ds} (kN/m)	$F_{ds,x}$ (kN/m)	$F_{ds,y}$ (kN/m)	F_{dw} (kN/m)	Total Thrust (TT)
CW	7.07	6.90	1.55	13.26	12.94	2.91	3.87	3.78	0.85	3.63	3.54	0.80	0	104.04
CW1	3.98	3.88	0.87	3.32	3.23	0.73	2.18	2.12	0.48	0.91	0.89	0.20	0.00	257.80
B1	21.66	21.53	2.39	13.26	13.18	1.46	14.22	14.13	1.57	7.56	7.51	0.83	8.80	
B2	38.26	38.03	4.22	16.58	16.47	1.83	30.53	30.34	3.37	9.45	9.39	1.04	11.00	438.16
B3	39.56	39.32	4.36	13.26	13.18	1.46	34.63	34.42	3.82	7.56	7.51	0.83	29.08	620.57
B4	47.52	47.23	5.24	13.26	13.18	1.46	43.70	43.43	4.82	7.56	7.51	0.83	34.02	821.28
B5	55.47	55.13	6.12	13.26	13.18	1.46	52.77	52.45	5.82	7.56	7.51	0.83	38.32	1040.09
B6	63.43	63.04	7.00	13.26	13.18	1.46	61.84	61.47	6.82	7.56	7.51	0.83	42.18	1287.33

	Inertia Forces	Blocks Weight	Soil Effect		Vertical Forces
	F_h (kN/m)	W (kN/m)	F_h (kN/m)	W (kN/m)	Total Thrust (TT)
CW	33.81	168.00	24.79	22.40	196.50
CW1	53.27	178.19	33.42	17.6	400.82
B1					
B2	57.35	166.25	26.00	43.5	621.03
B3	53.12	154.00	12.37	60.375	845.89
B4	61.94	179.55	0.00	94.35	1132.14
B5	58.80	170.45	0.00	0	1316.83
B6	66.45	192.64	0.00	0	1525.59

Table A.23 The horizontal and vertical forces and epicenter point for overturning factor for Kocaeli Earthquake

	Static Forces								Dynamic Forces							
	F_s (kN/m)	$F_{s,x}$ (kN/m)	$F_{s,y}$ (kN/m)	e_y	F_{ss} (kN/m)	$F_{ss,x}$ (kN/m)	$F_{ss,y}$ (kN/m)	e_y	F_{de} (kN/m)	$F_{de,x}$ (kN/m)	$F_{de,y}$ (kN/m)	e_y	F_{ds} (kN/m)	$F_{ds,x}$ (kN/m)	$F_{ds,y}$ (kN/m)	e_y
CW	7.07	6.90	1.55	0.67	13.26	12.94	2.91	1.00	3.87	3.78	0.85	0.67	3.63	3.54	0.80	1.00
CW1	3.98	3.88	0.87	2.24	3.32	3.23	0.73	2.25	2.18	2.12	0.48	2.24	0.91	0.89	0.20	2.25
B1	21.66	21.53	2.39	0.93	13.26	13.18	1.46	1.00	14.22	14.13	1.57	0.90	7.56	7.51	0.83	1.00
B2	38.26	38.03	4.22	1.18	16.58	16.47	1.83	1.25	30.53	30.34	3.37	1.15	9.45	9.39	1.04	1.25
B3	39.56	39.32	4.36	0.97	13.26	13.18	1.46	1.00	34.63	34.42	3.82	0.96	7.56	7.51	0.83	1.00
B4	47.52	47.23	5.24	0.97	13.26	13.18	1.46	1.00	43.70	43.43	4.82	0.97	7.56	7.51	0.83	1.00
B5	55.47	55.13	6.12	0.98	13.26	13.18	1.46	1.00	52.77	52.45	5.82	0.97	7.56	7.51	0.83	1.00
B6	63.43	63.04	7.00	0.98	13.26	13.18	1.46	1.00	61.84	61.47	6.82	0.97	7.56	7.51	0.83	1.00

	Hydrodynamic Force		Inertia Forces		Soil Effect	
	F_{dw} (kN/m)	e_y	F_h (kN/m)	e_y	F_{sh} (kN/m)	e_y
CW	0.00	0.00	33.81	0.90	24.79	1.09
CW1	8.80	0.67	53.27	1.30	33.42	1.25
B1						
B2	11.00	1.17	57.35	1.25	26.00	1.25
B3	29.08	0.97	53.12	1.00	12.37	1.00
B4	34.02	0.98	61.94	1.02	0.00	0.00
B5	38.32	0.99	58.80	0.99	0.00	0.00
B6	42.18	0.98	66.45	0.96	0.00	0.00

Table A.24 The factor of safety against sliding for Kocaeli Earthquake

	Horizontal Forces	Vertical Forces	F_{ss}
Crown Wall	104.04	196.50	0.94
Block 1	257.80	400.82	0.78
Block 2	438.16	621.03	0.71
Block 3	620.57	845.89	0.68
Block 4	821.28	1132.14	0.69
Block 5	1040.09	1316.83	0.63
Block 6	1287.33	1525.59	0.71

Table A.25 The factor of safety against overturning for Kocaeli Earthquake

	Overturning Moment	Resistant Moment	F_{so}
Crown Wall	142.879	409.79	2.87
Block 1	606.167	675.71	1.11
Block 2	1503.154	1310.54	0.87
Block 3	2585.315	2101.11	0.81
Block 4	4056.384	3331.64	0.82
Block 5	5950.073	4833.36	0.81
Block 6	8310.869	7423.50	0.89

APPENDIX–III: The results of FE analyses performed using free field boundary modelling method

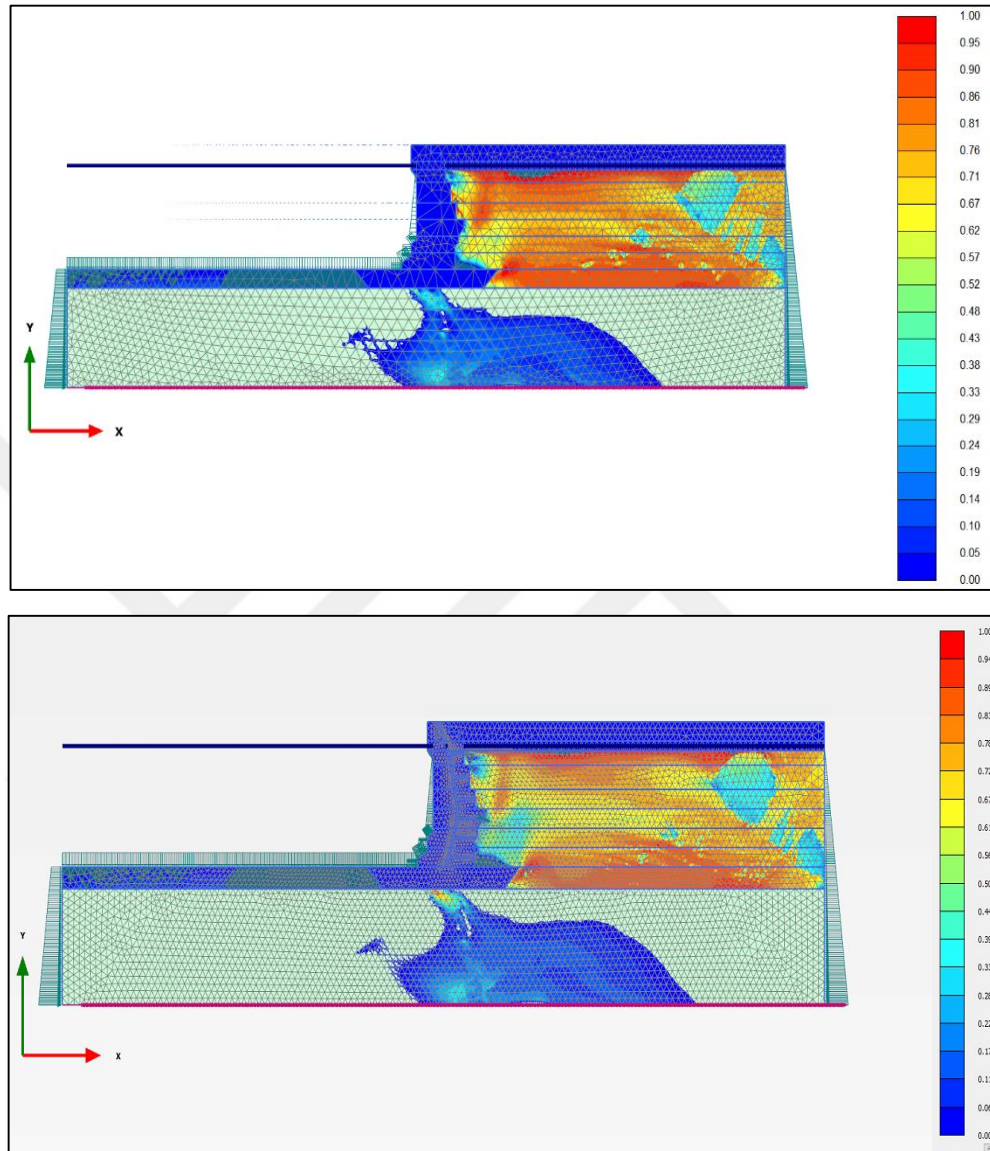


Figure A.11 The distribution of excess pore water pressure ratio in dynamic finite element model between the original wall and new model for Kobe Earthquake

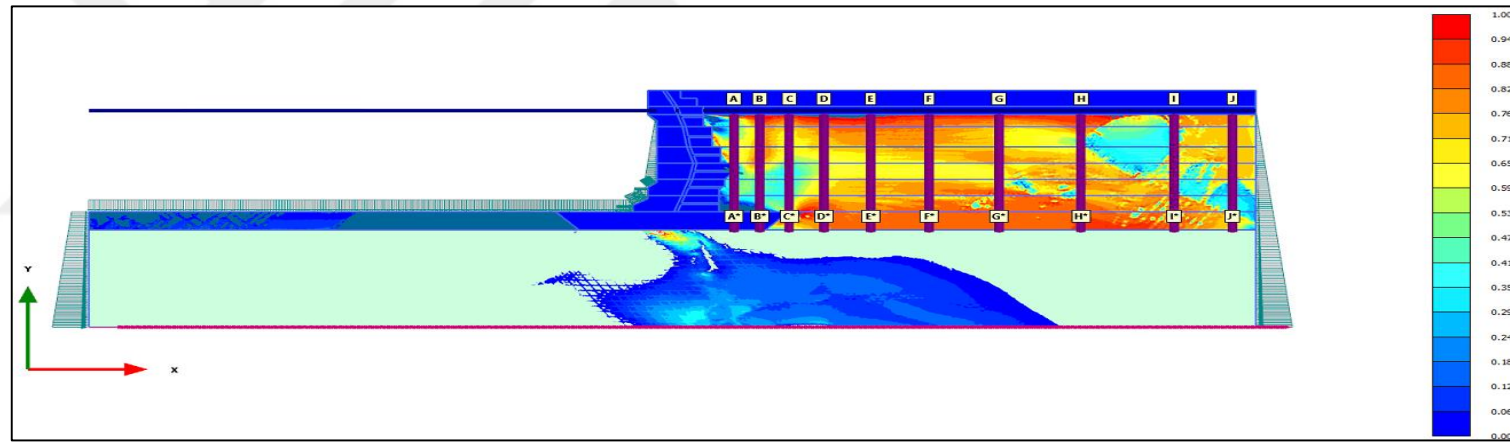


Figure A.12 The cross-sections for reviewing liquefaction behavior

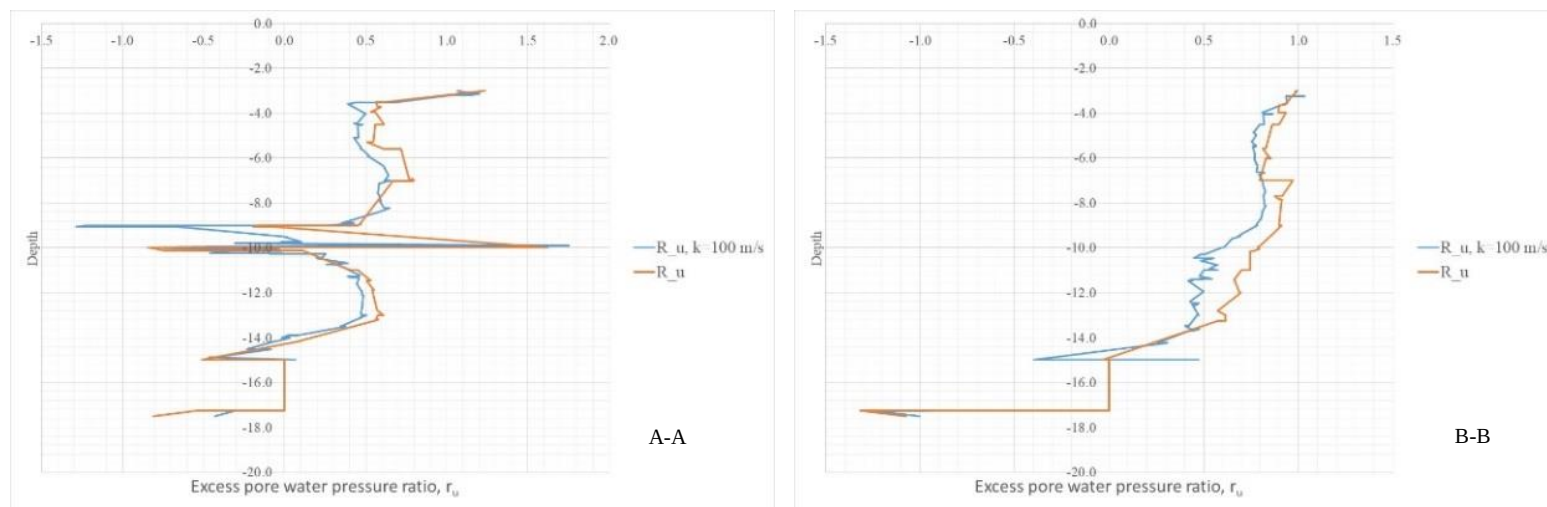


Figure A.13 Comparison of excess pore water ratio between the original wall and new model for Kobe Earthquake

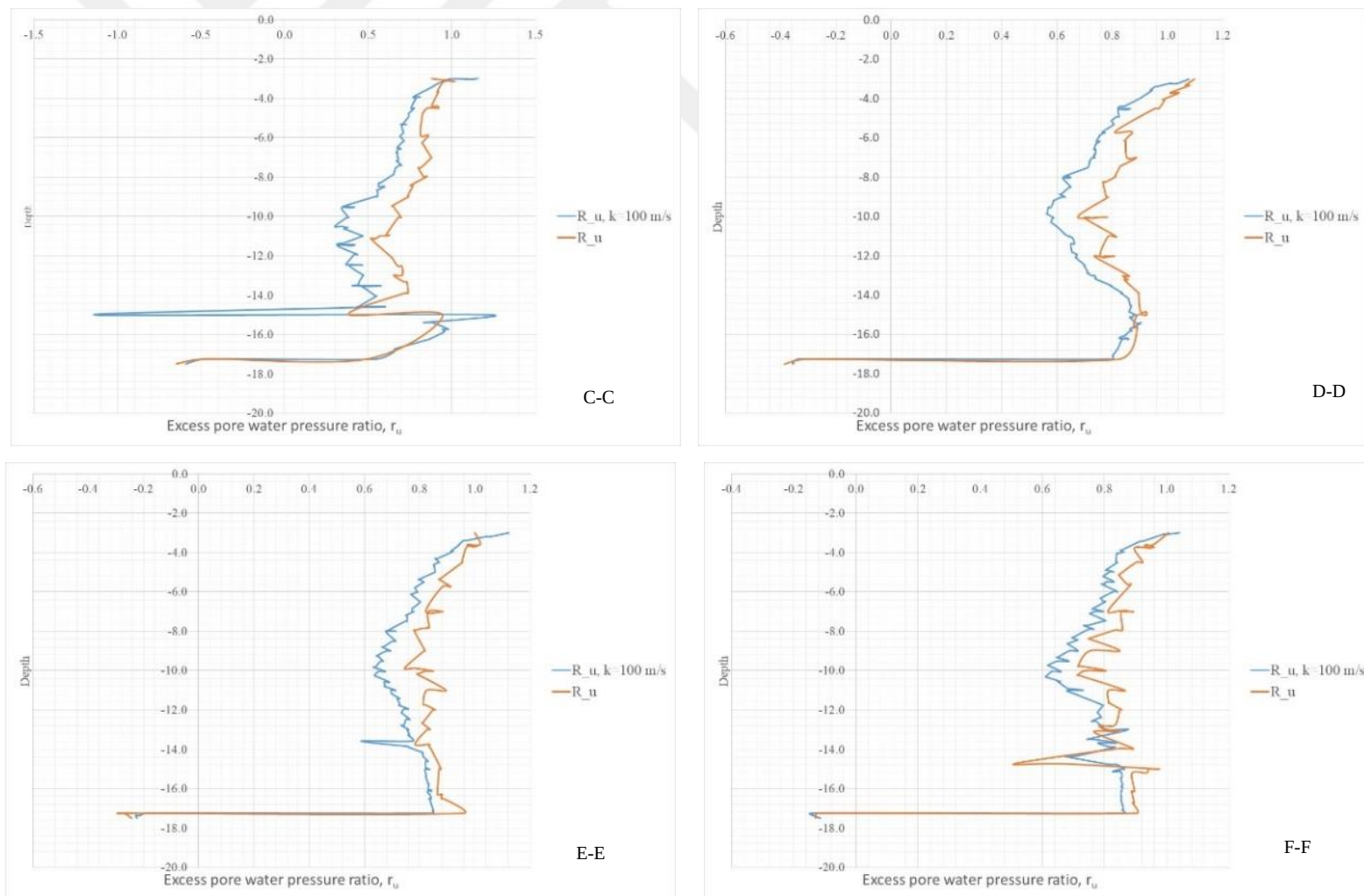


Figure A.13 continues

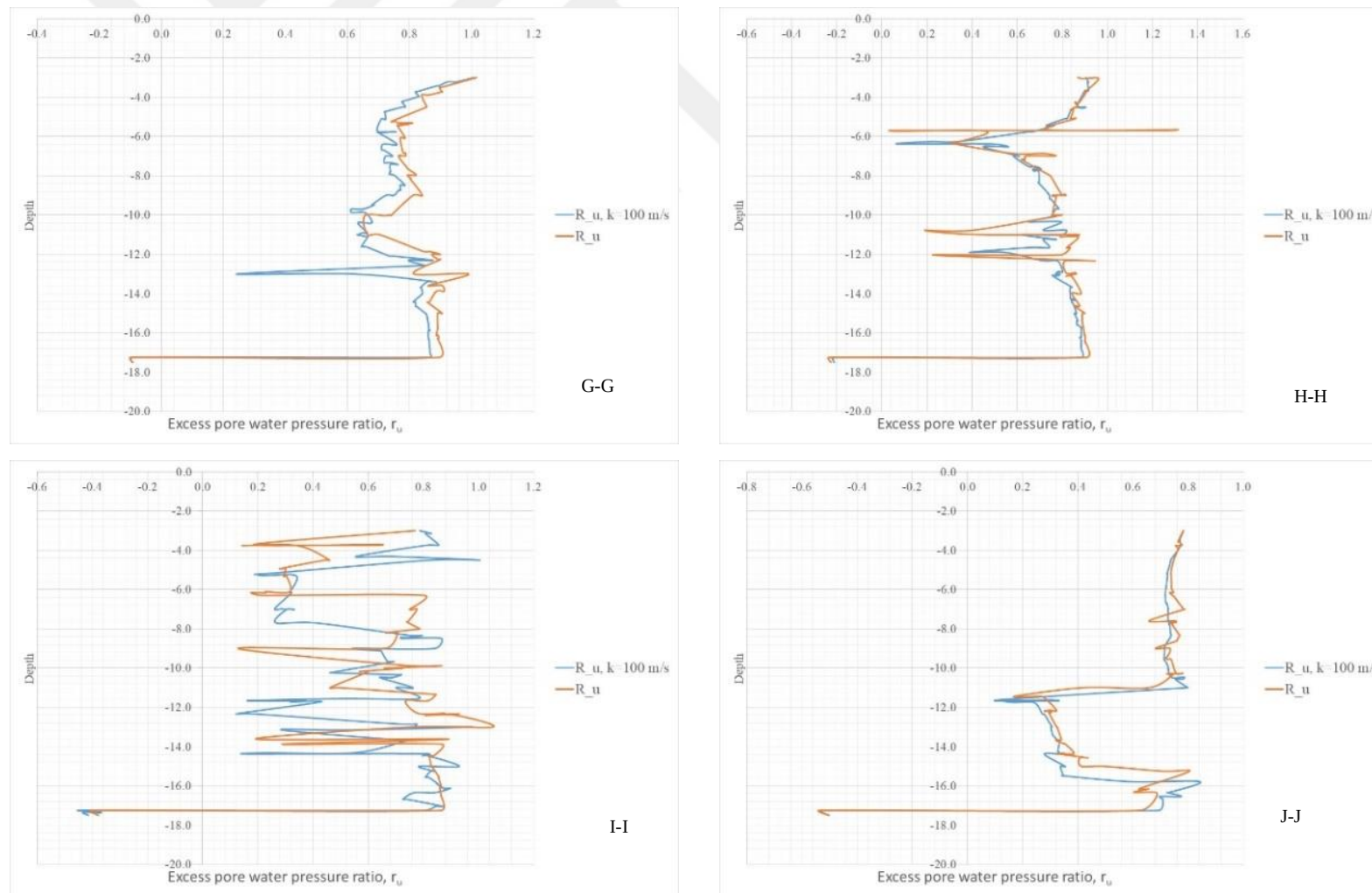
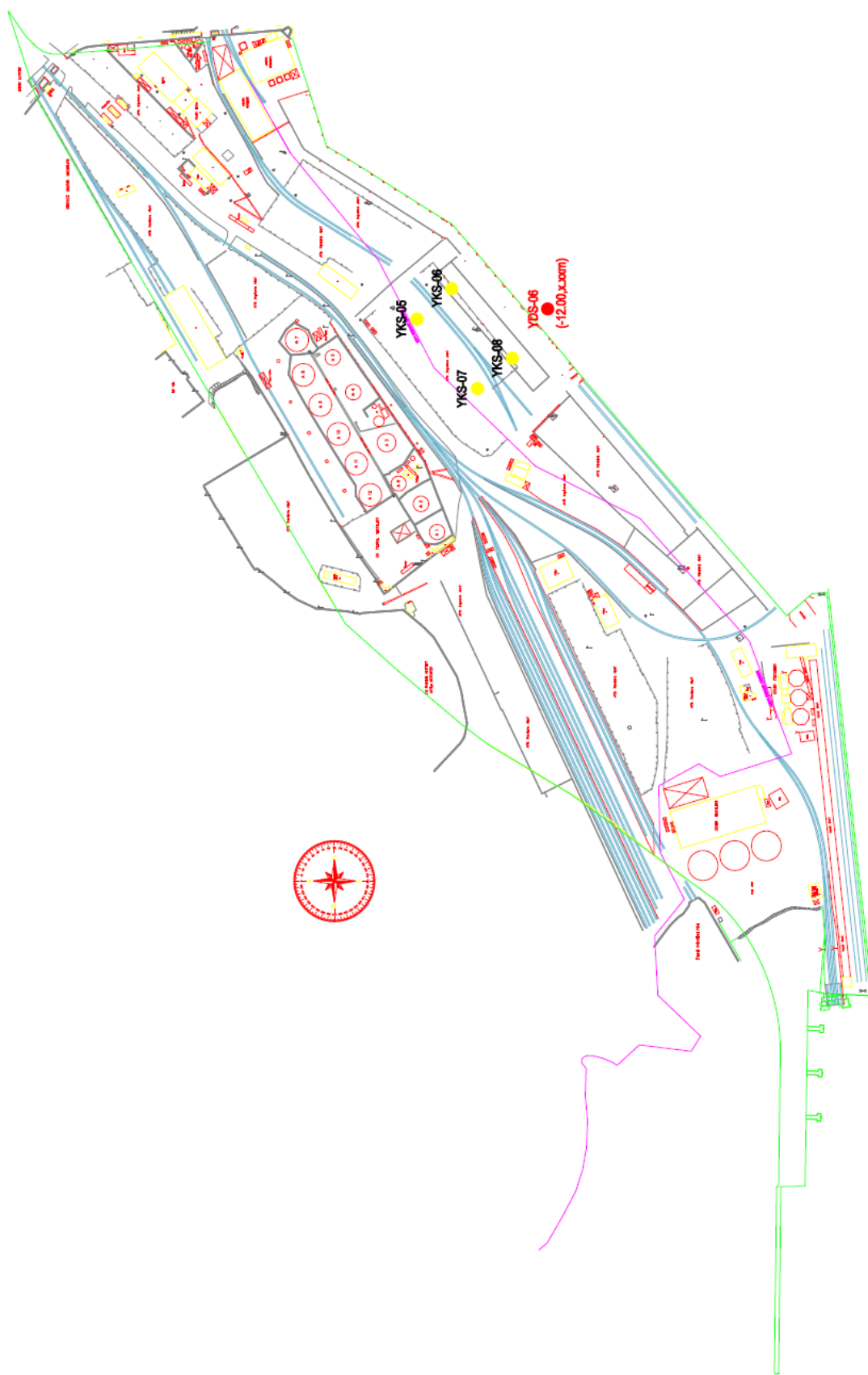


Figure A.13 continues

APPENDIX-IV : The borehole location plan and the borehole logs



SONDAJ LOGU

SONDAJ NO :
YDS-6

İŞVEREN:
PROJE:

SAFİPORT LİMANI

SONDAJ YERİ : DERİNCE/KOCAELİ

[illegible]

**GEOSAN**DOĞAL KAYNAKLAR VE HAMMADDELER İNŞAAT SANAYİ VE TİC. A.Ş.
Şenlikköy Yenibağlar Sok. Villa Bekan No. 29/4 Bakırköy-İst. Tel.: 224 67 71 Fax: 240 60 84**SONDAJ LOGU**SONDAJ NO :
YKS-5İŞVEREN:
PROJE: SAFİPORT LİMANI

SONDAJ YERİ : DERİNCE/KOCAELİ

KAYA						ZEMİN										Derinlik (m)	Litoloji Logu	Sımf.		KOORDİNATLAR: X: 0 486 063 Y: 4 512 958 (WGS-84, 3° Dilim, DOM 30°) Ağız kotu (Z): AÇIKLAMALAR																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
TCR		SCR		RQD		Veyn	SPT				Örnek			Zemin	Kaya																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
%	Grafik	%	Grafik	%	Grafik		Grafik				Darbe Sayısı							UD	D		No																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
						0	10	20	30	40	50	60	0-15	15-30	30-45	N ₃₀																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
																			1.50	1.95	3.00	3.45	4.50	4.95	1	2	3																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																

Sondaj Tipi : Rotary

Yeraltısuyu
Derinliği : -

Mühendis : Mazlum ÖZTÜRK

Sondaj Derinliği : 4.95 m

Başlangıç / Bitiş Tarihi :
08.05.2016 / 08.05.2016

Sondör : Seyfettin KİP

Kontrol : Can AKIN

Sayfa : 1/1

İŞVEREN:
PROJE:

SAFİPORT LİMANI

SONDAJ YERİ : DERİNCE/KOCAELİ

KAYA						ZEMİN										Derinlik (m)	Litoloji Logu	Sınıf.		KOORDİNATLAR: X: 0 486 096 Y: 4 512 920 (WGS-84, 3° Dilim, DOM 30°) Ağız kotu (Z): AÇIKLAMALAR		
TCR		SCR		RQD		Veyn	SPT				Örnek			Zemin	Kaya							
%	Grafik	%	Grafik	%	Grafik		Grafik	Darbe Sayısı			UD	D	No									
						0	10	20	30	40	50	60	0-15	15-30	30-45	N ₃₀						
																	0.00 0.30				BETON	
														1.50 1.95	1				DOLGU: Çakıllı, bloklu, kumlu.			
														3.00 3.45	2	3.00				KUM(DOĞAL DENİZ TABANI): İnce-kaba daneli, kuvars çakıllı, kavkı parçalı, kokulu, siyahımsı renkli.		
														4.50 4.95	3	4.95				ORTA SIKI Sondaj Sonu		
																	-10.00					
																		-15.00				
																		-20.00				
																		-25.00				
																		-30.00				

Sondaj Tipi : Rotary

Yeraltısuyu
Derinliği : -

Mühendis : Mazlum ÖZTÜRK

Sondaj Derinliği : 4.95 m

Başlangıç / Bitiş Tarihi :
07.05.2016 / 07.05.2016

Sondör : Seyfettin KİP

Kontrol : Can AKIN

Sayfa : 1/1

İŞVEREN:
PROJE:

SAFİPORT LİMANI

SONDAJ YERİ : DERİNCE/KOCAELİ

[illegible]

Sondaj Tipi : Rotary

Yeraltısuyu Derinliği : -

Mühendis : Mazlum ÖZTÜRK

Sondaj Derinliği : 4.95 m

Başlangıç / Bitiş Tarihi :
07.05.2016 / 07.05.2016

Sondör : Seyfettin KİP

Kontrol : Can AKIN

Sayfa : 1/1

İŞVEREN:
PROJE:

SONDAJ YERİ : DERİNCE/KOCAELİ

KAYA						ZEMİN										Derinlik (m)	Litoloji Logu	Sımf.		KOORDİNATLAR: X: 0 486 021 Y: 4 512 855 (WGS-84, 3° Dilim, DOM 30°) Ağız kotu (Z): AÇIKLAMALAR			
TCR		SCR		RQD		Veyn	SPT				Örnek			Zemin	Kaya								
%	Grafik	%	Grafik	%	Grafik		Grafik				Darbe Sayısı							UD	D		No		
						0	10	20	30	40	50	60	0-15	15-30	30-45	N ₃₀				0.00	0.20	BETON	
												7	9	15	24	1.50	1.95	1	3.00	3.45	2	3.00	KUM(DOĞAL DENİZ TABANI): Üst seviyeler kaba daneli, köşeli-yarı yuvarlak şekilli, kuvar-kiltaş kökenli çakıllı, seyrek ince çakıllı, kokulu, siyahımsı renkli. ORTA KATI-KATI
												2	3	4	7	4.50	4.95	3	5.00	Sondaj Sonu			
												2	3	4	7	6.00	6.45	4	6.45				
																		-10.00					
																		-15.00					
																		-20.00					
																		-25.00					
																		-30.00					

Sondaj Tipi : Rotary

Yeraltısuyu
Derinliği : —

Mühendis : Mazlum ÖZTÜRK

Sondaj Derinliği : 6.45 m

Başlangıç / Bitiş Tarihi :
07.05.2016 / 07.05.2016

Sondör : Seyfettin KİP

Kontrol : Can AKIN

Sayfa : 1/1