NEW SEISMIC DESIGN APPROACHES for BLOCK TYPE QUAY WALLS

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This is to certify that we have read this thesis and that in out opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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ABSTRACT

NEW SEISMIC DESIGN APPROACHES FOR BLOCK TYPE QUAY WALLS

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In this study, new design approaches are introduced for the seismic design of block type quay walls after reviewing the conventional methodologies. Within the development of the new design approaches an inverse triangular dynamic pressures distributions are applied to define both seismic earth pressures and seismic surcharge pressures. Differently from the conventional design methodology, the hydrodynamic forces are taken into consideration while dynamic forces are specified and equivalent unit weight concept is used during the both static and dynamic calculations. Compatibility of this new design approaches are tested by case studies, for the site and it is seen that the numerical results are in good agreement qualitatively with field measurements.

Keywords: Block type quay walls, conventional method, static and dynamic earth pressure, force-moment balance.

ÖZ

BLOK TİPİ RIHTIM DUVARLAR İÇİN YENİ TASARIM YAKLAŞIMLARI

Hülya KARAKUŞ
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Bu çalışmada, geleneksel metodların yeniden incelenmesi ile blok tipi rıhtım duvarların sismik tasarımı için yeni tasarım yaklaşımları ortaya konulmaktadır. Yeni tasarım yaklaşımlarında sismik toprak basınçlarının ve sismik sürsarj basınçlarının tanımlanabilmesi için ters üçgen dinamik basınç dağılımları uygulanmaktadır. Geleneksel tasarım yöntemlerinden farklı olarak, dinamik kuvvetler belirlenirken hidrodinamik kuvvetler de gözönünde bulundurulmuş ve statik ve dinamik hesaplamalar sırasında eşdeğer birim ağırlık kavramı kullanılmıştır. Yeni tasarım yaklaşımlarının uyumluluğu bir bölge için örnek çalışmalar kullanılarak test edilmiş ve elde edilen sayısal sonuçların gözlemsel ölçümler ile uyum içinde olduğu görülmüştür.

Anahtar Kelimeler: Blok tipi rıhtım duvarlar, geleneksel yöntem, statik ve dinamik toprak basıncı, kuvvet-moment dengesi.

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Dedicated to my family, my teachers and my friends

TABLE OF CONTENTS

LIST OF FIGURES

FIGURE

LIST OF TABLES

TABLE

LIST OF SYMBOLS

a_{max}	ground surface acceleration
a,b,c	length of the soil wedge
a_h	horizontal pseudostatic acceleration
a_v	vertical pseudostatic acceleration
A_0	effective ground acceleration coefficient
A_{10}	effective seismic coefficient (a_{max} / g for L1)
A_{20}	effective seismic coefficient (a_{max} / g for L2)
A_i	area
BF	bollard force
c	cohesion of soil (kN/m^2)
c_1	force coefficient
c_2	coefficient
e_i	earth pressures
E_i	earth force
E_0	bollard force (BF/2)
F_a	short period soil coefficient
F_{de}	dynamic earth force
F_{ds}	dynamic surcharge force
F_e	static earth force
F_h	horizontal force
F_{Hi}	total horizontal force for block i
$F_{i(de)}$	dynamic earth force for block i
$F_{i(ds)}$	dynamic surcharge force for block i
F_{dwi}	hydrodynamic force
$F_{i(se)}$	static earth force for block i
$F_{i(ss)}$	static surcharge force for block i
F_s	static surcharge force
F_v	vertical force
F_{vi}	total vertical force for block i
FS_s	factor of safety against sliding

FS_o	factor of safety against overturning
g	gravity acceleration
h	water depth for each block
h_i	height for each block
h_j	thickness of the soil class j^{th}
H	wall height
H_w	total water depth
H_{sub}	submerged soil height
I	structure importance coefficient
k_h	horizontal seismic coefficient
k_v	vertical seismic coefficient
k'_h	modified seismic coefficient
K_A	active earth pressure coefficient
K_{AE}	dynamic active earth pressure coefficient
K_{Ah}	horizontal active earth pressure coefficient
K_{Ahd}	horizontal active dynamic earth pressure coefficient
$K_{ai,d}$	active pressure coefficient for soil class i^{th}
K_d	dynamic active pressure coefficient
$\sum M_{r\,i}$	sum of the resisting moments around the toe of the wall for block i
$\sum M_{on}$	sum of the overturning moments around the toe of the wall for block i
ND	number of the dry soil class according to ground level
P_A	active earth pressure
P_{ad}	dynamic active force due to the weight of the soil
$p_{ad}(z)$	exchange function of dynamic active pressure due to the weight of soil
p_{ai}	active pressure for soil class i^{th}
$p_{ai,s}$	static active earth pressure for soil class i^{th}
$p_{ai,d}$	dynamic active earth pressure for soil class i^{th}
$p_{ai,ds}$	dynamic surcharge earth pressure for soil class i^{th}
P_{dwi}	hydrodynamic pressure for block i
P_i	weight of the each block
$P_{i(de)}$	dynamic earth pressure for block i

$P_{i(ds)}$	dynamic surcharge pressure for block i
$P_{i(se)}$	static earth pressure for block i
$P_{i(ss)}$	static surcharge pressure for block i
ΔP_{AE}	dynamic component
q	surcharge load
$q_{ad}(z)$	exchange function of dynamic active pressure due to the uniform surcharge load throughout the height of the structure
q_{sur}	uniformly distributed surcharge
Q_{ad}	dynamic active force due to the uniform surcharge load
S_{MS}	spectral acceleration value with respect to soil classification
S_s	spectral acceleration
W	weight of the failure mass
W_{dry}	dry weight of the block or soil
W_{sub}	submerged weight of the block or soil
x_i	moment arm for horizontal force
y_i	moment arm for vertical forces
Z_i	seismic force
α_A	inclination of the ground surface behind the wall
α_{ae}	active angle of failure
β	inclined angle with the horizontal
γ	unit weight of soil (kN/m^3)
γ_{b-conc}	submerged unit weight of concrete
γ_{conc}	unit weight of concrete
γ_{bj}	submerged unit weight of soil for soil class jth
γ_d	dry unit weight of soil (kN/m^3)
γ_{eq}	equivalent unit weight
γ_j	dry unit weight of soil for soil class jth
γ_s	submerged unit weight of soil (kN/m^3)
γ_{sat}	saturated total unit weight (kN/m^3)
γ_w	unit weight of seawater
δ	friction angle between wall and soil

θ	angle between the back of the retaining wall and the vertical plane
λ	coefficient
μ	coefficient of friction between the bottom of the wall body and the foundation
λ	internal friction angle
ψ	seismic inertia angle

CHAPTER 1

INTRODUCTION

For several years, people are trying to move over water, because they want to explore and conquer the world particularly from an economical point of view. Therefore traders and conquerors were sailing the oceans. Those people needed places to berth their ships, which later grow out to harbors. From these developments not only a large growth of prosperity is visible, but also a growth in knowledge about new technologies are occurred (Quay Wall Handbook, 2005). Ports and harbors play significant roles in the economic and cultural development of coastal communities, the location and nature of ports and harbors make them vulnerable to a wide range of hazards. If ports and harbors are built on fill material or soft natural material, or are surrounded by steep slopes, they can be suffered by earthquakes, tsunamis, and landslides.

Observation probability of a large earthquake may be a rare event near a major city but the economical and the social impact of this natural event can be devastating (PIANC, 2001). Although, it is known that ports are critical civil infrastructure system for centuries, it is only since the mid-twentieth century that seismic provisions for port structures have been adopted in design practice. However, historical data point out a lots of ports such as Kushiro Port, Kobe Port, Oakland Port, Port Vila and Derince Port were damaged seriously from earthquake, unfortunately, seismic risks at ports have not already received the proper amount of attention.

The heavy damage was observed on coastal structures such as refineries, petrochemical plants and ports the eastern Marmara earthquake occurred on 17 August 1999 with an Mw=7.4 and İzmit Bay and north-west Turkey had been seriously effected from this earthquake. Especially, earthquake was caused crucial damage mostly on ***block type quay walls*** at Derince Port in İzmit.

Quay walls are earth retaining structures for the mooring of ships. Due to the demanding big amount of investment and the large loads on the structure, which will increase in the future because of the trade, the design and construction of a quay wall becomes more interesting and complicated day by day. Block type wall is the simplest type of gravity quay wall, which consists of blocks of concrete or natural stone placed from the water side on a foundation consisting of a layer of gravel or crushed stone on top of each other. Blocks maintain their stability through friction between themselves and between the bottom block and the seabed.

The design of block type quay walls should be done considering stability, serviceability and safety as well as economy. Therefore several design guidelines are available to give recommendations for the design and construction of block type quay walls. And these guidelines use several approaches to evaluation of seismic slope stability, ranging from simple to complex, are available and it can be divided into three primary groups: pseudostatic methods; sliding block methods; and stress-deformation methods.

By using the Pseudostatic Method block type quay walls are designed to resist a prescribed level of seismic force specified as a fraction of gravity according to force-balance approach. This approach has contributed to the acceptable seismic performance of port structures, particularly when the earthquake motions are more or less within the prescribed design level (PIANC, 2001).

In Chapter 2, general information is given about the quay walls. The historical background, types and features of quay walls are defined briefly.

In Chapter 3, certain numbers of important studies, both theoretical and experimental, are carried out which define several approaches to make an appropriate seismic design for block type quay walls.

In Chapter 4, several design guidelines are explained and compared to each other. Thus, uncertainties and difficulties are determined during this process and new approaches are proposed to achieve this negativeness. The theoretical background is also given related to the new approaches.

In Chapter 5, stability computation for block type quay walls is carried out for Derince Port during 17 August 1999 eastern Marmara earthquake. Damages were observed at wharves 6, 7 and 8 in Derince Port. Inputs of the case studies and the calculation steps are presented in detailed. By using new approaches different cases are tested and the results are compared. Stability computation for block type quay walls is also carried out for Derince Port by using the Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007). The similarities and dissimilarities are explained briefly for all approaches.

In Chapter 6, conclusion and discussion are presented and future studies are recommended.

CHAPTER II

QUAYWALLS

2.1 Historical Background of Quay Walls

Quay walls are earth-retaining structures at which ships can berth. In order to provide moorings for ships and fendering to absorb the impacts of the vessels, they are usually equipped with bollards. Quay walls are used for the transshipment of goods by cranes or heavy equipment that moves alongside the ship. The superstructure is constructed and the quay is usually equipped with rails for cranes and with channels for the cables that supply power to the cranes. The foundation must provide the necessary stability. The entire structure must be able to satisfy numerous requirements imposed by soil conditions, water levels, and the size of ships and loads, supplemented by the specific demands placed upon it by the users and the managers of the quay. Owing to the growth of transport by water, the role of quay walls in determining the future design of ports will continue to increase, however design and the construction of quays is no simple matter.

For ages people are trying to move over water, because they want to explore and conquer the world, but also from an economical point of view. Therefore traders and conquerors were sailing the oceans. Those people needed places to berth their ships, which later grow out to harbors. From these developments not only a large growth of prosperity is visible, but also a growth in knowledge about new technologies. The oldest known port lies in India near Lothal and was already functioning 4000 years ago. It is founded due to a large trade between countries in Asia. Also in the Mediterranean, harbors were formed for trade. Alexandria was the last three ages before Christ the main trade centre in this region. Also the construction of harbors developed in these ages. The Romans were the first who used a kind of concrete for the construction of quay walls.

In the middle Ages the Vikings sailed the Western European waters with very fast ships. In this period there were two major problems: siltation of harbors and the poor equipment available in the harbor. In general there were no quay walls of stone and the cargo had to be transshipped by hand. Later, cranes became available to do this work, but with these cranes the next problem raised: strong subsoil was needed. This played a very important role in the development of quay walls with vertical bearing capacity.

In the nineteenth century the steam engine was presented and in the twentieth century there was a large development in the tonnage of ships. The consequences were larger ships with a larger draught. The draught of the ship has a lot of influence on the retaining height of the quay wall. Another consequence is the growing possibility of self-berthing of the ships and the extra scour due to propeller currents. Also the method of transshipment changed, which lead to higher loads at the quay and larger quay walls. All these developments lead to the development of a quay wall piled up by stones to a sophisticated design.

Figure 2.1: Roman ship transporting from Egypt to port of Ostia (Quay Wall Handbook, 2005)

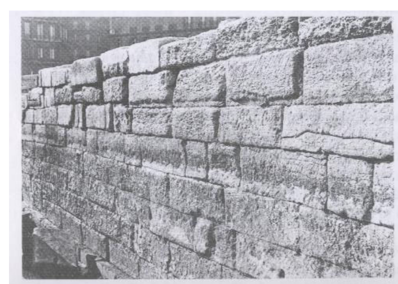


Figure 2.2: The exposure of quay wall close to Marseilles, constructed before the Christian(Quay Wall Handbook, 2005)

2.2 FUNCTION, FEATURES, MAIN TYPES & FAILURES of QUAY WALLS

2.2.1 Function of Quay Walls

A quay wall is a soil retaining structure, which occurs in many shapes. All these structures have the same function such as; mooring place for ships, soil retaining function, bearing capacity for crane loads, goods and storage, sometimes a water retaining function.

2.2.2 Features of Quay Walls

The requirements of the quay wall show variety according to the users as for the handling of freight, there must be a big enough storage area and that has sufficient bearing capacity to provide for future transhipment storage and transport and for the ships, there must be sufficient draught for the biggest vessels to berth.

In addition the following requirements are;

- The design and construction of quay wall must be well and there must be reasonable price quality relationship during the design and construction stages.
- The quay should have a low maintenance requirement and a long lifetime.
- The area must be sufficiently elevated to remain dry at high tide
- Water levels, tidal influences, soil characteristics of the ground and the climatic conditions of every place in the world are different, so great deal of experience, ingenuity and creativity should be gathered to make an optimum design essential in the design of quay walls.

In order to provide berthing facilities for ships, the wall must retain soil for the behind the quay, provide bearing capacity to carry loads imposed by the transshipment of freight and cranes and freight storage facilities, possibly also serve as a water retaining wall for the areas lying behind during periods with high water

2.2.3 Main Types of Quay Walls

3 main types of structures can be considered; gravity walls, embedded walls and open berth quays.

Gravity quay walls might have different types such as; block walls, L-walls, caisson wall, cellular wall and reinforced earth structure.

2.2.3.1 Gravity Quay Walls

A gravity quay wall is made of a caisson or other rigid wall put on the seabed, and maintains its stability through friction at the bottom of the wall (PIANC, 2001). These types of quay walls are so heavy and they cannot tilt or slide.

Gravity quay wall construction is suitable for;

- When the subsoil consists of rock or very firm sand
- When the subsoil has sufficient bearing capacity

The superstructure is used for berthing ships, such as bollards. To provide protection both ships and quay from damage, rubber and wooden fenders are installed on the quay. It is also important to protect the bed of the harbor in front of the soil retaining structure against erosion. Drainage can be necessary due to the falling and by doing this excess of pore pressure behind the structure can be prevented.

2.2.3.1.1 Block Type Walls: Block type wall is the simplest type of gravity wall, which consists of blocks of concrete or natural stone placed from the water side on a foundation consisting of a layer of gravel or crushed stone on top of each other. After placing, the blocks a reinforced concrete cap is placed as cast in situ. Block walls require much building material however labor necessity is relatively little. The height of this structure exceeds 20 m. It is important to have a good filter structure behind the wall to prevent the leakage of soil. This filter structure should involve thick filling of rock fill material with a good filter structure.

Blocks maintain their stability through friction between themselves and between the bottom block and the seabed. Typical failure modes during earthquakes involve seaward displacement, settlement, and tilting of blocks. Figure 2.3 shows typical section of block type wall.

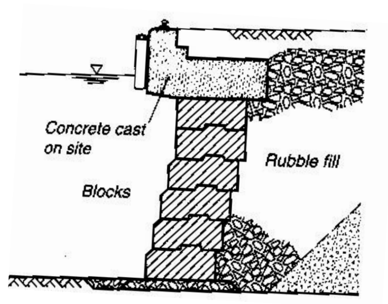


Figure 2.3: Typical section of block type wall (Quay Wall Handbook, 2005)

2.2.3.1.2 L-Walls: They provide their stability by the weight of concrete structure and the weight of the earth that rests them. This construction method is used if the bearing capacity of the subsoil is not sufficient for a block wall or if the aim is to save on material costs of a block wall. Figure 2.4 shows typical section of L wall.

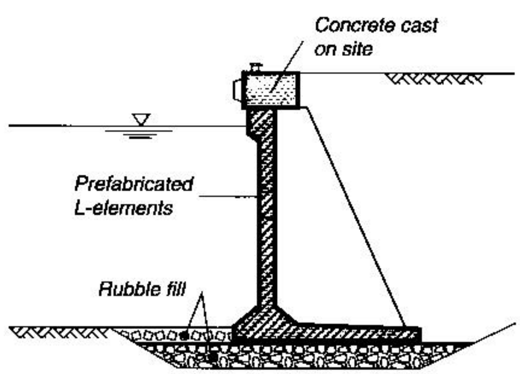


Figure 2.4: Typical section of L wall (Quay Wall Handbook, 2005)

2.2.3.1.3 Caisson Walls: Caissons are big cellular concrete elements. They are constructed in dock, on a floating pantoon or on a Synchro-lift. They are floated to the place where they are sunk onto the subsoil. This implicate that poor layers under the foundation must be removed and replaced by hardy material. Then, these prepared caissons are filled with the soil or other material in order to provide sufficient mass to resist the horizontal soil pressures. Figure 2.5 shows typical section of caisson wall.

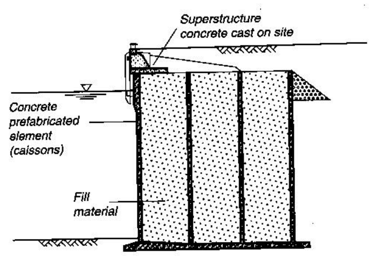


Figure 2.5: Typical section of caisson wall (Quay Wall Handbook, 2005)

2.2.3.1.4 Cellular Walls: The cells are constructed in water or on land and they are filled with sand or other material. The cellular wall consists of soil enclosed by steel rings in which only tensile stress occurs. Relatively little material is required but the walls are thin so they can damage when collisions occur. Figure 2.6 shows typical section of cellular wall.

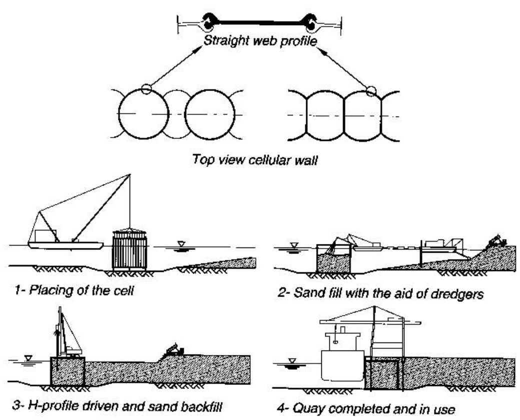


Figure 2.6: Typical section of cellular wall (Quay Wall Handbook, 2005)

2.2.3.1.5 Reinforced Earth Walls: Steel strips, steel rods and polymer reinforcements called as tension elements are inserted in to ground. The friction between the contact surfaces of tension elements and soil form the basic mechanism of stress transfer between the reinforcement and soil. Figure 2.7 shows typical section of reinforced type quay walls.

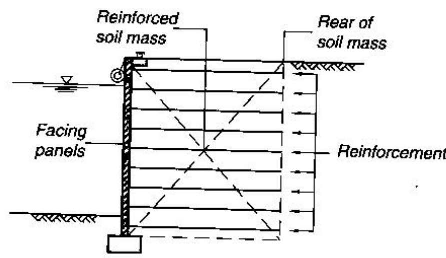


Figure 2.7: Typical section of reinforced earth wall (Quay Wall Handbook, 2005)

2.2.3.2a Embedded Walls

This type of structures derives their soil retaining function and stability from the fixation capacity of the soil, possibly in combinations with anchors. If the subsoil has poor bearing capacity, these walls are used. The sheet pile wall may be anchored. Rubber or wooden fenders are added to protect both quays and ship. Drainage is necessary to dispose of rainwater and to restrict the excess pore pressure behind the structure

2.2.3.2a.1 Cantilever Walls: If the sheet pile is not anchored, during the transfer of soil pressure to the subsoil, the sheet pile acts like a beam. Figure 2.8 shows typical section of cantilever wall.

2.2.3.2a.2 Anchored Walls: If the retaining height is long, it is necessary to anchor the upper side to bear the horizontal forces. Figure 2.9 shows typical section of anchored wall.

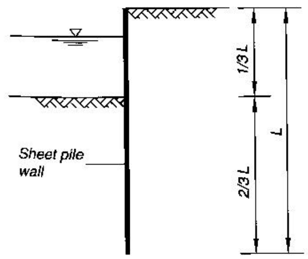


Figure 2.8: Typical section of cantilever wall (Quay Wall Handbook, 2005)

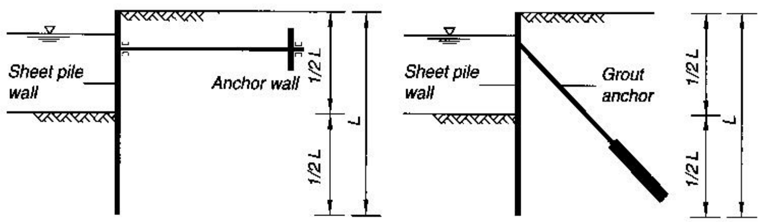


Figure 2.9: Typical section of anchored wall (Quay Wall Handbook, 2005)

2.2.3.2b Embedded Walls (with relieving platform)

The horizontal load on the front wall is reduced by the presence of a relieving platform. This platform can be installed at various heights. Figure 2.10 shows typical section of embedded wall.

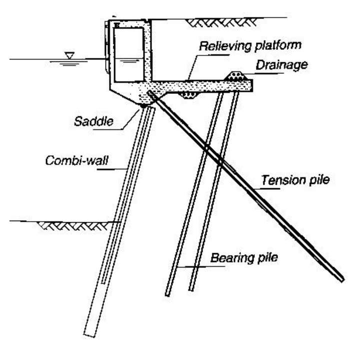


Figure 2.10: Typical section of embedded wall (Quay Wall Handbook, 2005)

2.2.3.3 Open Berth Quays

In this type of structures height is not bridged by a vertical wall, but a slope. When choosing the construction method and materials, it should be taken into consideration that the under side of the deck and the slope are difficult to access for maintenance.

2.2.3.3.1 Open Berth Quay Walls (with Retaining Wall): To limit the width of the super structure a vertical sheet pile wall can be constructed on the upper part of the slope. Figure 2.11 shows typical section of open berth quay wall with retaining wall.

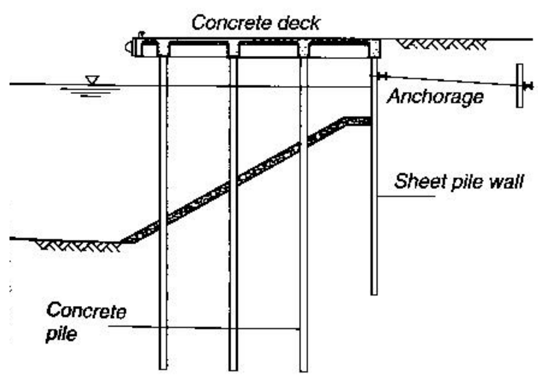


Figure 2.11: Typical section of open berth quay wall with retaining wall (Quay Wall Handbook, 2005)

2.2.3.3.2 Open Berth Quay Walls (with Embankment): They are built over a slope. The vertical forces are taken up by vertical piles and the horizontal forces are taken up by a pile trestle. Figure 2.12 shows typical section of open berth quay wall with embankment.

2.2.4 Failure Types of Gravity Quay Walls

Gravity quay walls are mostly used in Turkey and the general failure modes for gravity quay walls are shown in Figure 2.14 and Figure 2.15.

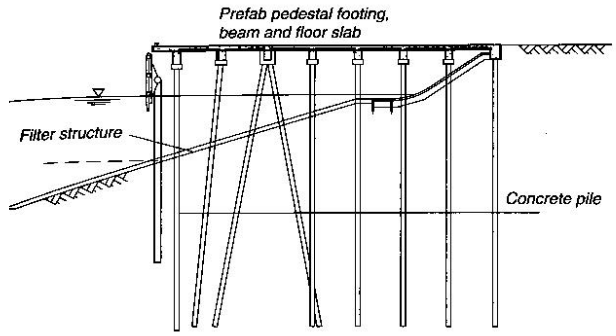


Figure 2.12: Typical section of open berth quay wall with embankment (Quay Wall Handbook, 2005)

For a gravity quay wall constructed on a firm foundation, an increase in earth pressure from the backfill plus the effect of an inertia force on the body of the wall result in the seaward movement of the wall as shown in Figure 2.13. If the width to height ratio of the wall is small, tilt may also be involved.

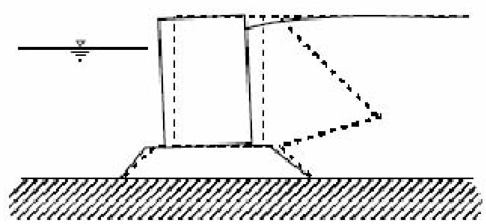


Figure 2.13: Deformation/failure modes of gravity quay wall on firm foundation (PIANC, 2001)

When the subsoil below the gravity wall is loose and excess pore water pressure increases in the subsoil, however, the movement of the wall is associated with significant deformation in the foundation soil, resulting in a large seaward movement involving tilt and settlement as shown in Figure 2.14.

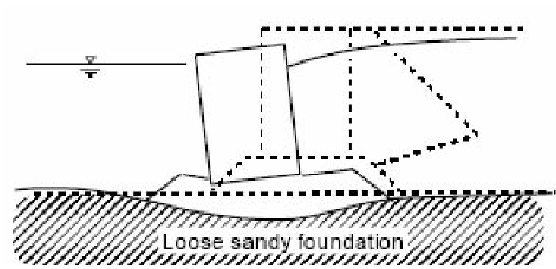


Figure 2.14: Deformation/failure modes of gravity quay wall on loose sandy formation (PIANC, 2001)

In Turkey, generally block type quay walls are used, thus in this study seismic design of block type quay walls will be presented. Design methodologies will be discussed in detail in view Conventional Methods.

CHAPTER III

LITERATURE SURVEY

Due to the unique physical infrastructure, long range planning process, complex operational structure, ports have seismic risk issues not encountered in other types of facilities and infrastructure systems. Thus, several methods have been developed and implemented in design practice of ports in many regions, often in the form of codes and standards. These codes and standards differ for each country and type of structure. In this thesis the *difference between design guidelines for block type quay walls used in Turkey compared to find an overview of design method.*

In order to define the relationship between the block type quay walls and the earthquake considerable researches have been conducted related retaining wall problems. These researches can be divided in two categories, *theoretical studies* and *experimental studies*. Analytical and numerical methods compose the theoretical studies. Shaking table tests performed under earth's gravity (1g), seismic centrifuge tests, and any rare occurrences of field testing compose experimental studies.

3.1. Theoretical Studies

Kumbasar and Kip (1999) solve a retaining wall by taking into consideration TSS, 1997. To find an overview seismic design *Ryul et al. (2004)* claim that magnitudes of force components acting on quay walls during earthquakes and the phase relationships among these force components should be defined properly. OCDI (2002) gives brief explanations about block type quay walls and the forces acting on it. While estimating the factor of safety values against the sliding and the overturning values offered by OCDI (2002) are recommended in this thesis. Fang et al. (1997) indicate that wall movement required for the backfill to reach an active state increases with increasing backfill inclination. The experimental active earth pressure value (K_A) is good agreement with the values determined with Coulomb and Terzaghi's theories, Rankine's solution tends to overestimate the active thrust, especially if the backfill with a negative sloping angle. *Green et al. (2003)* use Mononobe-Okabe Analysis for determining the lateral earth pressures of the wall if the earthquake is considered. *Caltabiano et al. (2000)* use the pseudo-static methods for the computation of soil thrust acting on retaining walls under seismic condition. The effect of the intensity of the surcharge and of its distance from the wall is investigated and the results are compared for the cases of soil-wall systems without surcharge. *Motta (1994)* work Coulomb active-earth pressure for distanced surcharge. A closed-form solution has been given for the evaluation of the active earth-pressure coefficient which takes into account the effects of both the soil weight and the surcharge applied to a certain distance from the head of the wall. Seismic effects have also been taken into account in a pseudostatic way by means of horizontal and vertical-seismic coefficients. The basic principles of Coulomb and Mononobe-Okabe Theorem are mentioned briefly in Geotechnical Earthquake Engineering book, *Kramer (1996)*. *Chen and Huang (2001)* use the finite-difference method in order to calculate the earthquake-induced hydrodynamic pressures of seawater and the pore water in seabed sediment. By using the different kinds of sediment depths and porosity, the dynamic response of a rigid coastal structure is investigated. At the end of this study it is proved that while designing the coastal structures in seismic zone, the hydrodynamic force and the seismic effects must be considered.

In the world, quay wall designs are given in different guidelines such as, *TUDelft (2006)* work about the different kinds of design guidelines and they pay attention to safety and design approach for quay wall structures. In 4 guidelines with a clear safety approach, also adapted to the latest design philosophies: CUR 166 and Handbook Quay Walls (both Dutch), EAU 2004 (German), Eurocode 7 (European Union). And two cases are considered. Similar to this thesis' object, *TUDelft (2006)* designs the quay walls comparing the different approaches given in the references.

3.2. Experimental Studies

Yang et al. (2001) makes series of centrifuge model tests and the nonlinear two-phase (solid-fluid) finite element program are used due to predict the seismic response of caisson-type quay walls and the liquefaction and deformation characteristic of the saturated cohesionless backfill. *Dewoolkar et al. (2000)* criticizes the static and seismic behavior of retaining walls with liquefiable backfills during earthquake. A series of centrifuge tests are conducted and accelerations on the walls and in the backfill, static and excess pore pressures in the soil are measured.

3.3. Performance-Based Design

In 1990s, in order to effectively mitigate disasters due to tsunamis, storm surges and high waves, coastal structures with high amenity and high disaster prevention effect has been tested and new conceptual design called as Performance Based Methodology has been developed. Thereby, even if the force balance exceeds the limit values, it can be possible to get some information about the performance of a structure.

Sasumu and Ichii (1998) study on the seismic performance based design for port structures. In this method, the required performance of a structure specified in terms of displacements and stress levels. This method contributes that the requirements of the seismic performance of a structure against the probabilistic occurrence of earthquake motions. *PIANC (2001)* published a book called as "Seismic Design Guidelines for Port Structures". This book address the limitations inherit in conventional design, and establish the framework for this approach. Seismic performance-based design of port structures and simulation techniques are discussed by *Sasumu (2003)*. Basic earthquake engineering knowledge and a strategy for seismic performance-based design is explained by using the figures and tables. The technical commentaries illustrate that specific aspects of seismic analysis and design, and provide examples of various applications of the guidelines. *Ahmed Ghobarah (2001)* is worked on state of development for performance-based design in earthquake engineering.

There are three analysis methods proposed in performance based design concept called as simplified analysis, simplified dynamic analysis and dynamic analysis. Simplified dynamic analysis method is studied by *Kim et al. (2005)* by taking into consideration the variation of wall thrust which is influenced by the excess pore pressure developed in backfill during earthquakes, the seismic sliding displacement of quay walls is estimated. Newmark sliding block concept is used for this method and by using the variable yield acceleration which varies according to the wall thrust, the quay wall displacement is calculated. The shaking table tests verify that the wall displacement can be predicted by using this method. *Mohajeri et al. (2004)* work on the sliding block concept for caisson walls. For this study, two series of shake table tests are used in order to evaluate the sliding displacement of caisson type quay walls and breakwaters due to earthquake motions. It is observed that the yield acceleration at the time that sliding tended to start and during the sliding this value is not a constant and after sliding, the yield acceleration decreases immediately. Thus, by the light of this result, it is advised to use two-level yield accelerations as static and dynamic levels. This result is important while combining the conventional method with the performance based method.

3.4. Example of Damaged Quay Walls

The numerical values about the damages of structures around ports due to West of Fukuoka Prefecture Earthquake are investigated by the *Hond and Kiyota (2005)*. Under the title of -location of Fukuoka and epicenter - information of earthquake the failure conditions for the quay walls and the piers were explained numerically by using several photos.

Tokachi-oki Earthquake of September 26 is studied by *Kobayashi et al. (2003)*. The brief explanation about the characteristic of the earthquake is given and it is investigated that, there is severe damage due to the tsunami impact as well as the structure, port facilities and lifeline damage due to strong shaking. The affect of this earthquake on the quay walls are given numerically according to region.

Andrus and Chung (1995) give some information about Kushiro Port in Japan which had shaken strongly in 1993. They states that many quay walls were damaged when liquefaction occurred in the fill materials behind the wall.

Tanaka (2000) states Kobe Port during the Great Hanshin Earthquake of January 17, 1995 had damaged, especially of the liquefaction of reclaimed fills, at two near-shore man-made islands, Port Island and Rokko Island. The paper first describes a general aspect of the earthquake event; second, the liquefaction damages at these two islands are discussed in the light of applicability of current evaluation procedures for soil liquefaction.

These studies are important because there are always seismic risks at ports which are very important. Several ports had been suffered from earthquake seriously and unfortunately they will injure also in future unless the necessary cautions are taken into consideration. Therefore, there is an intensive study currently going on the design of quay walls with special emphasis on performance based design (PIANC, 2001).

CHAPTER IV

DESIGN METHODOLOGIES of BLOCK TYPE QUAY WALLS

The methods used for the design of block type quays are;

- Conventional Design Method
    - based on Turkish Specification for Structures to be Built in Disaster Areas, 1975 (TSS, 1975)
    - based on Turkish Specification for Structures to be Built in Disaster Areas, 1997 (TSS, 1997)
    - **new design approaches**
    - based on Turkish Seismic Design codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007).
- Performance Based Design

In the conception of this thesis conventional design methods will be introduced and the applications will be carried on.

4.1. Conventional Design Methods

The conventional method is based on providing capacity to resist a design seismic force, but it does not provide information on the performance of a structure when the limit of the force-balance is exceeded. In conventional design for the relatively high intensity ground motions associated with a very rare seismic event, it is required that limit state not to be exceeded, if the construction cost will be too high. If force-balance design is based on a more frequent seismic event, then it is difficult to estimate the seismic performance of the structure when subjected to ground motions that are greater than those used in design (PIANC, 2001).

If the structure **collapses**, this can be seen as permanent failure. The state, in which a structure does not yet fails, is called a **limit state** (TUDelft, 2006).

Typical section of block type quay wall is given in Figure 4.1. Blocks are numbered up from foundation as 1 to n. After the placement of blocks, they are covered by reinforced concrete cap called as crown wall (Figure 4.1).

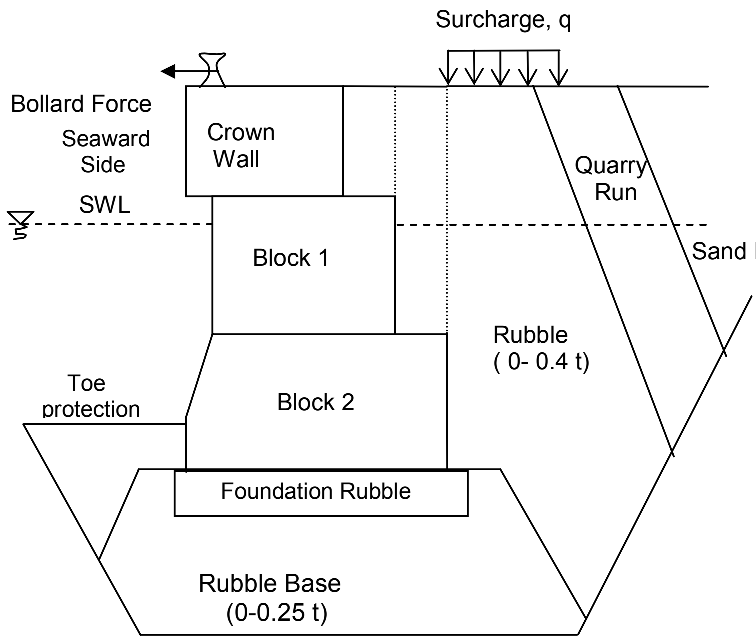


Figure 4.1: Typical section of block type quay wall

In the figure reference sea level is used as still water level (SWL) and backfill is composed of 3 different layer; rubble, quarry run and sand fill. Structure rests on foundation rubble and rubble base is protected by a rubble fill. Bollard force at crown wall and surcharge acting on concrete slab are also shown in Figure 4.1

4.1.1 Conventional Design Method Based on Turkish Specification for Structures to Be Built in Disaster Areas, 1975 (TSS, 1975)

In Turkey generally conventional design methodology given for seismic design based on TSS, 1975 is in use. Basically, seismic forces are defined as; (Figure 4.2)

$$F_{seismic} = C\ W \tag{4.1}$$

where; C is the seismic coefficient defined for each earthquake zone; W is the weight of the blocks.

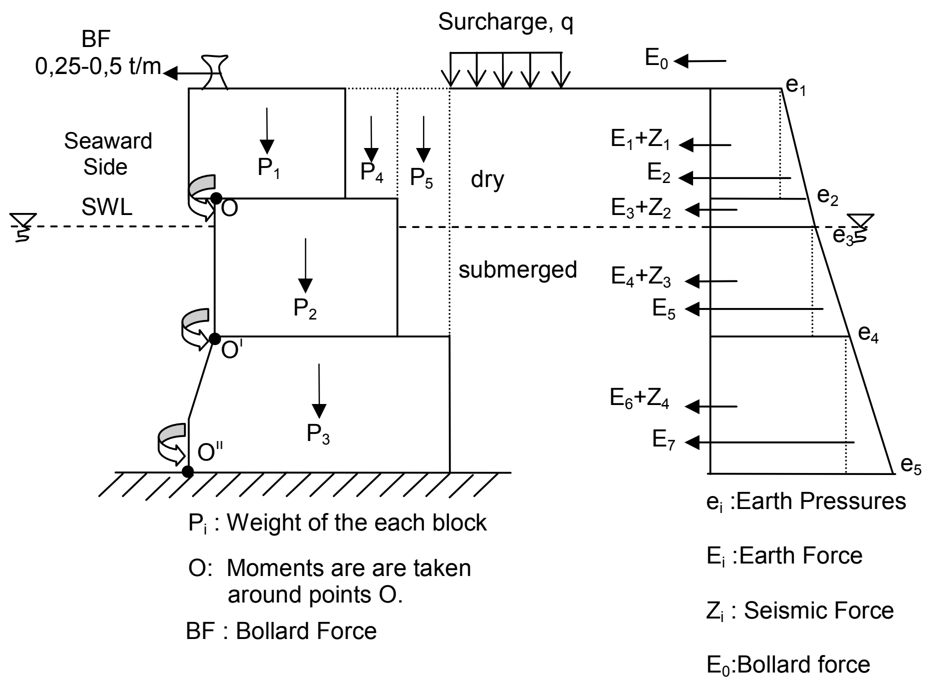


Figure 4.2: Loads both static and dynamic acting on block type quay walls according to TSS, 1975

This figure shows the loads affecting the block type quay walls during the earthquake. However, if the earthquake is not considered, the (Z_i) values shown in Figure 4.2 are ignored.

In conventional method the factor of safety against sliding (FS_s) and overturning (FS_o); for **normal (static) condition** are taken as; FS_s = 1.5 , FS_o = 1.5 ; for **seismic (dynamic) condition** are taken as; FS_s = 1.2 , FS_o = 1.5.

Detailed discussion of the forces (E_i and E_0) together with an application as an example cases are given in Appendix A.

4.1.1.1 Discussion on TSS, 1975

There are some difficulties and inaccuracy assumptions in seismic design of block type quay walls by using the conventional method based on TSS, 1975. These difficulties and inaccuracies will be explained briefly and the solutions which can be used in order to achieve these kinds of problems will be mentioned clearly.

In conventional method, for the **static case**, active earth pressure coefficient (K_A) is;

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2(\theta)\cos(\theta+\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right]^2} \tag{4.2}$$

and active earth pressure (P_A) is;

$$P_A = \frac{1}{2}K_A\gamma\, H^2 \tag{4.3}$$

(Eq.4.1) includes some parameters such as ϕ (internal friction angle), β (inclined angle with the horizontal), δ (friction angle between wall and soil), θ (angle between the back of the retaining wall and the vertical plane). While evaluating the P_A (active earth pressure) value, K_A is used, and P_A acts making the angle of δ with the wall. So, there must be a vertical and a horizontal component of P_A value. However, in seismic design of block type quay walls based on TSS, 1975 only horizontal component of P_A value is taken. The vertical component of P_A value is ignored.

For static case, the surcharge load (q) is multiplied with the horizontal active earth pressure coefficient (K_{Ah});

$$e_1 = (q \times K_{Ah}) \tag{4.4}$$

For the **dynamic case**, K_{Ahd} is used to define dynamic earth forces and surcharge forces. Both a_{max} and seismic coefficient (k_h) are disregarded in the computations. In order to calculate K_{Ahd}, the internal friction angle is reduced 6^o

($\phi' = \phi$-6°). This assumption can not be suitable for all conditions. In addition, this methodology assumes that the forces due to the earthquake motion act as a point load on a specific point for the each block. However, researches show that in real situation this assumption is not true, because earthquake force is distributed load on each block. Actually, the distribution of the earthquake forces' is more like "S" curve. However, since the actual distributions of the forces are not known exactly, it is not possible to define the forces acting on the blocks accurately enough.

Surcharge load is taken (q/2) and then (q/2) is multiplied with the horizontal active dynamic earth pressure coefficient (K_{Ahd}) value which is computed according to ($\phi' = \phi$-6°).

$$E_2 = (q \times K_{Ahd}) \qquad (4.5)$$

Both ground surface acceleration (a_{max}) and the seismic coefficient (k_h) are disregarded in the computations.

In conventional method there is only one surcharge pressure calculation. For static case, according to static condition surcharge pressures are computed and for dynamic case, according to dynamic condition surcharge pressures are computed. However, the surcharge forces which are calculated for static and dynamic cases to find the total force due to the surcharge load are not added to each other.

Moreover, during the seismic calculations, hydrodynamic forces are not taken into consideration in conventional method.

In order to make more economical design, recommended factor of safety values can be taken less than these values. Especially for seismic design the factor of safety values can be taken as FS_o=1.1 and FS_o=1.0 (OCDI, 2002).

Under the light of these given discussions, it is understood that TSS, 1975 has some deficiencies and it needs to be modified. Thus, TSS, 1997 was introduced as a modified method for the design of block type quay walls. Same approaches are used in computing the forces acting on the structure. However, distributions of the forces for both static and dynamic cases in TSS, 1997 display some dissimilarities with respect to TSS, 1975.

4.1.2 Conventional Design Method Based on TSS, 1997

In this method, for static case the distribution of surcharge and earth forces are shown in rectangular and triangular and for dynamic case the distribution of surcharge and earth forces are shown in inversed triangular and parabolic for vertical wall (Figure 4.3) (Kumbasar and Kip, 1999).

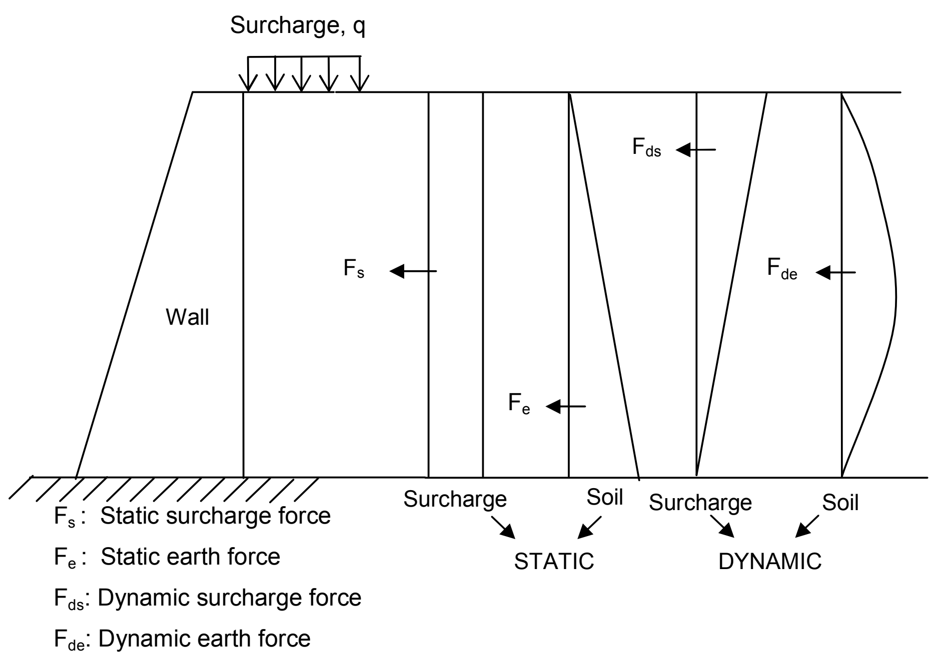


Figure 4.3: Forces both static and dynamic cases acting on a vertical wall

The forces acting on block type quay walls for static and dynamic case earthquake are shown in Figure 4.4 and Figure 4.5 respectively according to TSS, 1997.

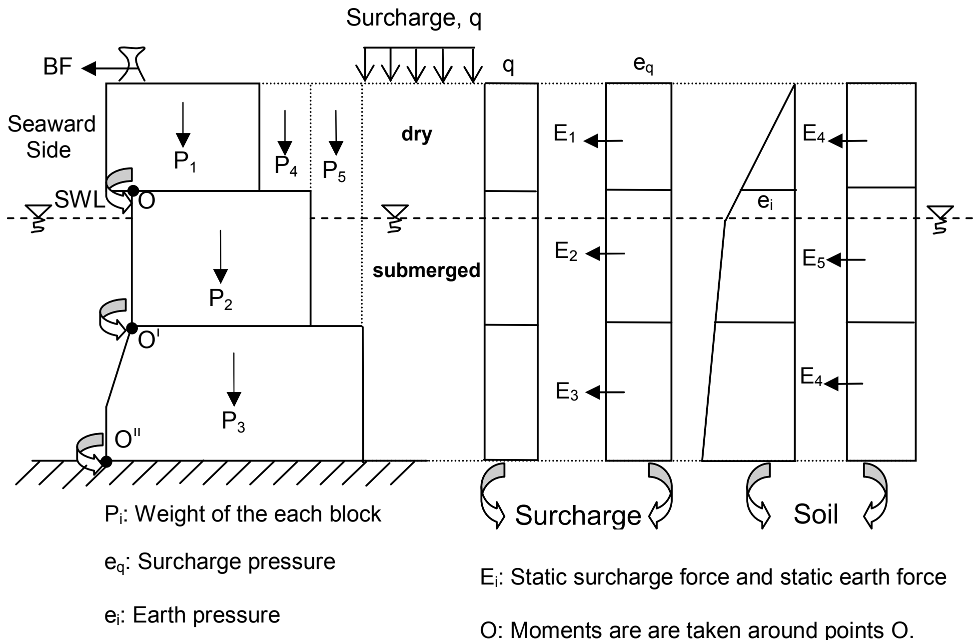


Figure 4.4: The static forces affecting on the block type quay walls (TSS, 1997)

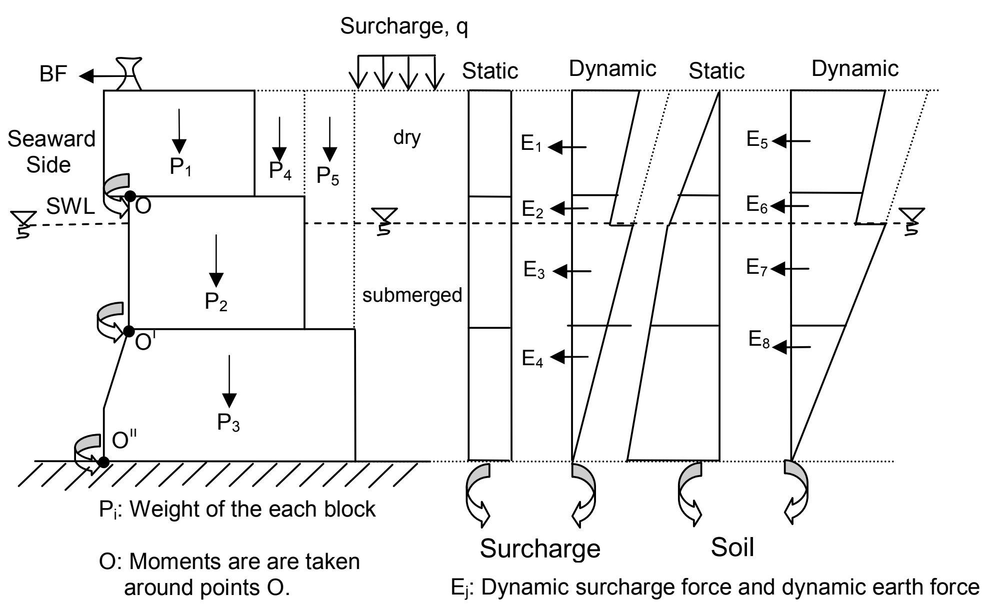


Figure 4.5: The dynamic forces affecting on the block type quay walls (TSS, 1997)

4.1.2.1 Discussion on TSS, 1997

Block type quay walls are designed by using the principles of TSS, 1997 and similar to conventional method, it is understood that this method also includes some difficulties and deficiencies.

In TSS, 1997, for the **static case*,*** the differences between the unit weight of the soil being either above or under still water level is taken into consideration. This approach can cause some problems during the seismic calculations which will explain below.

In TSS, 1997, for the **dynamic case**, two different seismic angle (ψ) values are computed and depending on these two values, two different dynamic active earth pressure coefficients (K_{AE}) are computed according to soil condition being either above (dry) or below (submerged) of still water level. However, this approach is not in agreement with the Mononobe-Okabe Method (M-O Method) principle. Because, M-O Method had been derived for dry soil. Thus, currently instead of this approach, use of an average unit weight which takes into consideration both dry and submerged soil is accepted to be consistent with the M-O Method.

In TSS, 1997 adopts that the upper wall is vulnerable for the **dynamic case** and to reinforce this assumption, inversed triangular distribution (base at the soil surface) is employed to show the dynamic force of the soil and the surcharge (Figure 4.5). In view of the latest researches, the assumption of inversed triangular distribution in order to designate dynamic force due to the soil can be a correct approach (EC 1110-2-6058, 2003). However, the Figure 4.3 shows parabolic distribution instead of inversed triangular distribution for dynamic active earth pressure. Thus, dynamic earth thrust should be zero both at the top and the bottom of the structure. It can be said that, there is a problem to define the distribution of the dynamic active earth pressure for block type quay walls.

In addition, for dynamic case hydrodynamic forces are also ignored in this method.

According to TSS, 1997, the factor of safety against sliding and overturning; for normal (static) condition are taken as; FS_s=1,5, FS_o= 1,5; for seismic (dynamic) condition are taken as; FS_s =1,0, FS_o= 1,0.

Based on these discussions carried on the TSS, 1975 and TSS, 1997 new design approaches to conventional method for block type quay wall are introduced and discussed in details.

4.1.3. New Seismic Design Approaches for Block Type Quay Walls

4.1.3.1. Methodology

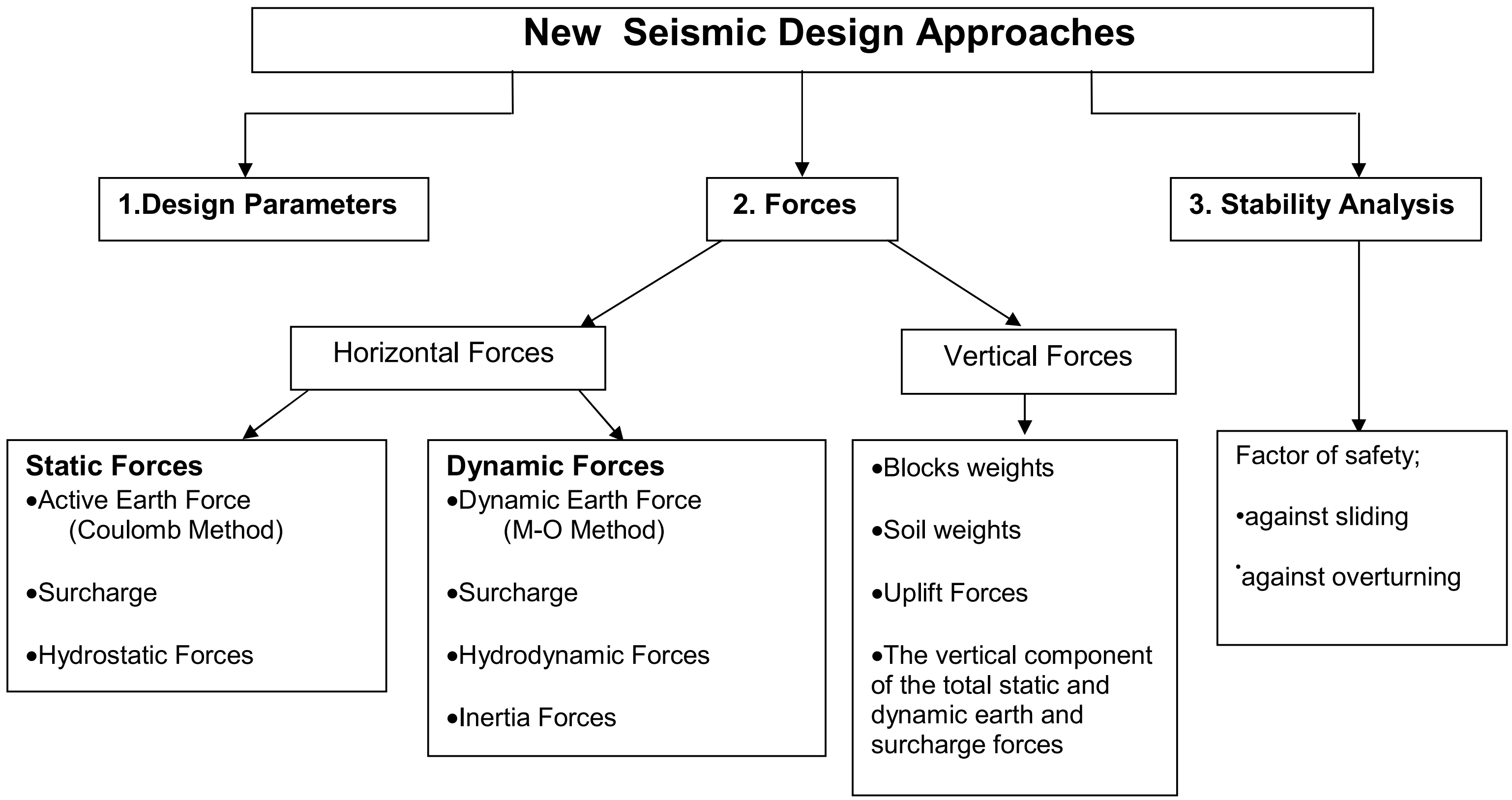
Conventional design methods usually require estimating the earth pressure behind a wall and choosing the wall geometry in order to satisfy equilibrium conditions with specified factors of safety (Caltabiano et al., 2001). In general, earth pressure acting on a vertical wall is assumed to be active earth pressure, and is determined taking into account the influence of surcharge on the ground surface behind the wall and the influence of hydraulic pressure (due to water level difference both two sides of the wall).

The researches have shown that the inertial forces of the quay wall, the dynamic earth and pore water pressures generated in the backfill, and the reduction of shear resistance at the interface between the quay wall and the foundation soils are the causes of wall movement during earthquakes (Lee, 2004).

In the development of new design approach of block type quay wall, forces should be defined correctly and to make it firstly design parameters should be defined both for static and dynamic conditions as in conventional methods. The results in the study are important to improve the analytical procedures to evaluate quay wall stability during earthquakes.

The general calculation steps for new design approaches are given in Table 4.1.

Table 4.1: Calculation steps of new seismic design approaches



The first step in the new design approaches for block type quay wall is to determine the characteristic of design parameters (Table 4.1);

Design Parameters

Design parameters are divided into three parts;

1.Geotechnical parameters

c : cohesion of soil (kN/m^2)

ϕ : internal friction angle of soil (deg)

γ : unit weight of soil (kN/m^3)

δ : wall friction angle (deg)

2.Geometric parameters

H : wall height (m)

β : inclined angle with the horizontal

θ : angle between the back of the wall and the vertical plane (deg)

SWL : still water level (m)

3.Load-related parameters

q : surcharge on the ground surface behind the wall (kN/m^2)

k_h : horizontal seismic coefficient

W: weight of the soil

and P_A is the active earth pressure

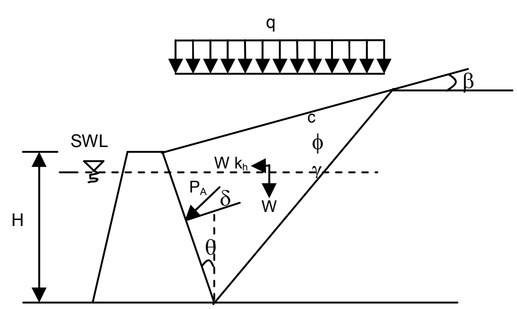


Figure 4.6: All the parameters used for calculating the forces

In order to define the parameters, basic assumptions are used.

4.1.3.2. Basic Assumptions

Basic assumptions used in the development stages are listed below.

- The Rankine and Coulomb methods are commonly used to calculate the active earth pressure force. The discussion in this study is limited to granular (cohesionless) backfill soil, which is a typical condition relating to retaining walls.

- The active earth pressure force (P_A) is a function of the earth pressure coefficient (K_A), the unit weight of the soil and the height of the wall.

- Wall movement must occur in order to develop the full active earth pressure force.

- Other lateral forces are superimposed on the lateral earth pressure force to derive the total lateral force.

- Vertical wall design is iterative and seeks to provide wall geometry that produces suitable factors of safety for sliding, overturning and bearing capacity.

- Vertical walls must also be checked for tolerable settlement and global stability (Richard P. Weber, 2000).

- It is assumed that soil improvement techniques are used for the site where the existing soil conditions are expected not to lead to unsatisfactory performance. Usually, large soil movements are accepted as unsatisfactory performance. If only static condition is absent, unacceptable movement generally results from poor soil strength and/or stiffness. However, during and/or after earthquake, this movement consist horizontal and/or vertical components. In addition, the build up of excess pore water pressure can lead to very large deformations. Consequently, all techniques used to mitigate of seismic hazard aim to reduce the aptitude of soil to generate excess pore water pressure during earthquake and to increase the strength and stiffness of soil. Increased strength and stiffness is generally desirable for both dynamic and static conditions. The most common soil improvement techniques can be classified into 4 parts: densification techniques, reinforcement techniques, grouting/ mixing techniques, and drainage techniques (Kramer, 1996). In general, port structures are designed to prevent the liquefaction in backfill. However, the parametric study showed that the wall may keep its stability even if the liquefaction of the backfill occurs, and the wall can be unstable when liquefaction does not occur in the backfill. Therefore, wall displacement must be evaluated with the consideration of the excess pore pressure development in the backfill in the design stage of quay walls (Kim et al., 2005).

After defining the design parameters according to basic assumptions, forces can be calculated.

4.1.3.3 Pseudostatic Analysis

The most common approach to seismic slope stability evaluation is the pseudostatic analysis, which is based on limit state methods. In the 1920s, Mononobe and Okabe developed a method to estimate the lateral earth pressures acting on retaining structures during earthquakes. The first explicit application of the pseudostatic approach to analysis of seismic slope stability has been attributed to Terzaghi (1950). Inertial forces, F_h and F_v, are assumed to act through the centered of the failure mass. The magnitudes of the pseudostatic forces are (Kramer,1996);

$$F_h = \frac{a_h W}{g} = k_h W \tag{4.6}$$

$$F_v = \frac{a_v W}{g} = k_v W \tag{4.7}$$

where, F_h and F_v are horizontal and vertical pseudostatic forces; a_h and a_v are horizontal and vertical pseudostatic accelerations; k_h and k_v are dimensionless horizontal and vertical pseudostatic coefficients; W is weight of the failure mass.

Similar to static limit equilibrium design methods, pseudostatic analyses provide a factor of safety against failure, but no information regarding permanent slope deformations.

The vertical pseudostatic force has less effect on the factor of safety, it can reduce or increase (depending the direction) both the driving force and resisting force. Thus, the influence of vertical acceleration is ignored in pseudostatic analysis (Kramer, 1996).

Coulomb (1776) first proposed the failure-wedge method by assuming a plane-failure surface and imposing the equilibrium conditions. Based on the Coulomb-wedge method, MueUer Breslau (1906) derived a closed-form solution for the active- earth-pressure coefficient taking into account the slope of the ground profile and the friction between the wall and the soil. Successively, Mononobe (1929) and Okabe (1926) extended the Coulomb solution taking also into account the earthquake-induced pressure in a pseudostatic way (Motta, 1994).

4.1.3.4 Forces Acting on Block Type Quay Wall

The second step is forces, and forces are divided into two parts as horizontal and vertical forces. Static and dynamic forces constitute horizontal forces. Block weights, soil weights and the vertical component of the total static and dynamic earth and surcharge forces (Table 4.1)

4.1.3.4.1. Static Horizontal Forces

The static forces are classified into three parts; active earth forces, surcharge forces and hydrostatic forces.

4.1.3.4.1.1. Active Earth Force

Active earth pressure coefficient (K_A) is computed using the Coulomb Theory.

Coulomb Theory

The Coulomb approach is based on the limit-equilibrium method (Motta, 1994) and by using this equilibrium, the magnitude of the soil thrust acting on the wall for both minimum active and maximum passive conditions (Kramer, 1996). As the wall starts to move, the earth pressure decrease rapidly and eventually a limiting active pressure is reached (Fang et al., 1997). In this thesis, the passive condition is neglected and only active condition is studied. For the same wall the passive wall displacement required to reach a passive state is approximately 230 times the displacement required to reach an active state (Fang et al., 1997).

Using Coulomb expression, K_A and P_A are defined as (Figure 4.7);

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2(\theta)\cos(\theta+\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right]^2} \tag{4.8}$$

$$P_A = \frac{1}{2}K_A\gamma\, H^2 \tag{4.9}$$

where; K_A is active earth pressure coefficient, P_A is active earth pressure, θ is angle between the back of the retaining wall and the vertical plane , δ is friction angle between wall and soil, β inclined angle with the horizontal, ϕ internal friction angle, H is structure height , α_A is inclination of the ground surface behind the wall (Figure 4.7).

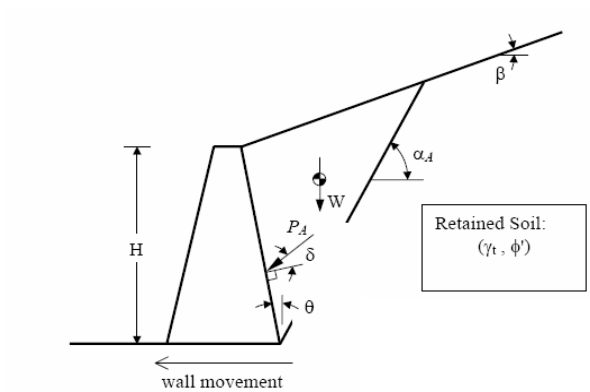


Figure 4.7: The parameters used to calculate active earth pressure (P_A)

K_A formulation includes unknown parameters. These unknown parameters are related to property of soil and relation between the soil and blocks. For example, ϕ value, it can be observed that the displacements calculated by the proposed models are very sensitive to the interface friction angle. Therefore, it is important to properly evaluate the frictional resistance between a wall and foundation (Kim et al., 2005).

After the soil improvement techniques are used and the unknown parameters are defined, active earth pressure (P_A) value can be calculated for each block with respect to their heights. These pressure accounts are managed to find these forces for each block and triangular distribution is used throughout the block type quay wall to exhibit the effect of these forces (Figure 4.8). The necessary formulations are given below;

$$P_{i(se)} = K_A \ \gamma \ h_i \tag{4.10}$$

$$F_{i\,(se)} = \left(\frac{P_{i(se)} + P_{i-1}}{2} \right) h_i \tag{4.11}$$

where, $P_{i(se)}$ is the static earth pressure for block i, $F_{i(se)}$ is the static earth force for block i, and h_i is the height for block i, γ is the unit weight of soil (kN/m^3).

- During the dynamic calculation, it is realized that equivalent unit weight should be used owing to the M-O principals. Thus, unit weight of soil (γ) is taken as equivalent unit weight (γ_{eq}).

4.1.3.4.1.2. Surcharge

Given a soil-wall system without surcharge, it is possible to determine the system failure wedge. In many practical problems however, it may be of interest to evaluate the lateral earth thrust due to the soil weight as well as to a surcharge acting on retained backfill. If a surcharge is placed on this failure wedge, independently of its intensity, it will affect the failure mechanism. This implies that the system will collapse for a lower seismic acceleration and with a larger inclination of the failure wedge than the case of the system without surcharge. Available solutions for active earth pressure acting on walls retaining surcharged backfill are suitable for static conditions only (Caltabiano, 2000). In this study the surcharge is applied to a certain distance from the top of the structure and is uniformly distributed.

The dispersal influence of surcharge for static condition is adopted like a rectangular shape throughout the block type quay wall (TSS, 1997 and Kumbasar and Kip, 1999) (Figure 4.8). Thus, static surcharge pressures and static surcharge forces are calculated for each block by;

$$P_{i(ss)} = q\ K_A \tag{4.12}$$

$$F_{i(ss)} = P_{i(ss)}\ h_i \tag{4.13}$$

where, $P_{i(ss)}$ is the static surcharge pressure, $F_{i(ss)}$ is the static surcharge force and h_i is the height for each the block.

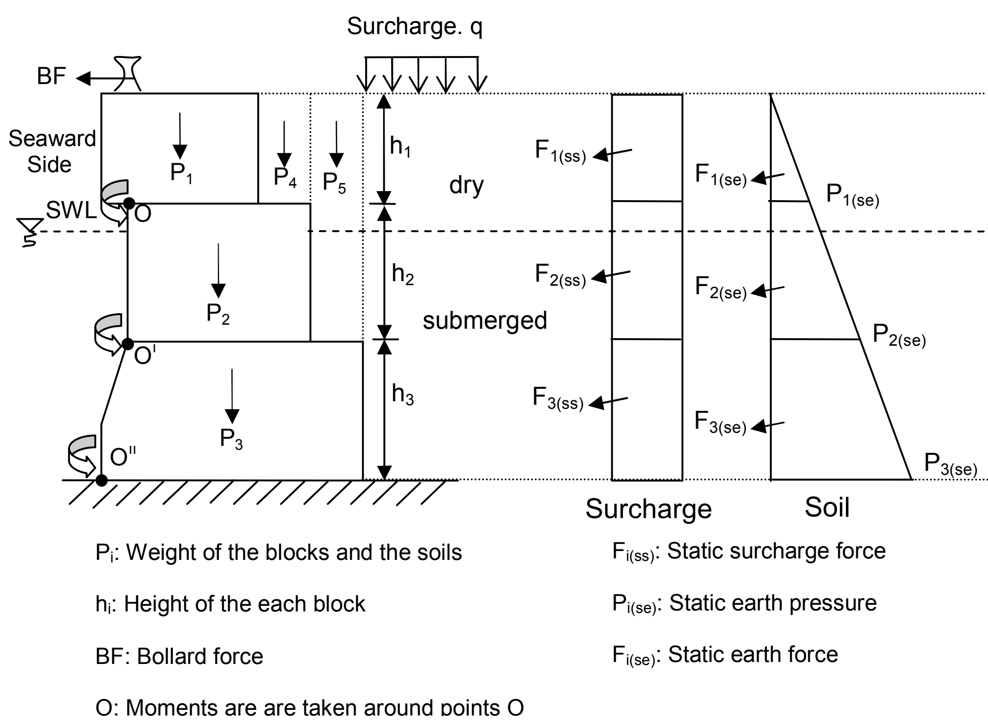


Figure 4.8: Forces acting on block type quay wall in static condition

4.1.3.4.1.3 Hydrostatic Forces

It is assumed that the water levels on both sides of the wall were the same and thus, the static water forces acting on the both sides of the wall and the hydrostatic pressure due to the variation of water table are not considered.

4.1.3.4.2 Dynamic Forces

Dynamic forces are classified into four parts; dynamic active earth forces, dynamic surcharge forces, hydrodynamic forces and inertia forces.

4.1.3.4.2.1 Dynamic Active Earth Forces

During an earthquake, dynamic forces in addition to static forces act on block type quay wall such as: the dynamic earth forces, dynamic surcharge forces, inertia forces and the dynamic water forces (Kim et al., 2004). In order to define these forces dynamic active earth pressure coefficient (K_{AE}) is computed using the Mononobe-Okabe Method (M-O Method).

Mononobe-Okabe Method (M-O Method)

The M-O Method (Okabe, 1926; Mononobe and Matsuo, 1929) is a commonly used form of *pseudostatic analysis* used to determine the earth pressures on retaining structures and is an extension of the Coulomb theory for static stress conditions.

Seed and Whitman (1970) in Green et al. (2003) indicates the method entails three fundamental assumptions:

1. Wall movement is sufficient to ensure either active or passive conditions

2. The driving soil wedge inducing the lateral earth pressures is formed by a planar failure surface starting at the heel of the wall and extending to the free surface of the backfill. Along this failure plane the maximum shear strength of the backfill is mobilized.

3. The driving soil wedge and the retaining structure act as rigid bodies and therefore experience uniform accelerations throughout the respective bodies.

Using M-O expression K_{AE} and P_{AE} are defined as;

$$K_{AE}=\frac{(1-k_v)\cos^2(\phi-\theta-\psi)}{\cos\psi\cos^2(\theta)\cos(\theta+\delta+\psi)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta-\psi)}{\cos(\delta+\theta+\psi)\cos(\beta-\theta)}}\right]^2} \tag{4.14}$$

$$P_{AE}=\frac{1}{2}K_{AE}\gamma\, H^2(1-k_v) \tag{4.15}$$

K_{AE} is the dynamic active earth pressure coefficient, P_{AE} is the total active thrust (kN/m), γ is the unit weight of soil (kN/m^3), ϕ is the internal friction angle of soil (deg), θ is the angle between the back of the retaining wall and the vertical plane (deg), ψ is the seismic inertia angle, δ is the friction angle between wall and soil, k_v is the vertical seismic coefficient and H is the height of the structure.

Several laboratory 1g (acc.) shaking table tests on retaining wall models have been performed in the past. Some of the early tests suffered from inadequate instrumentation unrealistic frequencies and amplitudes of input vibrations and lack of plane strain conditions which are assumed in most analytical and design methods. These tests generally indicated that the M-O method gives the magnitude of the total resultant force reasonably well and the incremental dynamic earth pressure acts at somewhere between 0.45 and 0.55 H from the base depending on the wall movement, where H is the wall height (Dewoolkar et al., 2000). However, at the larger levels of shaking, the M-O expressions failed to predict the induced stresses on the wall (Green et al., 2003).

As it is seen from the Eq.(4.12), K_{AE} formulation includes two unknown parameters (the other unknown parameters are explained in part 4.1.3.4.1.1**)**. One of them is seismic inertia angle (ψ) the other one is vertical seismic coefficient (k_v). To define (ψ), horizontal seismic coefficient (k_h) and peak ground acceleration (PGA=a_{max}) should also be defined. Peak ground horizontal acceleration PGH_H or simply PGA (a_{max}) is the maximum absolute value reached by ground horizontal acceleration during the earthquake. It is also called peak acceleration or maximum acceleration.

- **Peak ground acceleration PGA (a_{max});**

TSDC-CRA, 2007 approach is used to define these unknown parameters. This approach assumes that spectral acceleration value for T=0.2 sec (S_s) and spectral acceleration value for T=1.0 sec (S_1) can be evaluated by using the coordinate of the region where the seismic calculations will be made (Erdik et al., 2006). Spectral acceleration (S_s) is taken as the basic parameter. Then, (S_s) and short period soil coefficient (F_a) are multiplied to compute the spectral acceleration value with respect to soil classification (S_{MS}) Eq.(4.16). F_a values are obtained from Table 4.1.

$$S_{MS} = F_a \times S_s \tag{4.16}$$

Table 4.2: Spectral acceleration (g)[a] for T=0.2 sec

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]

Look Ek 4

[a] Interpolation is made to find S_s.

[b] Special geotechnical analysis and dynamic soil attitude analysis is made.

a_{max}/g is called as effective seismic coefficient (A_{10} or A_{20}) according to probability of exceedance %50 (L1 earthquake motion) and probability of exceedance %10 (L2 earthquake motion). A_{10} and A_{20} can be calculated by using the Eq.(4.16) and Eq.(4.17).

$$A_{10} = 0,4 \times S_{MS} \quad \text{for L1} \quad \text{(probability of exceedance \%50)} \tag{4.17}$$

$$A_{20} = 0,4 \times S_{MS} \quad \text{for L2} \quad \text{(probability of exceedance \%10)} \tag{4.18}$$

where, A_{10} is effective seismic coefficient for L1, A_{20} is effective seismic coefficient for L2.

- **Horizontal seismic coefficient (k_h)**

After defining these parameters the horizontal seismic coefficient (k_h) can be calculated according to earthquake motions, k_h is;

$$\text{for L1} \Rightarrow \quad k_h = (2/3)\, A_{10} \tag{4.19}$$

$$\text{for L2} \Rightarrow \quad k_h = A_{20} \qquad (A_{20} \leq 0.20) \tag{4.20}$$

$$k_h = (1/3)\, A_{20}^{(1/3)} \qquad (A_{20} > 0.20) \tag{4.21}$$

- **Vertical seismic coefficient (k_v)**

A parametric study, using an earthquake motion recorded at Kobe Port Island site (PGA_H =55.44 m/s^2, PGA_V =2 m/s^2) shows that the vertical component either increases or decreases the displacement of the blocks less than 10%, thereby the influence of the vertical component is not primary importance. Thus, it is assumed that k_v= 0 (Mohajeri et al., 2004). In addition, Kramer (1996) also states that the influence of vertical acceleration is ignored in pseudostatic analysis.

- **Equivalent unit weight (γ_{eq})**

Unit weight of soil (γ) takes different values according to soil condition being either above (dry) or below (submerged) of still water level. However, there is a problem during the calculation of the total active thrust for submerged soil, since the M-O Method had been derived for dry soil. Thus, during the calculations it can be more convenient to use equivalent unit weight (Kramer, 1996 and PIANC, 2001). Furthermore, using different unit weight values for soil induces some complexity during the calculations. Therefore, it is supposed that the equivalent unit weight can be calculated by using Eq.(4.22) and Eq.(4.23) (Kramer, 1996). In this way by defining only equivalent unit weight for both dry and submerged soil condition, the unknown parameters can be calculated easily and this assumption becomes more compatible with the M-O Method principle.

Calculation method of equivalent unit weight (γ_{eq}) is shown in (Figure 4.9) (Kramer, 1996).

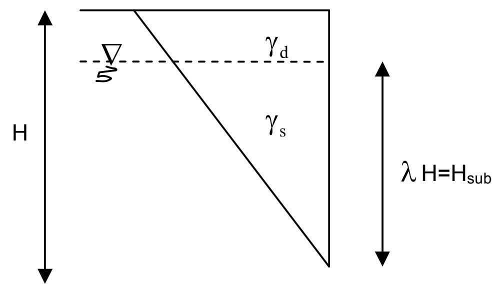


Figure 4.9: Calculation method of equivalent unit weight (Kramer, 1996)

$$\lambda H = H_{sub} \tag{4.22}$$

$$\gamma_{eq} = \lambda^2 \gamma_s + (1-\lambda^2)\lambda_d \tag{4.23}$$

where; H is the height of the structure, H_{sub} is the submerged soil height, λ is coefficient, γ_s is the submerged unit weight of soil (kN/m^3) and γ_d is the dry unit weight of soil (kN/m^3).

- Also by using PIANC, 2001; γ_{eq} can be calculated.

$$\gamma_{eq} = \gamma_{wet}\left[1-\left(\frac{H_{sub}}{H}\right)^2\right] + \gamma_b\left(\frac{H_{sub}}{H}\right)^2 \tag{4.24}$$

- **Seismic inertia angle (ψ)**

Both k_h and k_v are obtained then seismic inertia angle (ψ), (PIANC. 2001) is;

$$\psi = \arctan\left(\frac{k_h}{1-k_v}\right) \tag{4.25}$$

Amano et al. (1956) states that if the soil is dry seismic inertia angle (ψ) is calculated using Eq.(4.25). The M-O equation had been derived for yielding backfill retained by a wall. When the backfill is saturated with water it is common to practice adopt the assumption that pore water moves with the soil grains. For fully saturated Coulomb wedge, the horizontal inertia force is proportional to saturated total unit weight, γ_{sat}, and the vertical force is proportional to buoyant unit weight ($\gamma_b = \gamma_{sat} - \gamma_w$). Thus the modified seismic coefficient (k'_h) is given by (PIANC, 2001).

$$k'_h = \frac{\gamma_{sat}}{\gamma_w} \tag{4.26}$$

For partially submerged soil wedge the modified seismic coefficient (k'_h) can be obtained by modifying Eq.(4.26);

$$k'_h = \frac{\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^{N}(\gamma_{sj} h_j) + q_o}{\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^{N}(\gamma_{bj} h_j) + q_o} k_h \tag{4.27}$$

where; γ_j is dry unit weight of soil for soil class j^{th}, h_j is thickness of the soil class j^{th}, ND is number of the dry soil class according to ground level, q_o is surcharge load, γ_{bj} is submerged unit weight of soil for soil class j^{th}, γ_{sj} is saturated unit weight of soil for soil class j^{th}.

The seismic inertia angle (ψ) for submerged soil is;

$$\psi = \arctan\left(k'_h\right) \tag{4.28}$$

After all the parameters are defined the dynamic active earth pressure coefficient (K_{AE}) is calculated, then the total active thrust (P_{AE}) can be obtained by Eq.(4.15).

- **Dynamic Component (ΔP_{AE})**

In M-O Method the total thrust (P_{AE}) can be divided into two parts; static component (P_A) and dynamic computed (ΔP_{AE}) using Eq.(4.29).

$$P_{AE}= \Delta P_{AE} + P_A \tag{4.29}$$

It is clear that, in order to find the dynamic component (ΔP_{AE}); static component (P_A) is subtracted from the total thrust (P_{AE})(Eq.(4.30)).

$$\Delta P_{AE} = P_{AE} - P_A \tag{4.30}$$

which will enable to distribute to dynamic component.

Although determination of the magnitude and distribution of seismic earth pressures is very important in seismic design of retaining walls, most studies did not attempt to measure lateral earth pressures on model walls (Dewoolkar et al., 2000). Kramer (1996) in Seed and Whitman (1970) recommended that the dynamic component be taken to act at approximately at 0.6 H from the bottom to the surface. And Dewoolkar et. al. (2000) states that dynamic component be taken to act at 0.45 and 0.55 H from the base depending on the wall movement, where H is the wall height. However, this assumption also gives a fix approach point like TSS 1975. It is necessary to distribute the dynamic component of the total thrust along the block type quay wall and during distributing dynamic component of the total thrust; it should be pay attention to upper blocks stability. Already, it is accepted that upper blocks are vulnerable during the earthquake and to provide this assumption, inversed triangular distribution is employed to show the distribution of the dynamic component of the total thrust (Figure 4.10). Then, dynamic component (ΔP_{AE}) is split up with proportional to the backfill area acting on each block. It is assumed that the failure surface makes an angle of $(\alpha_{ae} = 45 + \phi/2)$ with the horizontal plane (Figure 4.10). Also PIANC (2001) recommended that by using the Eq.(4.31), the active angle of failure can be computed (PIANC, 2001).

$$\alpha_{ae} = \phi + \arctan\left[\frac{-\tan\phi + \sqrt{\tan\phi(\tan\phi + \cot\phi)(1+\tan\delta\cot\phi)}}{1+\tan\delta(\tan\phi + \cot\phi)}\right] \tag{4.31}$$

Both active angle of failure formulations are evaluated and compared. Then, it is decided to use Eq.(4.31) the areas of inversed triangular' acting on the each block is computed by (Figure 4.10);

$$A_1 = \left(\frac{b+a}{2}\right)(h_3 - h_2) \quad A_2 = \left(\frac{b+c}{2}\right)(h_2 - h_1). \quad A_3 = \frac{ch_1}{2}$$

$$\sum A = A_1 + A_2 + A_3 \tag{4.32}$$

$$\frac{\Delta P_{AE}}{\sum Area} = c_1 \text{ (force coefficient)} \tag{4.33}$$

This coefficient is multiplied for each area value. A_1, A_2, A_3; thereby, the dynamic component is distributed uniformly on the block wall (Figure 4.10).

$$F_{i(de)} = (c_1)\,(A_i) \tag{4.34}$$

4.1.3.4.2.2. Surcharge

As it is mentioned before, in the pseudo-static approach, the earth pressures are usually estimated using the M-O equation for completely dry soil (Mononobe, 1924; Okabe, 1924). In case of a uniformly distributed surcharge (q_{sur}), γ_d should be substituted with $(\gamma_d + (q_{sur}/H))$. Furthermore, the surcharge (q_{sur}) used for seismic design is typically half of the surcharge used for static stability design (PIANC, 2001).

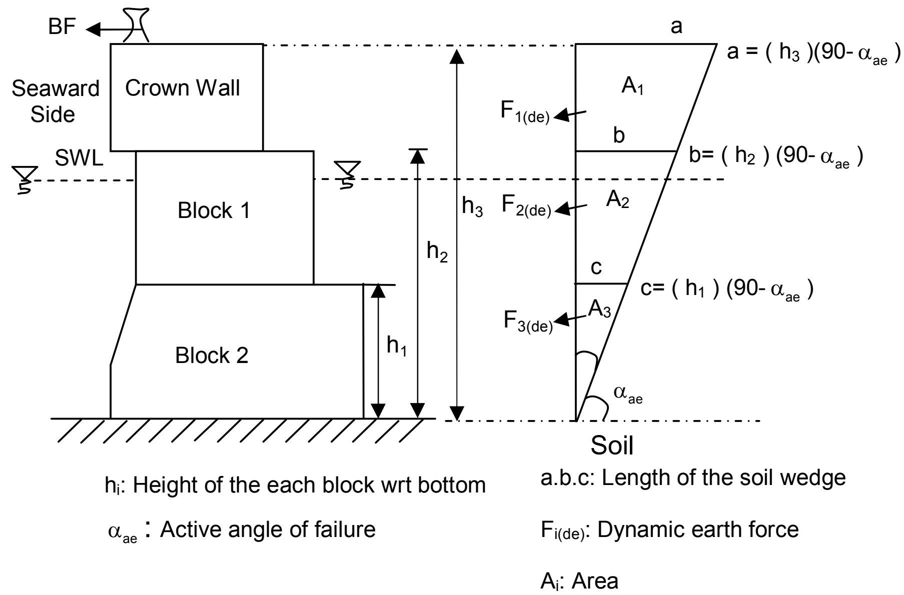


Figure 4.10: For dynamic case effect of soil on block type quay wall

Thus, it is decided that, in order to define surcharge force influencing on the block type quay wall approximately same pressure distribution assumption which is defined for the dynamic component of total thrust can be used. In other words, the dispersal of the surcharge load on block type quay wall can be guessed as also inversed triangular (Figure 4.12). And, the areas of inversed triangular' acting on each block is calculated beforehand to define the dynamic component of total thrust. These areas are also employed to find the forces for each block due to the surcharge load for dynamic case.

For the seismic design, in first step $q_{sur}/2$ is computed this computed surcharge load value is converted into weight in other words; surcharge load is multiplied with the affected surface length (a) and unit width (Figure 4.11). As it is known that, weight is a kind of vertical force and in order to denote this force as a horizontal force, weight should be multiplied with the horizontal seismic coefficient (k_h) or modified seismic coefficient ($k_h^{'}$) according to soil condition being submerged or dry (Eq.4.19 or Eq.4.27). In this way, the surcharge load is represented as a horizontal force and dividing this force to total area, coefficient (c_2) is gained.

$$W = (q/2)\,(a) \tag{4.35}$$

$$\frac{(W)(k_h)}{\sum \text{Area}} = c_2 \quad \text{or} \quad \frac{(W)(k'_h)}{\sum \text{Area}} \tag{4.36}$$

$$A_1 = \left(\frac{b+a}{2}\right)(h_3 - h_2) \quad A_2 = \left(\frac{b+c}{2}\right)(h_2 - h_1). \quad A_3 = \frac{ch_1}{2}$$

$$\sum A = A_1 + A_2 + A_3 \tag{4.37}$$

This coefficient (c_2) is multiplied for each area value. A_1, A_2, A_3; thereby, the surcharge is distributed uniformly according to area (Figure 4.11).

$$F_{i(ds)} = (c_2)\,(A_i) \tag{4.38}$$

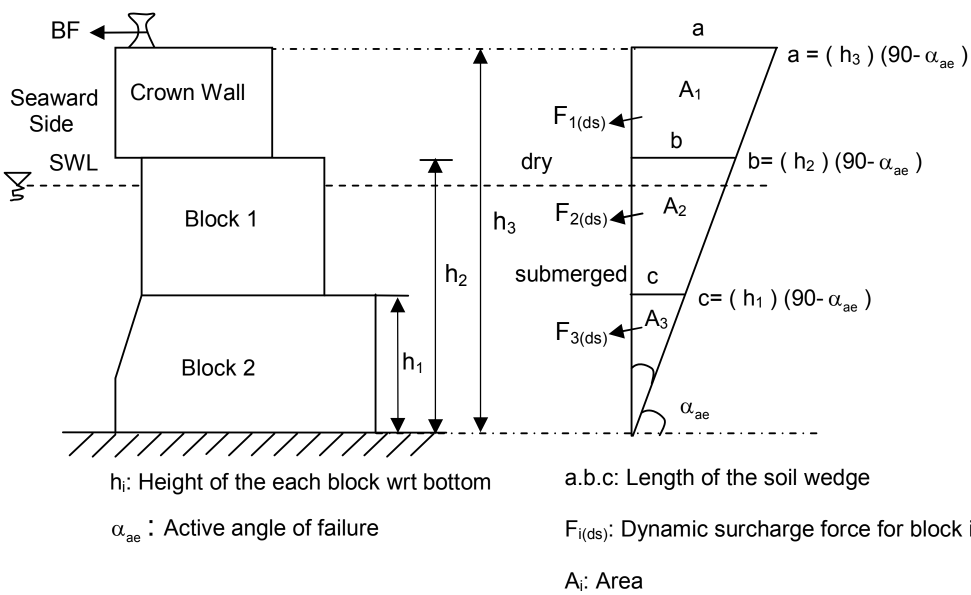


Figure 4.11: The dynamic effect of surcharge on block type quay wall

In addition to this point, the static bollard force (BF) can be taken into account by reducing its value by 50% for seismic design (PIANC, 2001).

4.1.3.4.2.3. Effects of Water on Wall

The presence of water plays an important role in designating the loads on waterfront vertical walls during and after earthquakes. Although the dynamic pressure due to existence of water induced by earthquakes might be significant for coastal structures located in a seismic zone, the related problems are rarely studied. At the seaward side of a structure water can exert dynamic pressures during earthquake on the seaside of the wall. Water in the backfill is also important parameter acting on back of the wall. As few coastal structures are fully impermeable, the level of water in backfill is generally at the same level as the free water at the seaward side of the wall. The total water pressures acting on a structure in the absence of seepage within the backfill can be categorized into two components: hydrostatic pressure, which increases linearly according to depth of the wall and hydrodynamic pressure, which arises due to the earthquake motion (Kramer, 1996).

4.1.3.4.2.3.1 Water Seaward Side of Wall

The significance of earthquake-induced hydrodynamic force on coastal embankment was first reported by Chen (1995). In that study, no sediment deposit was considered in the sea wall system. That is, the seaward side of the sea wall is measured from the SWL to the bottom. Using the conventional approach, only hydrostatic force is present at the landward side of the vertical wall, however, at the seaward side both hydrostatic force and wave force are present. The damage of quay-wall caisson at Taichung Port during Chi-Chi 1999 earthquake is clear evidence that the seismic wave-induced dynamic force could affect the safety of the coastal structures (Chen and Huang, 2001). Dynamic wave forces are not taken into consideration in this study, in case of fully sheltered quays against the waves.

Thus, it is clear that during seismic shaking, the free water in front of the structure exerts a cyclic dynamic loading on the wall and when suction pressure is applied on the wall the critical mode occurs (PIANC, 2001).

The amplitude of the hydrodynamic pressure is; (Westergaard, 1933)

$$p_{dw} = \frac{7}{8} k_h \gamma_w \sqrt{H_w h} \tag{4.39}$$

The resultant hydrodynamic thrust is; (Westergaard, 1933)

$$P_{dw} = \frac{7}{12} k_h \gamma_w H^2 \tag{4.40}$$

where, γ_w is the unit weight of seawater, H_w is the total water depth, h is the water depth for each block (Kramer, 1996).

The point of approach of this force lays 0.4 H_w above sea bottom (PIANC, 2001).

The total water pressure on the face of the wall is the sum of the hydrostatic and hydrodynamic water pressures. Similarly, the total lateral thrust due to the water is equal to the sum of the hydrostatic and hydrodynamic thrusts (Kramer, 1996) (Figure 4.12).

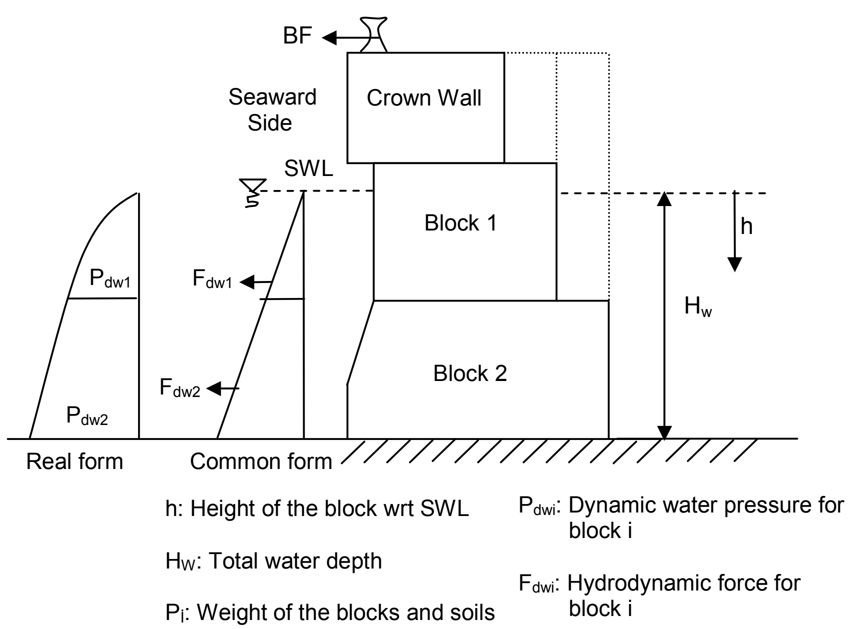


Figure 4.12: Earthquake-induced hydrodynamic force on coastal structure

In order to enable the calculations, the first area is assumed that **triangular** and the other area is assumed that **trapezoidal**; so the earthquake-induced hydrodynamic pressure Eq.(4.40) and force Eq.(4.41) can be computed.

$$p_{dwi} = \frac{7}{8} k_h \gamma_w \sqrt{H_w h_n} \tag{4.41}$$

$$F_{dwi} = \left(\frac{p_{dwi} + p_{dw(i-1)}}{2} \right) h_i \tag{4.42}$$

For dynamic case combined horizontal forces (surcharge + earth and water) and the vertical forces acting on block type quay wall are shown in (Figure 4.13).

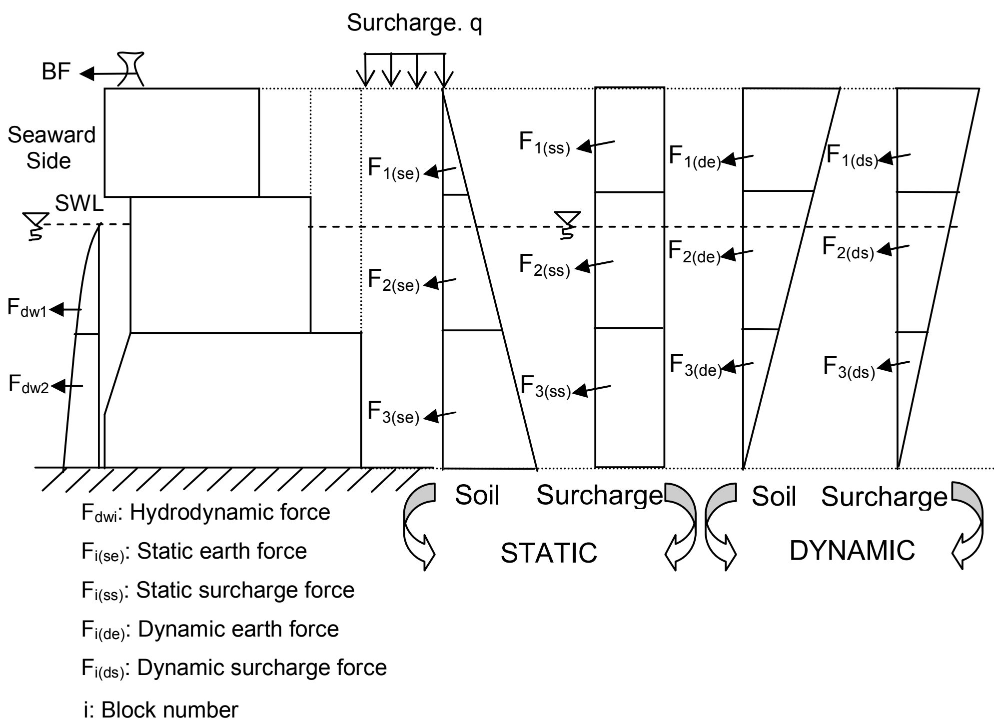


Figure 4.13: For dynamic case combined horizontal forces (surcharge + earth and water) and the vertical forces acting on block type quay wall

4.1.3.4.2.3.2. Water in Backfill

The presence of water in the backfill behind structure can be effective in three ways: (1) changing the inertial forces within the backfill (2) developing the hydrodynamic pressures within the backfill and (3) allowing excess pore water pressure generation because of the cyclic straining of the backfill soils (Kramer, 1996).

The magnitude and the phase relationship of each force component vary with time and are largely affected by the magnitude of excess pore pressure developed in the backfill soil of the quay wall. The dynamic thrust develops at the contact surface between the backfill soil and the wall as a result of the interaction among these force components (Kim et al., 2004). Many laboratory tests indicate, however, that the pore water pressure generally will not remain constant once the soil begins to deform (PIANC, 2001). Since effects of exceedance pore water pressure is poorly defined and mainly depends on engineering judgment.

4.1.3.4.3 Inertia Forces

By using the Pseudostatic Analysis; the inertia forces are calculated accrording to Eq. (4.6);

$$F_h = \frac{a_h W}{g} = k_h W$$

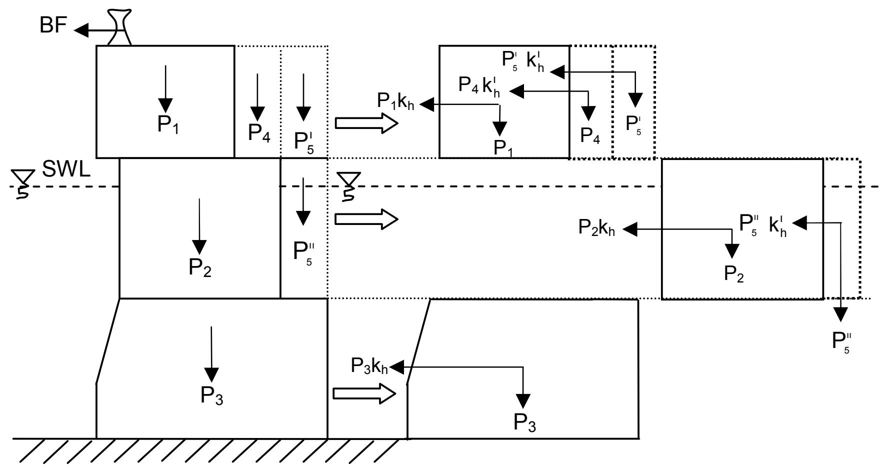
where; F_h is the horizontal force, a_h is the maximum gravitational acceleration, W is the weight of the block or soil, g is the gravity acceleration, k_h is the horizontal seismic coefficient.

Weights of the blocks are computed overrating the buoyancy effect of water or ignoring the buoyancy effect. To define the weights of the soils adjacent to the blocks, equivalent unit weight (γ_{eq}) is used for both dry and submerged part of the soil. Then, weights of the blocks (submerged or dry) are multiplied with the horizontal seismic coefficient (k_h) and weights of the soils are multiplied with the seismic coefficient (k_h) or modified seismic coefficient (k'_h). In this way vertical forces are converted into horizontal forces (Figure 4.14). And, the vertical and horizontal forces acting on block type quay wall are shown in Figure 4.15.

4.1.3.4.4 Vertical Forces

Vertical forces are classified into four parts namely; vertical forces due to block, vertical forces due to soil uplift forces and the vertical component of the total static and dynamic earth and surcharge forces.

4.1.3.4.4.1. Vertical Forces Due to Blocks: By taking into consideration the buoyancy force, the weights of the blocks are calculated.



P_i: Weight of the blocks and soils

k_h: Horizontal seismic coefficient

Figure 4.14: Inertia forces acting on block type quay wall

4.1.3.4.4.2 Vertical Forces Due to Soils: Soils above the blocks are taken as vertical forces.

4.1.3.4.4.3 Uplift Force: As there is no sea level difference the uplift force due to hydrostatic force (results from water table difference) is not considered in this study.

4.1.3.4.4.4 Vertical Component of Total Thrust: The vertical component of the total static and dynamic earth and surcharge forces are taken as vertical forces.

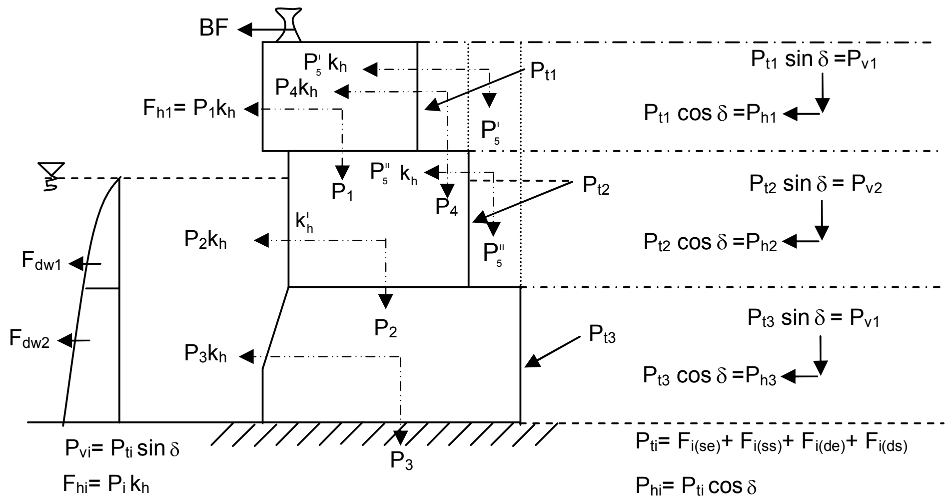


Figure 4.15: The vertical and horizontal forces on block type quay wall

4.1.3.4.5 Stability Analysis (Force and Moment Balance)

By using same approach with TSS, 1975 and TSS, 1997 the factor of safety against sliding and overturning calculation stages are given in Appendix A.

For gravity type quay walls and breakwaters on firm foundations, typical failure modes during earthquakes are not only due to damage structures, but also excessive deformation such as settlement sliding and tilting (Mohajeri et al., 2004). Therefore, the stability of gravity quay walls is evaluated according to sliding of the wall, overturning of the wall and bearing capacity of the foundation. Sliding is generally a critical condition for wall if the wall has a large width to height ratio. Overturning or poor bearing capacity value can be the critical condition for wall if the wall has a small width to height ratio. For strong seismic shaking condition, the instability with respect to overturning and/or bearing capacity is much more serious than the sliding. If the wall tilts, the wall will collapse. Thereby, it is practice to assign a higher safety factor for overturning and bearing capacity than sliding. A wall with relatively small width to height ratio, typically less than about 0.75, will exhibit a predominant tilting failure mode rather than horizontal displacement (PIANC, 2001).

If the earthquake is considered, the factor of safety against sliding and overturning values are recommended as; (OCDI, 2002)

Sliding,

For seismic condition ≥ 1.1

$$FS_S = \frac{\mu F_v}{F_H}$$

Overturning,

For seismic condition ≥ 1.2

$$FS_o = \frac{\sum M_r}{\sum M_o}$$

where; μ is coefficient of friction between the bottom of the wall body and the foundation, F_v is resultant vertical force acting on the wall (kN/m), F_h is resultant horizontal force acting on the wall (kN/m), $\sum M_r$ is the sum of the resisting moments around the toe of the wall, $\sum M_o$ is the sum of the overturning moments around the toe of the wall, FS_s is factor of safety against sliding, FS_o is factor of safety against overturning.

The factor of safety against sliding and overturning are calculated for each block. The forces acting on crown wall, block 1 and block 2 are shown in Figure 4.16, Figure 4.17 and Figure 4.18.

CROWN WALL:

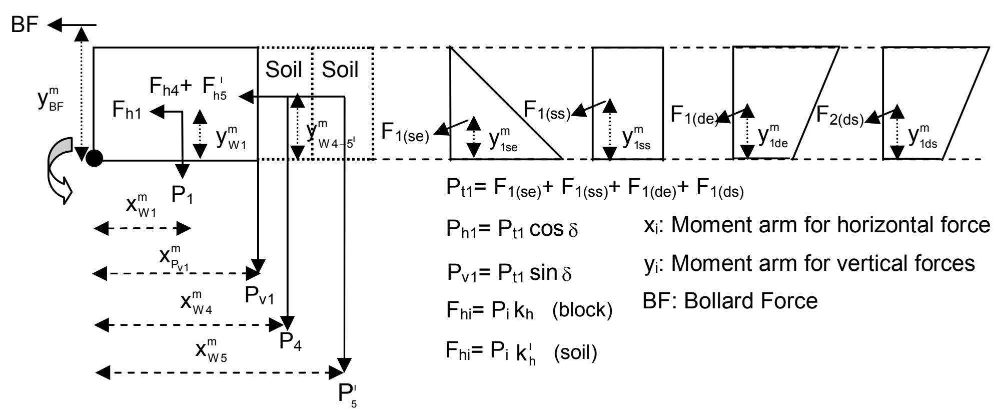


Figure 4.16: The vertical and horizontal forces on crown wall

Total Horizontal Force (F_{Hc}) = $P_{h1} + F_{h1} + F_{h4} + F^{'}_{h5} + BF$

Total Vertical Force (F_{Vc}) = $P_1 + P_{v1}$

Overturning Moments (M_{oc}) = $(F_{1(se)})(y^m_{1se}) + (F_{1(ss)})(y^m_{1ss}) + (F_{1(de)})(y^m_{1de})$

$+ (F_{1(ds)})(y^m_{1ds}))\cos\delta + (F_{h1})(y^m_{w1}) + (F_{h4} + F^{'}_{h5})(y^m_{w4-5'}) + (BF)(y^m_{BF})$

Resisting Moments (M_{rc}) = $(P_1)(x^m_{w1}) + (P_{v1})(x^m_{Pv1})$

for sliding; for overturning;

$$FS_S = \frac{\mu F_v}{F_H} > 1.1 \qquad FS_o = \frac{\sum M_r}{\sum M_o} > 1.2$$

BLOCK 1:

Forces acting on Block 1 are shown in Figure 4.17.

For Block 1:

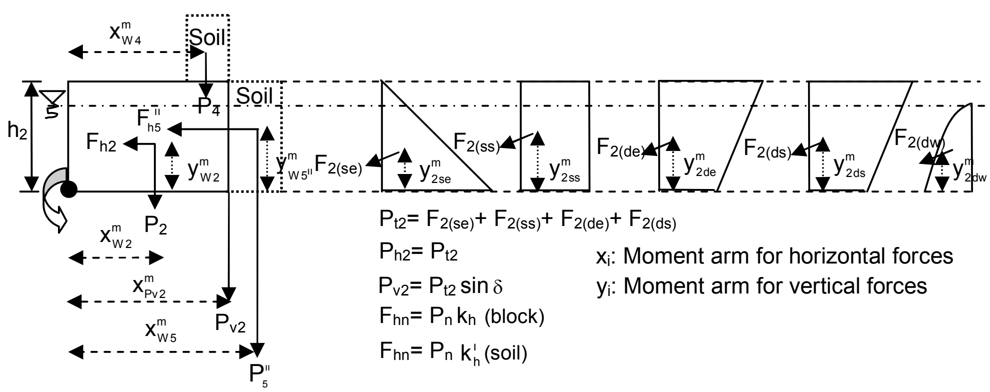


Figure 4.17: The vertical and horizontal forces on the Block 1

$M_{o.Total} = M_{o.c} + \sum M_{o,b1}$

$M_{r.Total} = M_{r.c} + \sum M_{r,b1}$

$\sum$ **Horizontal Force ($F_{H.b1}$)** = $F_{Hc} + P_{h2} + F_{h2} + F^{''}_{h5} + F_{2dw}$

$\sum$ **Vertical Force ($F_{V.b1}$)** = $F_{Vc} + P_2 + P_4 + P_{v2}$

Resisting Moments ($M_{r.b1}$) = $M_{rc} + (P_2)(x^m_{w2}) + (P_4)(x^m_{w4}) + (P_{v2})(x^m_{Pv2})$

Overturning Moments ($M_{o.b1}$) = $M_{oc} + (F_{Hc})(h_2) + ((F_{2(se)})(y^m_{2se}) + (F_{2(ss)})(y^m_{2ss}) + (F_{2(de)})(y^m_{2de}) + (F_{2(ds)})(y^m_{2ds}))\cos\delta + (F_{h2})(y^m_{w2}) + (F''_{h5})(y^m_{w5''}) + (F_{2(dw)})(y^m_{2ss})$

for sliding;

$$FS_S = \frac{\mu F_v}{F_H} > 1.1$$

for overturning;

$$FS_o = \frac{\sum M_r}{\sum M_o} > 1.2$$

BLOCK 2:

Forces acting on Block 2 are shown in Figure (4.18).

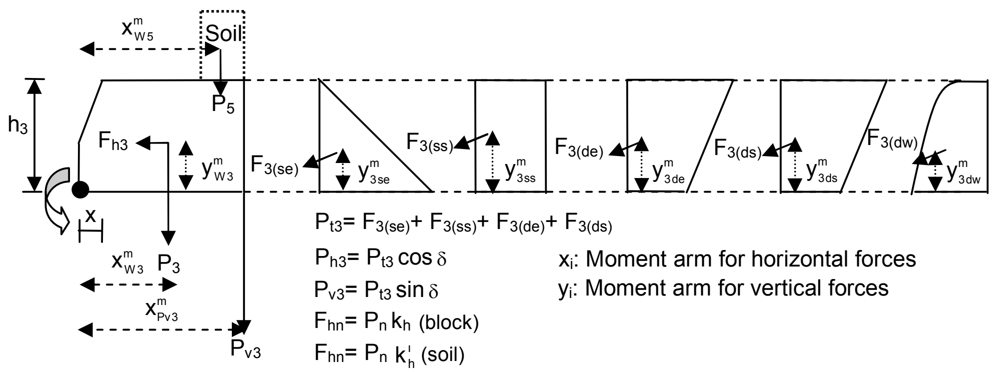


Figure 4.18: The vertical and horizontal forces on the Block 2

$M_{o,b2} = M_{o,(c+b1)} + \sum M_o$

$M_{r,b2l} = M_{r,(c+b\ 1)} + \sum M_r$

$\sum$ **Horizontal Force ($F_{H.b2}$)** $= F_{H.b1} + P_{h3} + F_{h3} + F_{3(dw)}$

$\sum$ **Vertical Force ($F_{V.b2}$)** $= F_{V.b1} + P_3 + P_5 + P_{v3}$

Resisting Moments ($M_{r.b2}$) = $M_{r.b1} + (F_{V.b1})(x) + (P_3)(x^m_{w3}) + (P_5)(x^m_{w5}) + (P_{v3})(x^m_{Pv3})$

Overturning Moments ($M_{o.b2}$) = $M_{o.b1} + (F_{H.b1})(h_3) + ((F_{3(se)})(y^m_{3se}) + (F_{3(ss)})(y^m_{3ss}) + (F_{3(de)})(y^m_{3de}) + (F_{3(ds)})(y^m_{3ds}))\cos\delta + (F_{h3})(y^m_{w3}) + (F_{3(dw)})(y^m_{3dw})$

for sliding;

$$FS_S = \frac{\mu F_v}{F_H} > 1.1$$

for overturning;

$$FS_o = \frac{\sum M_r}{\sum M_o} > 1.2$$

During seismic stability analysis using the new design approach to conventional method, it is realized that 4 different approaches, Approach 1 (which is explained in detail above), Approach 2, Approach 3 and Approach 4 can be applied. Thus the most critical condition can be determined comparing these approaches' results. Same calculations steps are used for all approaches in general. By changing only two parameters (k_h and W) four different approaches are applied for block type quay wall (Table 4.3). The details of the approaches are given in Chapter 5: Case Study on Derince Port, Block Type Quay Wall.

Table 4.3: Different parameters used for approaches

Approaches	**Seismic Coefficient**		***Inertia Forces**		
	k_h	k'_h	W_{dry} (above SWL)	W_{sub} (below –SWL)	W_{dry} (below SWL)
Approach 1		√	√	√	
Approach 2		√	√		√
Approach 3	√		√	√	
Approach 4	√		√		√

*

To define the vertical forces all approaches use the submerged weight for the blocks below the SWL.

CHAPTER 5

A CASE STUDY ON DERINCE PORT, BLOCK TYPE QUAY WALL

The eastern Marmara earthquake occurred on 17 August 1999 with an M_w=7.4 and struck the İzmit Bay and eastern Marmara Sea region, north-west Turkey. The main fault is a single strike-slip fault approximately 140 km long, starting from Sapanca Lake in the east and ending in İzmit Bay in the west (Yüksel et al., 2002).

During Kocaeli Earthquake 1999, over 15.000 fatalities and 20 billion US dollars in losses were observed. Especially at Derince Port is damaged seriously (http://www.jsceint.org/Report/report/kocaeli/kocaeli_chap6.pdf). Derince Port is located near İzmit and the largest port in the area with about 1.5 km of waterfront structures and with eight wharves. The peak ground accelerations were obtained approximately 0.25g to 0.3g (Yüksel et al., 2002, http://www.jsceint.org/Report /report/kocaeli/kocaeli_chap6.pdf).

The earthquake occurred in Izmit, 1999 caused serious damage mostly on block type quay wall in Derince Port (Figure 5.1). Observations revealed that the block type quay wall moved seaward without any vertical displacement. However, 0.5 m lateral displacement towards the sea and 0.5-0.8 m settlement on the backfill behind the quay wall were observed (Yüksel et al., 2002). PIANC 2001, states that 0.7 m lateral displacement was occurred at Derince Port. At some quays mid-span deflections and relative corner movements were observed. The settlement of backfill caused the tilting of a crane on rails. One of the cranes was overturned while others were derailed due to the rocking response to the earthquake shaking. Damages of these cranes caused important loss of serviceability. There was one crane that was fixed to the foundation that did not suffer apparent damage. Also liquefaction was serious problem for the backfill behind the quay wall. The most liquefaction occurs at a location where near a river basin mainly caused by the complexity of sedimentation of the soil.

However, the major problem is sandy backfill material behind the quay walls dredged from a river mount by the sea probably a kind of delta sediment (http://www.jsceint.org/Report/report/kocaeli/kocaeli_chap6.pdf).

The peak ground acceleration and horizontal displacement are known for Derince Port. Thus, to make a comparison by using both the new design approaches and Turkish Seismic Design codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007); stability analysis are made for Derince Port. The new design approaches are classified into four parts, namely, Approach 1, Approach 2, Approach 3 and Approach 4. The proposed approaches' assumptions will be discussed in Chapter 5. The first approach is Approach 1. Figure 5.1 shows the block type quay wall in Derince Port.

5.1 Approach 1

5.1.1 Basic Assumptions and Calculations

1. It is assumed that soil improvement techniques are used for this site so the internal friction of soil taken as ϕ=40^o. And, the friction angle between the soil and block is taken as $(\delta = \phi/3 = 13.33)$. As it is seen from the Figure 4.7, angle between the back of the wall and the vertical plane (θ) and the inclined angle with the horizontal (β) is taken 0 for this study (Table 5.1).

Table 5.1: Necessary parameters to calculate the forces acting on block type quay walls

Φ (deg)	**δ(deg)**	**β(deg)**	**θ (deg)**	**H (m)**	**H_{sub} (m)**	**q (kN/m^2)**
40	13.33	0	0	15	12.5	30

Bollard (kN)	**Cronman Length (m)**	γ_{conc} **(kN/m^3)**	γ_{b-conc} **(kN/m^3)**	γ_{dry} **(kN/m^3)**	γ_{sub} **(kN/m^3)**	γ_w **(kN/m^3)**
800	16.16	23	13	18	11	10

2. Coordinates of Derince port is 29.80 longitude and 40.8 latitude for %50 exceedance probability S_s = 0.76 (TSDC-CRA.2007).

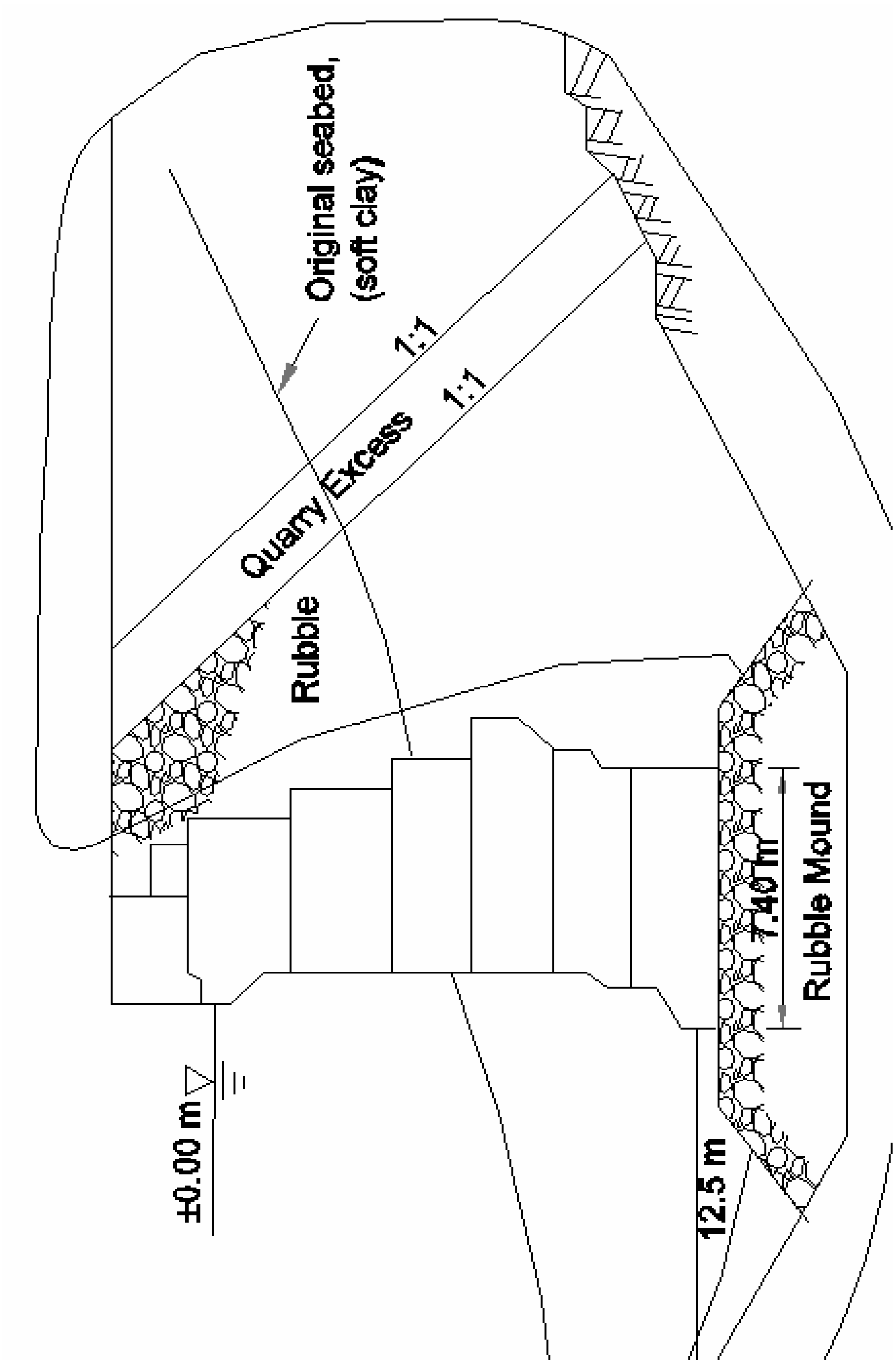


Figure 5.1: Block type quay wall in Derince Port

3. The soil classification is chosen as A, the short period soil coefficient (F_a) is found by using the Table 4.2.

Table 4.2: Short period soil coefficient (F_a) (TSDC-CRA. 2007)

Soil Class	Spectral accelaration (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]

Look Ek 4

[a] Interpolation is made to find S_s.

[b] Special geotechnical analysis and dynamic soil attitude analysis is made.

$S_s = 0.76 \Rightarrow F_a = 0.8$

4. The spectral acceleration value with respect to soil classification (S_{MS}) is; (Eq. 4.16)

$S_{MS} = 0.8 \times 0.76 = 0.61$

5. Effective seismic coefficient for L1 (A_{10}) is; (Eq. 4.18)

$A_{10} = 0.4 \times 0.61 = 0.243$ (a_{max}/g)

6. Horizontal seismic coefficient (k_h) is; (Eq. 4.19)

$k_h = (2/3)(0.243) = 0.16$

7. The modified seismic coefficient (k'_h) is; (Eq. 4.27)

$$k_h^{I} = \frac{18\times 2.5 + 21\times 2 + 21\times 2.5 + 21\times 2 + 21\times 2 + 21\times 2 + 21\times 2 + 15}{18\times 2.5 + 11\times 2 + 11\times 2.5 + 11\times 2 + 11\times 2 + 11\times 2 + 11\times 2 + 15}\times 0.16$$

k_h^{I} =0.26

8. Seismic inertia angle (ψ) is; (Eq. 4.28)

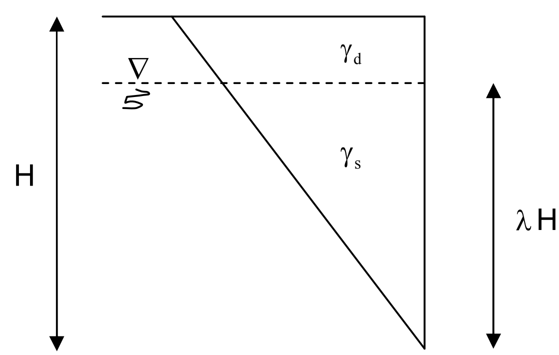
$$\psi = \arctan(k_h^{I})$$

$$\psi = \arctan(0.26) \quad \Rightarrow \quad \psi = 14.57^{o}$$

9. Equivalent unit weight (γ_{eq}) is; (Eq. 4.23) **(Kramer, 1996)**

$$\lambda \times H = H_{sub}$$

$$\lambda \times 15 = 12.5 \quad \Rightarrow \quad \lambda = 0.83$$



H= 15 m

H_{sub}= 12.5 m

γ_d = 18 kN/m^3

γ_s = 11 kN/m^3

and;

$$\gamma_{eq} = \lambda^2\gamma_s + (1-\lambda^2)\lambda_d$$

$$\gamma_{eq} = 0.83^2 11 + (1-0.83^2)18 \quad \Rightarrow \quad \gamma_{eq} \mathbf{= 13.14\ kN/m^3}$$

Also by using PIANC, 2001; γ_{eq} can be calculated (Eq. 4.24).

5.1.1.2 Forces Acting on Block Type Quay Wall

5.1.1.2.1 Static Horizontal Forces

5.1.1.2.1.1 Active Earth Pressure

- Active earth pressure coefficient (K_A) is; (Eq. 4.8)

$$K_A = \frac{\cos^2(40-0)}{\cos^2(0)\cos(0+13.33)\left[1+\sqrt{\dfrac{\sin(40+13.33)\sin(40-0)}{\cos(13.33+0)\cos(0-0)}}\right]^2} \Rightarrow \mathbf{K_A=0.202}$$

- Active earth pressure **(P_A)** is; (Eq. 4.9)

$$P_A = \frac{1}{2}(0.202)\ (13.14)\ (15)^2 \Rightarrow \mathbf{P_A=298.606\ kN/m}$$

- Static earth pressures ($P_{i(se)}$) and static earth forces ($F_{i(se)}$) are calculated and ($F_{i(se)}$) shown in Figure 5.2 (Eq. 4.10 - Eq.4.11).

Crown Wall:

$P_{1(se)} = K_A\ \bar{\gamma}\ H_1 \Rightarrow P_{1(se)} = (0.202)\ (13.14)\ (2) = 5.308\ kN/m^2$

$$F_{1\ (se)} = \frac{5.308 \times 2}{2} = 5.31\ kN/m$$

Block 1:

$P_{2(se)} = K_A\ \bar{\gamma}\ H_2 \Rightarrow P_{2(se)} = (0.202)\ (13.14)\ (4.5) = 11.943\ kN/m^2$

$$F_{2(se)} = \left(\frac{5.308+11.943}{2}\right) \times 2.5 = 21.56\ kN/m$$

Block 2:

$P_{3(se)} = K_A\ \bar{\gamma}\ H_3 \Rightarrow P_{3(se)} = (0.202)\ (13.14)\ (7) = 18.580\ kN/m^2$

$$F_{3(se)} = \left(\frac{18.580+11.943}{2}\right) \times 2.5 = 38.15\ kN/m$$

Block 3:

$P_{4(se)} = K_A\ \bar{\gamma}\ H_4 \Rightarrow P_{4(se)} = (0.202)\ (13.14)\ (9) = 23.888\ kN/m^2$

$$F_{4(se)} = \left(\frac{18.580 + 23.888}{2}\right) \times 2 = 42.47 \text{ kN/m}$$

Block 4:

$P_{5(se)} = K_A \bar{\gamma} H_5 \Rightarrow P_{5(se)} = (0.202)(13.14)(11) = 29.197 \text{ kN/ m}^2$

$$F_{5(se)} = \left(\frac{29.197 + 23.888}{2}\right) \times 2 = 53.09 \text{ kN/m}$$

Block 5:

$P_{6(se)} = K_A \bar{\gamma} H_6 \Rightarrow P_{6(se)} = (0.202)(13.14)(13) = 34.505 \text{ kN/ m}^2$

$$F_{6(se)} = \left(\frac{34.505 + 29.197}{2}\right) \times 2 = 63.70 \text{ kN/m}$$

Block 6:

$P_{7(se)} = K_A \bar{\gamma} H_7 \Rightarrow P_{7(se)} = (0.202)(13.14)(15) = 39.814 \text{ kN/ m}^2$

$$F_{7(se)} = \left(\frac{34.505 + 39.814}{2}\right) \times 2 = 74.32 \text{ kN/m}$$

Static earth forces distribution and values are shown in Figure 5.2.

5.1.1.2.1.2 Surcharge

Static surcharge pressures $(P_{n(ss)})$ and static surcharge forces $(F_{n(ss)})$ are calculated and $(F_{n(ss)})$ for each block are shown in Figure 5.2 (Eq. 4.12 - Eq.4.13).

Crown Wall:

$P_{1(ss)} = q\,K_A \Rightarrow P_{1(ss)} = (30)(0.202) = 6.06 \text{ kN / m}^2$

$F_{1(ss)} = P_{1(ss)}(H_1) \Rightarrow F_{1(ss)} = (6.06)(2) = 12.12 \text{ kN/m}$

Block 1:

$P_{2(ss)} = (30)(0.202) = 6.06 \text{ kN / m}^2$

$F_{2(ss)} = P_{2(ss)}(H_2 - H_1)$

$F_{2(ss)} = (6.06)(4.5-2.0) = 15.15 \text{ kN/m}$

Block 2:

$P_{3(ss)}$ =(30) (0.202) = 6.06 kN / m^2

$F_{3(ss)}$ = $P_{3(ss)}$ (H_3- H_2)

$F_{3(ss)}$ = (6.06) (7-4.5) = 15.15 kN/m

Block 3:

$P_{4(ss)}$ =(30) (0.202) = 6.06 kN / m^2

$F_{4(ss)}$ = $P_{4(ss)}$ (H_4- H_3)

$F_{4(ss)}$ = (6.06) (9-7) = 12.120 kN/m

Block 4:

$P_{5(ss)}$ =(30) (0.202) = 6.06 kN / m^2

$F_{5(ss)}$ = $P_{5(ss)}$ (H_5 - H_4)

$F_{5(ss)}$ = (6.06) (11-9) = 12.12 kN/m

Block 5:

$P_{6(ss)}$ =(30) (0.202) = 6.06 kN / m^2

$F_{6(ss)}$ = $P_{6(ss)}$ (H_6 - H_5)

$F_{6(ss)}$ = (6.06) (13-11) = 12.12kN/m

Block 6:

$P_{7(ss)}$ =(30) (0.202) = 6.06 kN / m^2

$F_{7(ss)}$ = $P_{7(ss)}$ (H_7- H_6)

$F_{7(ss)}$ = (6.06) (15-13) = 12.12 kN/m

Static surcharge forces distribution and values are shown in Figure 5.3.

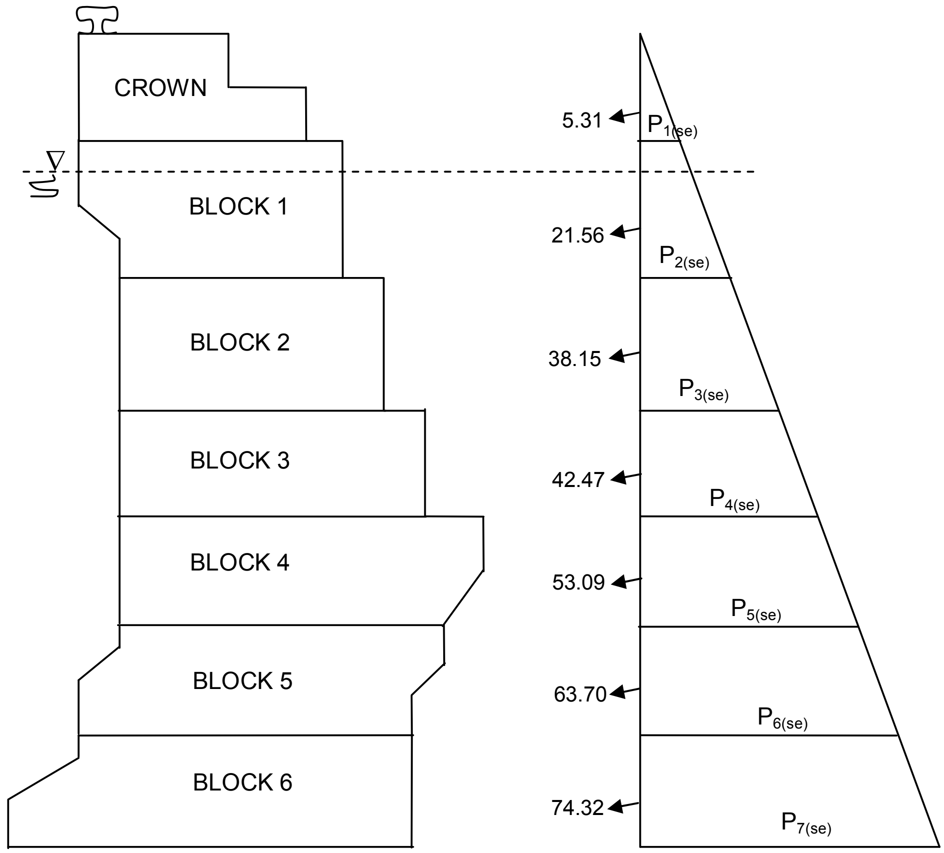


Figure 5.2: The soil effect on block type quay wall for static case

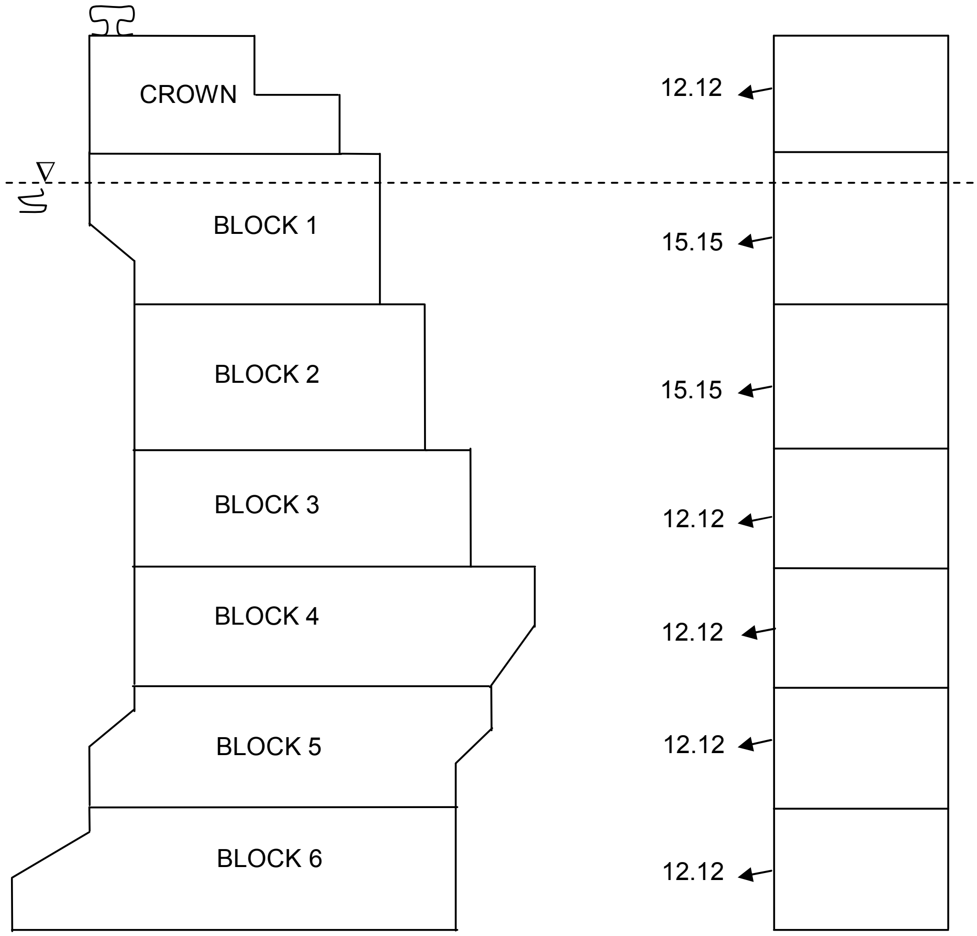


Figure 5.3: The surcharge effect on block type quay wall for static case

5.1.1.2.1.3 Hydrostatic Forces

It is assumed that the water levels on both sides of the wall are the same. The static water forces acting on the both sides of the wall are not considered.

5.1.1.2.2 Dynamic Horizontal Forces

5.1.1.2.2.1 Dynamic Active Earth Pressure

- Dynamic active earth pressure coefficient (K_{AE}) is; (Eq. 4.14)

$$K_{AE} = \frac{(1-0)\cos^2(40-0-14.57)}{\cos 14.57 \cos^2(0)\cos(0+13.33+14.57)\left[1+\sqrt{\dfrac{\sin(40+13.33)\sin(40-14.57)}{\cos(13.33+0+14.57)\cos(0-0)}}\right]^2}$$

$\mathbf{K_{AE} = 0.361}$

- Total active thrust (P_{AE}). ($k_v \cong 0$) (Eq. 4.15)

$P_{AE} = \frac{1}{2}(0.361)(13.14)(15)^2 \Rightarrow$ **P_{AE}= 533.484 kN/m**

- Dynamic component of total thrust (ΔP_{AE}) is ; (Eq. 4.30)

$\Delta P_{AE} = 533.484 - 298.606 \quad \Delta P_{AE} = 234.878$ **kN/m**

- Dynamic earth pressures ($P_{i(de)}$) and dynamic earth forces ($F_{i(de)}$) are calculated and ($F_{i(de)}$) for each block are shown in Figure 5.4 (Eq. 4.32 - Eq.4.34).

1. To begin with; seismic active angle of failure is (α_{ae});

$\alpha_{ae} = (\phi/2 + 45) = 65^o$

$\alpha = 90\text{-}65^o \Rightarrow \alpha = 25^o$

or (α_{ae}) is; (PIANC, 2001) (Eq.4.31)

$$\alpha_{ae} = 40 + \arctan\left[\frac{-\tan 40 + \sqrt{\tan 40(\tan 40 + \cot 40)(1 + \tan 13.33 \cot 40}}{1 + \tan 13.33(\tan 40 + \cot 40)}\right] = 64^0$$

$\alpha = 90\text{-}64 = 26^o$ is chosen; (PIANC, 2001)

2. Then the areas are; (Figure 5.4)

$$A_1 = \left(\frac{7.31 + 6.34}{2}\right) \times 2 = 13.65 \ \text{m}^2$$

$$A_2 = \left(\frac{6.34 + 5.12}{2}\right) \times 2.5 = 14.33 \ \text{m}^2$$

$$A_3 = \left(\frac{5.12 + 3.90}{2}\right) \times 2.5 = 11.28 \ \text{m}^2$$

$$A_4 = \left(\frac{3.90 + 2.93}{2}\right) \times 2 = 6.83 \ \text{m}^2$$

$$A_5 = \left(\frac{2.93 + 1.95}{2}\right) \times 2 = 4.88 \ \ m^2$$

$$A_6 = \left(\frac{1.95 + 0.97}{2}\right) \times 2 = 2.92 \ \ m^2$$

$$A_7 = \left(\frac{0.97 \times 2}{2}\right) = 0.97 \ \ m^2$$

$$\sum \text{Area} = \ 54.86 \ m^2$$

3. $\Delta P_{AE} = 234.878$ kN/m for total area so in order to find ΔP_{AE} for each block;

$$\frac{\Delta P_{AE}}{\sum \text{Area}} = \frac{234.878}{54.86} = 4.28 \ \ \text{(coefficient)}$$

4. Dynamic earth forces $(F_{i(de)})$ are;

$F_{i(de)}$= (coefficient) (A_i)

$F_{1(de)}$= (4.28) (13.65) = 58.44 kN/m

$F_{2(de)}$= (4.28) (14.33) = 61.33 kN/m

$F_{3(de)}$= (4.28) (11.28) = 48.28 kN/m

$F_{4(de)}$= (4.28) (6.83) = 29.23 kN/m

$F_{5(de)}$= (4.28) (4.88) = 20.89 kN/m

$F_{6(de)}$= (4.28) (2.92) = 12.50 kN/m

$F_{7(de)}$= (4.28) (0.97) = 4.15 kN/m

Dynamic earth forces distribution and values are shown in Figure 5.4.

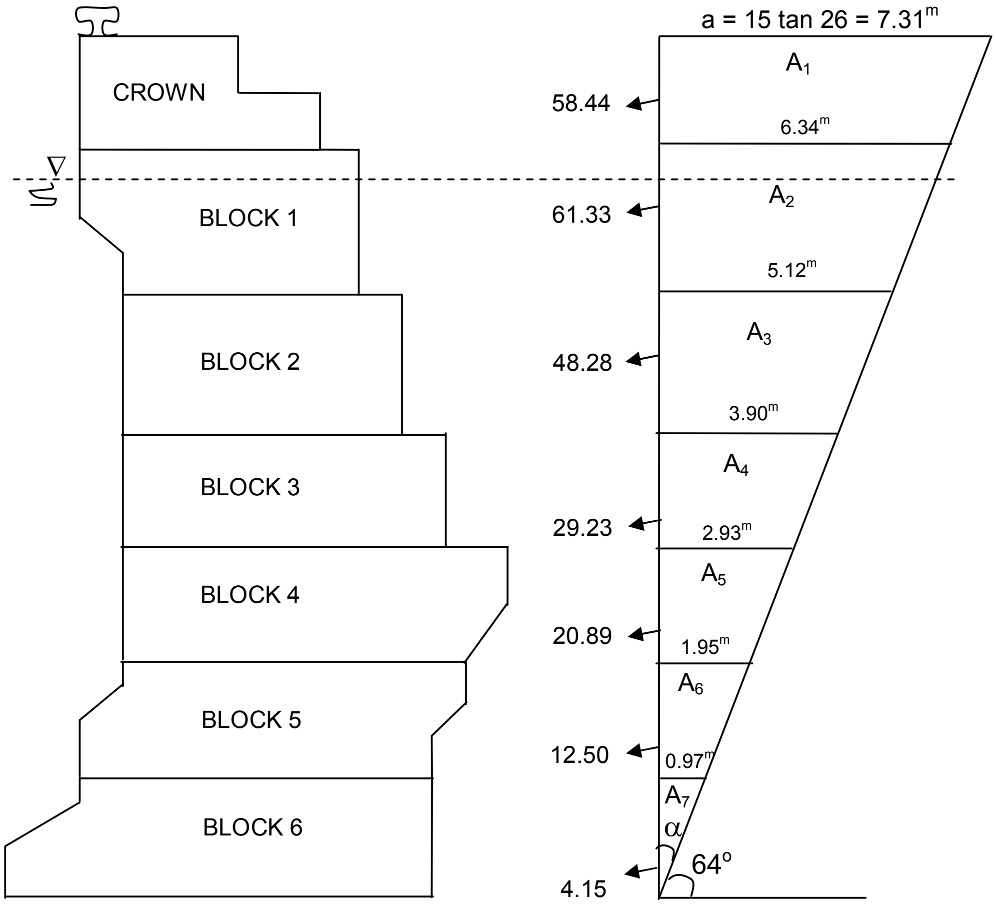


Figure 5.4: The soil effect on block type quay wall for dynamic case

5.1.1.2.2.2 Surcharge

Dynamic surcharge pressures $(P_{i(ds)})$ and dynamic earth forces $(F_{i(ds)})$ are calculated and $(F_{i(ds)})$ for each block are shown in Figure 5.5 (Eq. 4.35 - Eq.4.38).

- q = 30 kN / m^2

  For this calculation (q / 2) is taken.

- Areas are calculated;

$A_1 = 13.65\ m^2$, $A_2 = 14.33\ m^2$,
$A_3 = 11.28\ m^2$, $A_4 = 6.83\ m^2$, $A_5 = 4.88\ m^2$, $A_6 = 2.92\ m^2$.

$A_7 = 0.97\ m^2$ and $\sum Area = 54.86\ m^2$

- W = (q / 2) (a) (b)

  W = (30/2) (7.31) (1) $\Rightarrow$ W = 109.65 kN/m

- $F_{ds} = W\ k_h^I$

  F_{ds} = (109.65) (0.26)= 28.51 kN/m

- In order to find surcharge force for each blocks;

$$\frac{F_{ds}}{\sum Area} = \text{coefficient} \text{ and } \text{coefficient} = \frac{28.51}{54.86} = 0.52$$

$F_{i(de)}$= (coefficient) (A_i)

$F_{1(ds)}$= (0.52) (13.65) = 7.10 kN/m

$F_{2(ds)}$= (0.52) (14.33) = 7.45 kN/m

$F_{3(ds)}$= (0.52) (11.28) = 5.87 kN/m

$F_{4(ds)}$= (0.52) (6.83) = 3.55 kN/m

$F_{5(ds)}$= (0.52) (4.88) = 2.54 kN/m

$F_{6(ds)}$= (0.52) (2.92) = 1.52 kN/m

$F_{7(ds)}$= (0.52) (0.97) = 0.50 kN/m

Dynamic surcharge forces distribution and values are shown in Figure 5.5.

The static earth forces $(F_{i(se)})$, static surcharge forces $(F_{i(ss)})$, dynamic earth forces $(F_{i(de)})$ and dynamic surcharge force $(F_{i(ds)})$ are added and horizontal and vertical components are shown in Figure 5.6.

5.1.1.2.2.3 Hydrodynamic Forces

Dynamic water pressure (P_{dwi}) and dynamic water forces (F_{dwi}) are calculated and (F_{dwi}) for each block are shown in Figure 5.7.

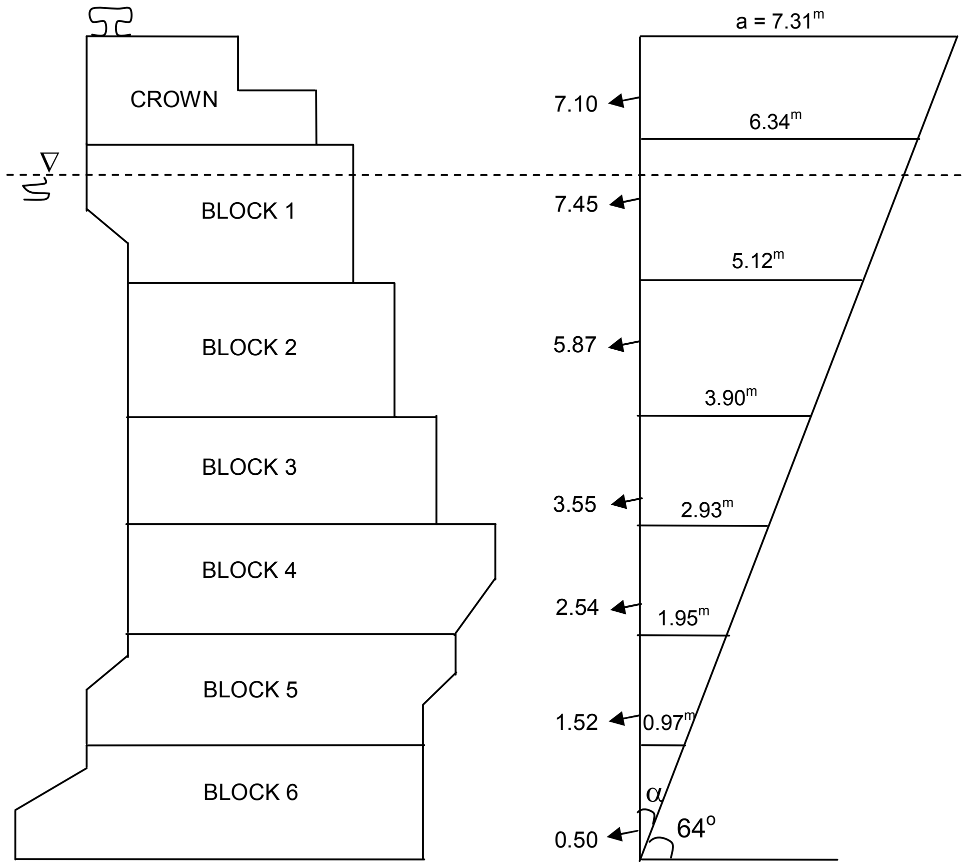


Figure 5.5: The surcharge effect on block type quay wall for dynamic case

Using Eq.4.40 and Eq.4.42; the hydrodynamic pressures and forces are calculated.

$$p_{dw1} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(2)} = 7.00 \text{ kN/m}^2$$

$$p_{dw2} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(4,5)} = 10.50 \text{ kN/m}^2$$

$$p_{dw3} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(6.5)} = 12.62 \text{ kN/m}^2$$

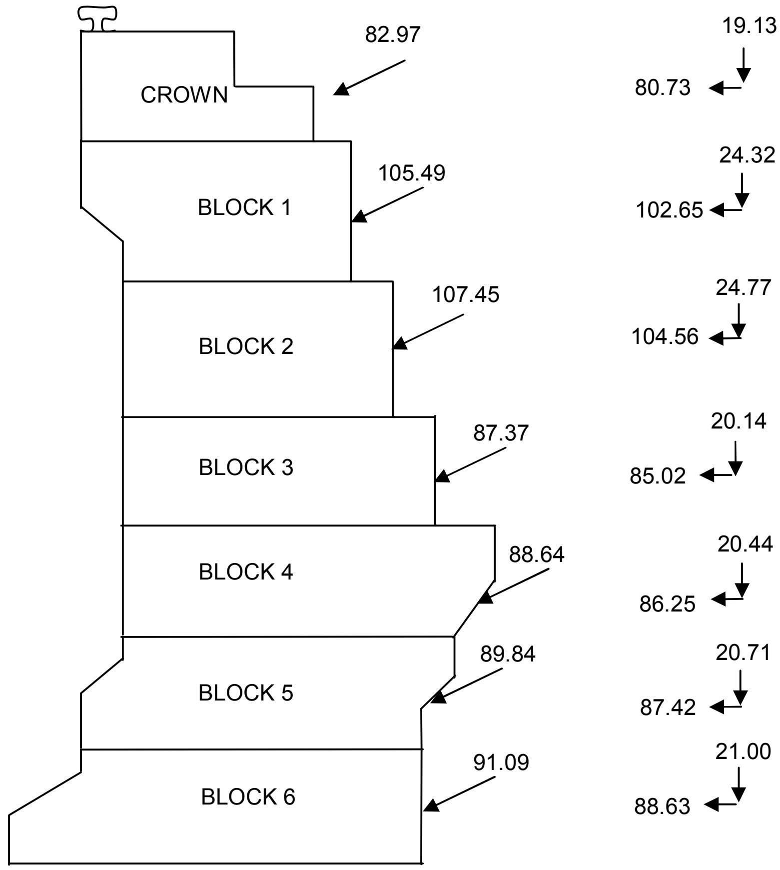


Figure 5.6: The total force (static + dynamic (soil&surcharge)) on block type quay wall.

$$p_{dw4} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(8.5)} = 14.43\ \text{kN/m}^2$$

$$p_{dw5} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(10.5)} = 16.04\ \text{kN/m}^2$$

$$p_{dw6} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(12.5)} = 17.50\ \text{kN/m}^2$$

The first area is assumed that **triangular**;

$$F_{dw1} = \frac{7.00 \times 2}{2} = 7.00\ \text{kN/m}$$

The other areas are assumed that **trapezoidal**;

$$F_{dw2} = \left(\frac{7.00 + 10.50}{2}\right)2.5 = 21.88 \text{ kN/m}$$

$$F_{dw3} = \left(\frac{10.50 + 12.62}{2}\right)2 = 23.12 \text{ kN/m}$$

$$F_{dw4} = \left(\frac{12.62 + 14.43}{2}\right)2 = 27.05 \text{ kN/m}$$

$$F_{dw5} = \left(\frac{14.43 + 16.04}{2}\right)2 = 30.47 \text{ kN/m}$$

$$F_{dw6} = \left(\frac{16.04 + 17.50}{2}\right)2 = 33.54 \text{ kN/m}$$

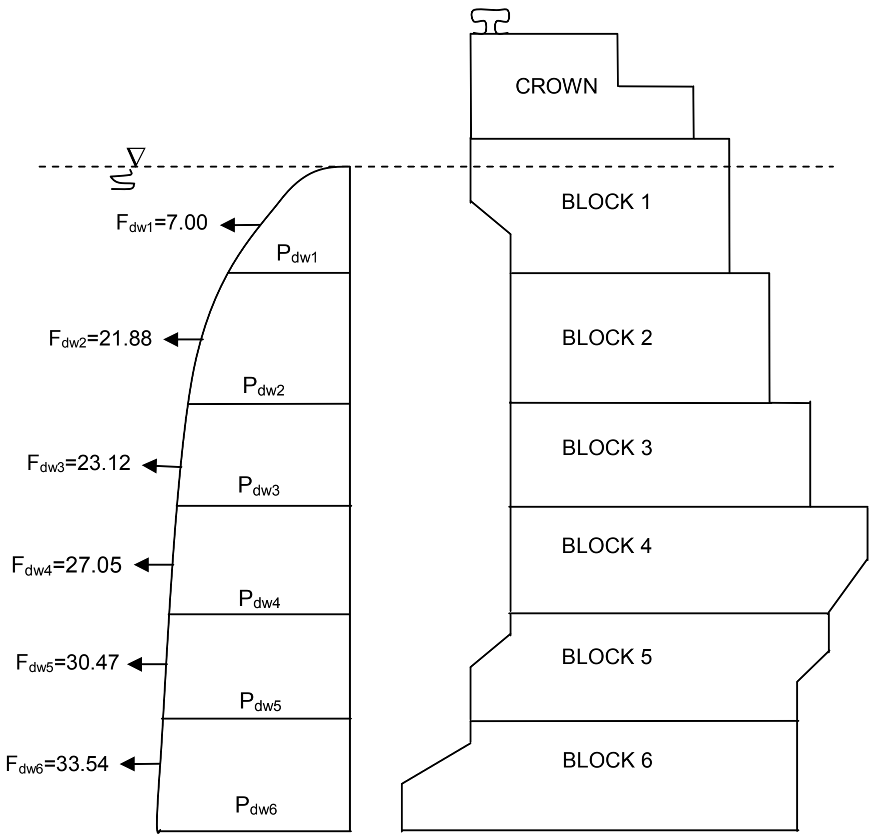


Figure 5.7: The hydrodynamic forces on the block type quay wall

5.1.1.2.2.4 Inertia Forces

To obtain inertia forces, firstly submerged and dry weight of the blocks and weight of the soils should be calculated.

- Submerged weights of the blocks and weights of the soils (γ_{eq} is used):

- **Crown Wall**

W= (a) (b) (γ_c)

W_{CA}=(2.8)(2.0)(23)= 128.8 kN

W_{CB}=(1.4)(1.0)(23)= 32.2 kN

W_{cT}=161 kN/m

The center of gravity;

$$e_{xc} = \frac{128.8 \times 1.4 + 32.2 \times (2.8 + 1.4/2)}{128.8 + 32.2} = 1.82^m$$

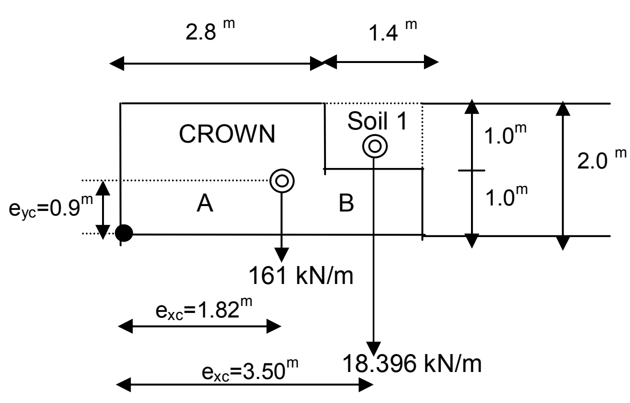
$$e_{yc} = \frac{128.8 \times 1.0 + 32.2 \times 0.5}{128.8 + 32.2} = 0.90^m$$

Soil 1

W_{s1} = (1.4) (1)(13.14)= 18.396 kN/m

The center of gravity;

e_{xs1} =1.4/2+2.8=3.5^m



- **Block 1**

W= (a) (b) (γ_c)

W_{1A}=(4.75)(0.5)(23)= 54.63 kN/m

W_{1D}=(4.75)(2.0)(13)= 123.5 kN/m

W_{1B}=(0.75)(0.75)/2(13)= 3.656 kN/m

W_{1C}=(0.75)(0.75)(13)= 7.313 kN/m

W_{b1T} = 54.63+123.5-3.656-7.313 =167.16 kN/m

The center of gravity;

$$e_{xb1} = \frac{(54.63)(2.375) + (123.5)(2.375) - (3.66)(0.50) - (7.313)(0.375)}{167.16} = 2.51$$

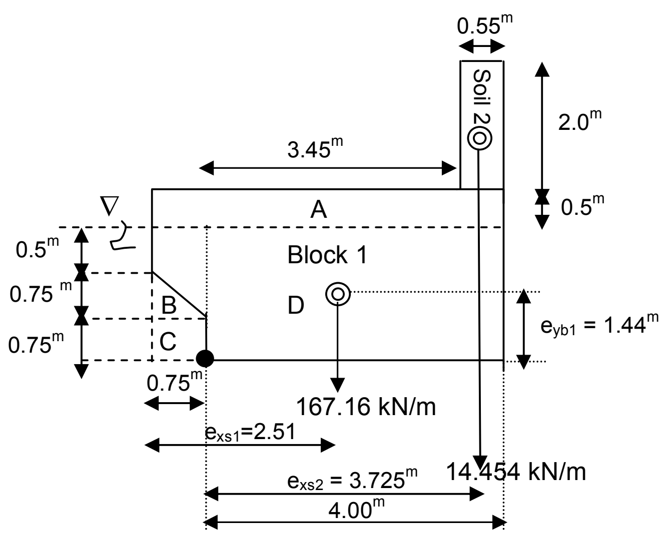
$$e_{yb1} = \frac{(54.63)(2.25) + (123.5)(1.0) - (3.66)(1.0) - (7.313)(0.375)}{167.16} = 1.44$$

Soil 2

W_{s2} = (0.55) (2)(13.14)= 14.454 kN/m

The center of gravity;

e_{xs2} =0.55/2+3.45=3.725 ^{m}



- **Block 2**

W= (a) (b) (γ_c)

W_{b2}= (4.75)(2.5)(13)=154.375 kN/m

The center of gravity;

e_{xb2}= 4.75/2 +4=2.375^m

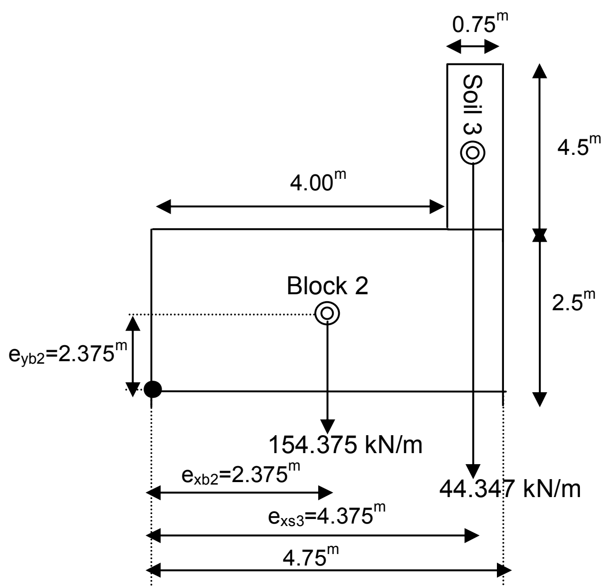
e_{yb2}= 4.75/2 +4=2.375^m

Soil 3

W_{s3}=(0.75)(4.5)(13.14)=44.347 kN/m

The center of gravity;

e_{xs3}= 0.75/2 +4=4.375^m



- **Block 3**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{3T}= (5.5)(2.0)(13)=143.00 kN/m

The center of gravity;

e_{xb3} = 5.5/2 =2.75^m

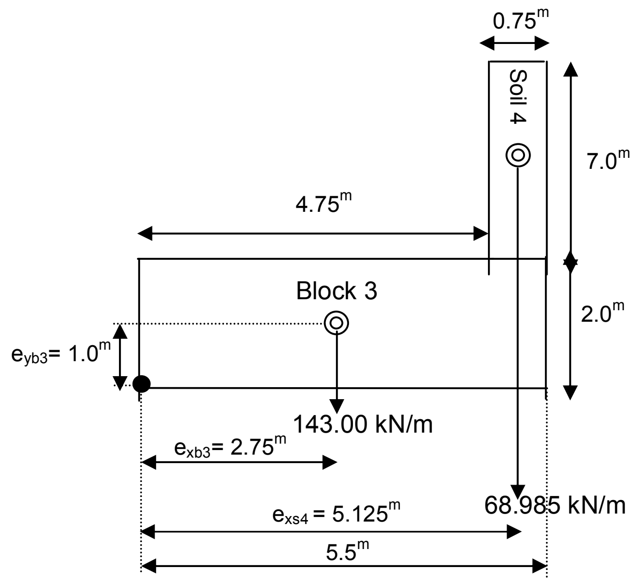
e_{yb3} = 2.2/2 = 1.0^m

Soil 4

W_{s4}=(0.75)(7.0)(13.14)=68.985 kN/m

The center of gravity;

e_{xs4}= 0.75/2 +4.75=5.125^m



- **Block 4**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{4A}= (6.6)(2.0)(13)=171.6 kN/m

W_{4B}= $\dfrac{(0.75)(1.0)}{2}$(13)=4.875 kN/m

W_{b4T}= 171.6-4.875 =166.725 kN/m

The center of gravity;

$$e_{xb4}=\frac{(171.6)(6.6/2)-(4.875)(5.85+0.5)}{166.725}=3.21^m$$

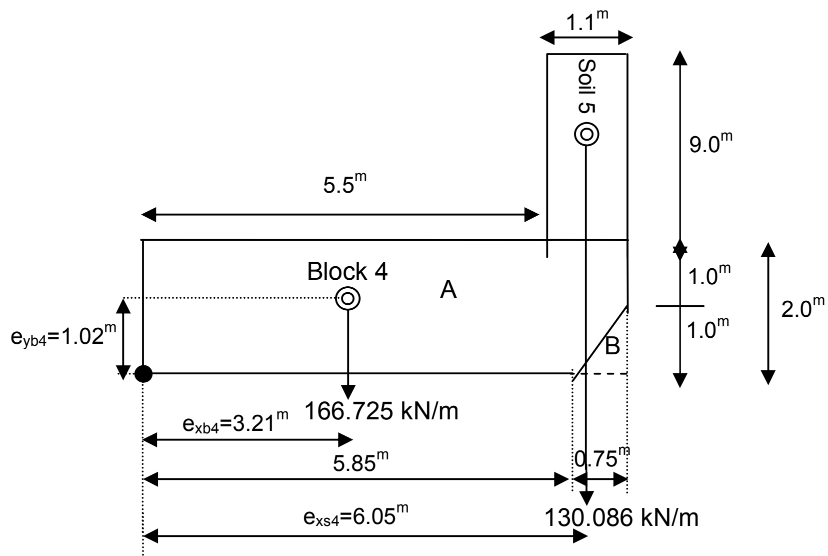
$$e_{yb4}=\frac{(171.6)(2.0/2)-(4.875)(0.333)}{166.725}=1.02^m$$

Soil 5

W_{s5}=(1.10)(9.0)(13.14)=130.086 kN/m

The center of gravity;

e_{xs5}= 1.1/2 +5.5=6.05^m



- **Block 5**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{5A}= (6.6)(2.0)(13.0)= 171.6 kN/m

W_{5B}= (0.75)(0.5)(13.0)= 4.875 kN/m

W_{5C}= $\frac{(0.5)(0.5)}{2}(13.0) = 1.625$ kN/m

W_{5D}= $\frac{(0,6)(0,75)}{2}(13,0) = 2.925$ kN/m

W_{5E}= (0.4)(0.75)(13.0)= 3.9 kN

W_{b5T}= 171.6-4.875-1.625-2.925 -3.9 =158.275 kN/m

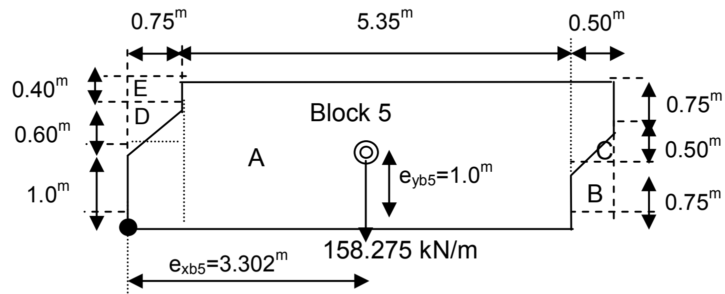
The center of gravity;

$$e_{xb5}=\frac{(171.60)(6.60/2)-(4.875)(6.35)-(1.625)(6.43)-(2.925)(0.25)}{158.275}$$

$$\frac{(3.9)(0.375)}{158.275} = 3.302\ ^{m}$$

$$e_{yb5}=\frac{(171.60)(1.0)-(4.875)(0.375)-(1.625)(0.917)-(2.925)(1.40)}{158.275}$$

$$-\frac{(3.9)(1.80)}{158.275}=1.0^m$$



- **Block 6**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{6A}= (7.4)(2.0)(13.0)= 192.40 kN/m

W_{6B}= $\frac{(1.30)(0.8)}{2}(13.0)$ = 6.76 kN/m

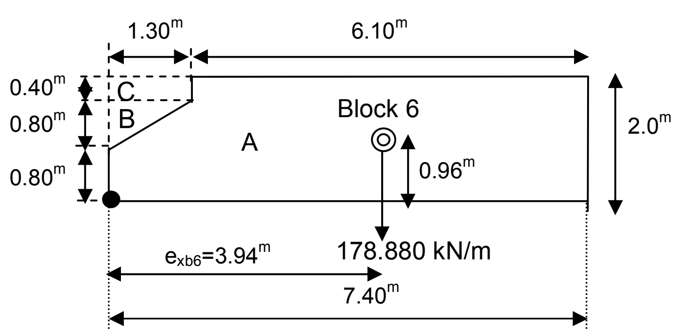
W_{6C}= (1.30)(0.4)(13.0)= 6.76 kN/m

W_{b6T}= 192.40-6.76-6.76 = 178.88 kN/m

The center of gravity;

$$e_{xb6}=\frac{(192.40)(7.40/2)-(6.76)(1.30/3)-(6.76)(1.30/2)}{178.88}=3.94^m$$

$$e_{yb6}=\frac{(192.40)(2.0/2)-(6.76)(1.33)-(6.76)(1.80)}{178.88}=0.96^m$$



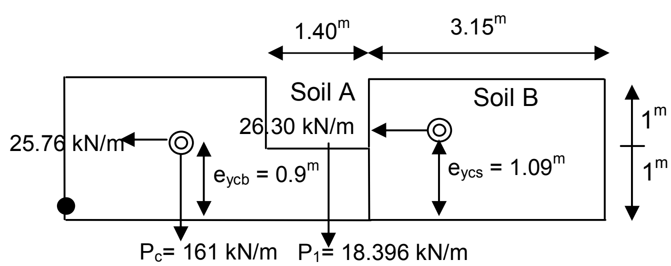
➢ Dry weights of the blocks and weights of the soils (γ_{eq} is used) Table 5.2:

Table 5.2: Dry weights of the blocks and weights of the soils

BLOCKS			$W_{(dry)}$	(e_x)	(e_y)
Block 1-a	2.80	2.00	128.80	1.40	1.00
Block 1-b	1.40	1.00	32.20	1.82	0.90
Soil 1	1.40	1.00	18.394	3.50	1.50
Block1			253.72	1.777	1.44
Soil 2	0.55	2.00	14.45	3.725	1.00
Block2	4.75	2.50	273.13	2.375	1.25
Soil 3	0.75	4.50	44.344	4.375	1.25
Block3	5.50	2.00	253.00	2.750	1.00
Soil 4	0.75	7.00	68.98	5.125	1.00
Block 4			294.98	3.211	1.02
Soil 5	1.10	9.00	130.08	6.05	1.00
Block 5			280.03	3.302	1.00
Block 6			316.48	3.939	0.96

The submerged and dry weights of the blocks and weights of the soils are obtained thus inertia forces can be calculated. By using the principle of Pseudostatic Method (Eq 4.5);

- **Crown Wall**



From crown wall;

W_{cT} =128.8+32.2= 161 kN and k_h=0.16

F_{hC}= (161)(0.16)=25.76 kN/m

Total weight of Soil A : (γ_{eq} =13.14 kN/m^3)

(1.4)(1.0)(13.14)= 18.396 kN/m

From Soil A ;

(18.396)(0.26)= 4.78 kN/m

Total weight of Soil B: (γ_{eq} =13.14 kN/m^3)

$3.15 \times 2 \times 13.14 = 82.782$ kN/m

From Soil B;

(82.782)(0.26)=21.52 kN/m

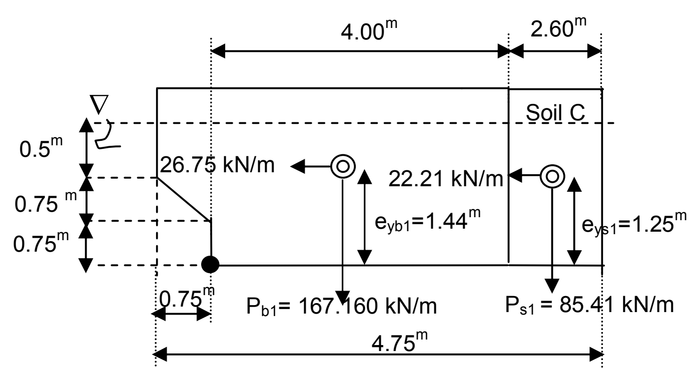
** Total Blocks Force = 128.8+32.2 = 161 kN/m

$$e_{ycb} = \frac{(128.8)(1.0)+(32.2)(0.5)}{161} = 0.9$$

** Total Soil Force = 4.78+21.52=26.30 kN/m

The center of gravity of soil= $\frac{(4.78)(1.0/2+1.0)+(21.52)(1.0)}{26.30} = 1.09^m$

- **Block 1**



Total weight of Block 1:

W_{b1T}=167.160 kN/m

From Block 1;

F_{hb1}= (167.16) (0.16)= 26.75 kN/m

e_{yb1} = 1.44^m

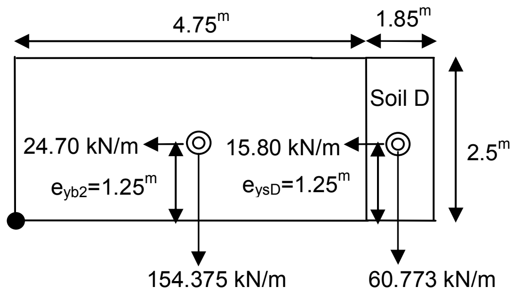
Total weight of Soil C : (γ_{eq} =13.14 kN/m^3)

W_{sC}= (2.60)(2.50)(13.14) = 85.41 kN/m

From soil C ;

$F_{hsc(sub)}$= (85.41)(0.26)=22.21 kN/m

- **Block 2**



Total weight of Block 2:

W_{b2T}= 154.375 kN/m

From block 2 ;

F_{hb2}= (154.375)(0.16) =24.70 kN/m

$e_{yb2} = 1.25^m$

Total weight of Soil D: (γ_{eq} =13.14 kN/m^3)

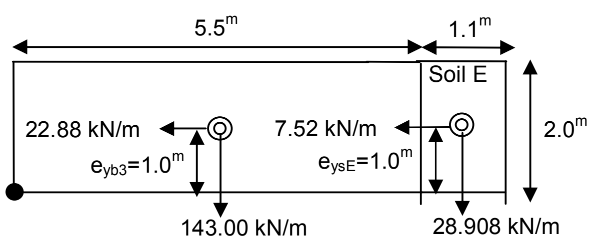
W_{sD}=(1.85)(2.5)(13.14) = 60.773 kN/m

From Soil D ;

(60.773)(0.26)=15.80 kN/m

$e_{ysD} = 2.5/2 = 1.25^m$

- **Block 3:**



Total weight of Block 3:

W_{b3T}=143 kN/m

From block 3;

F_{hb3}= (143.00)(0.18)=22.88 kN/m and $e_{yb3} = 1.0^m$

Total weight of Soil E: (γ_{eq}=13.14 kN/m^3)

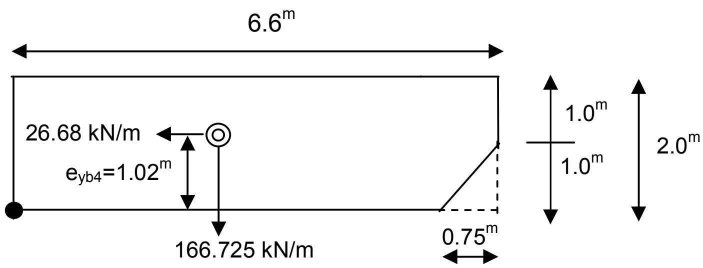
W_{s3}=(1.10)(2.0)(13.14) = 28.908 kN/m

From Soil E ;

(28.908)(0.26)= 7.52 kN/m

$e_{ysE} = 2/2 = 1^m$

- **Block 4:**



Total weight of Block 4:

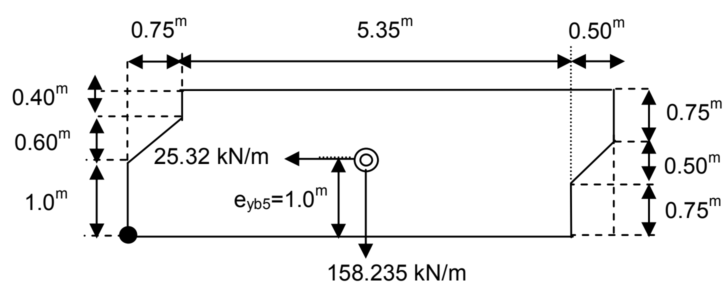
W_{4T}= 166.725 kN/m

From block 4;

F_{hb4}= (166.725)(0.16)=26.68 kN/m

$e_{yb4} = 1.02^m$

- **Block 5:**



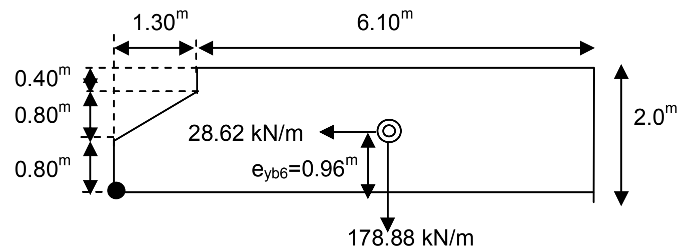
Total weight of Block 5:

W_{b5T}= 158.235 kN/m

From block 5;

F_{hb5}= (158.275)(0.16)=25.32 kN/m and $e_{yb5} = 1.0^m$

- **Block 6:**



Total weight of Block 6:

W_{b6T} =178.88 kN/m

From block 6;

F_{h6}= (178.880)(0.16)=28.62 kN/m

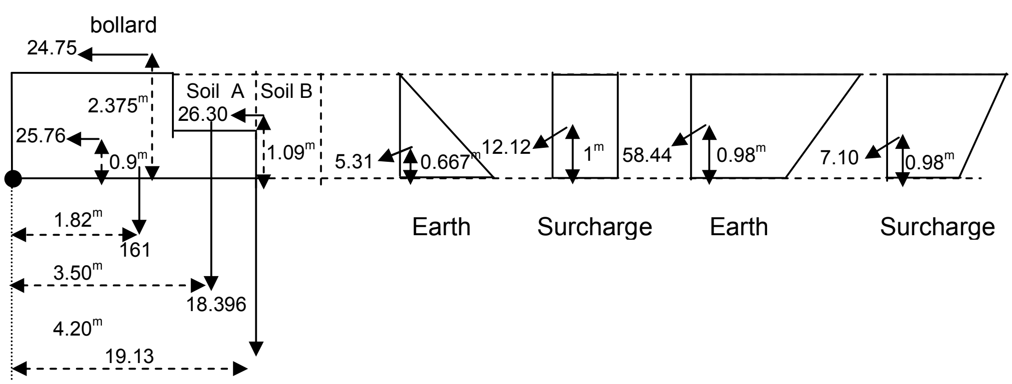
e_{yb6}=0.96^m

5.1.1.2.3 Vertical Forces

The vertical forces are submerged unit weights of the blocks and unit weight of the soils and these values are calculated in 5.1.1.2.2.4 Inertia Forces.

5.1.1.2.4 Stability Analysis

- **Crown Wall**



$\sum$ **Horizontal Force** =(5.31+12.12+58.44+7.10)cos13.33+25.76+26.30 +24.75=157.54 kN/m

∑ **Vertical Force** = 161+18.396+19.13 =198.53 kN/m

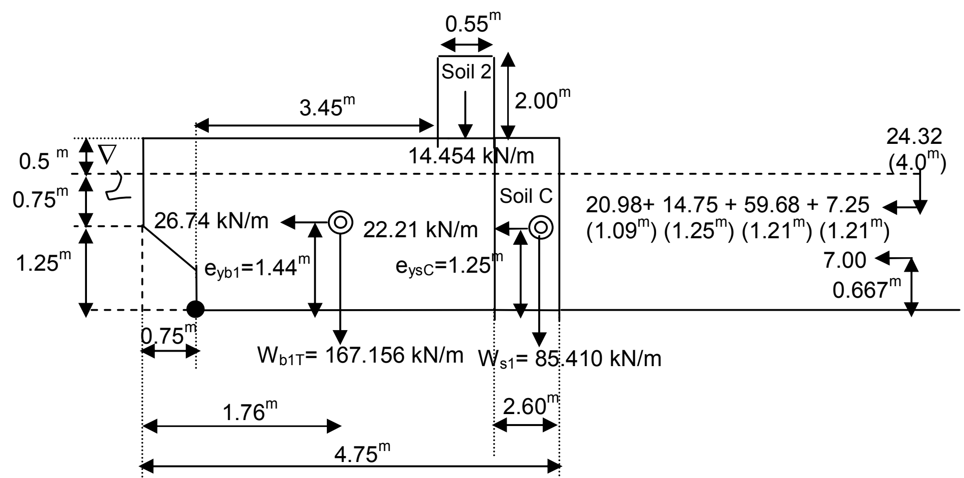
Overturning Moments =((5.31)(0.67)+(12.12)(1.00)+(58.44)(0.98)
+(7.10)(0.98))cos13.33+(25.76)(0.90)+(26.30)(1.09)+(24.75)(2.375)
=188.44kNm/m

Resisting Moments = (161)(1.82)+(18.396)(3.50) +(19.13)(4.20)
= 437.75 kNm/m

FACTOR of SLIDING and OVERTURNING

$FS_s = \frac{(0.50)(198.53)}{157.54} = 0.63 < 1.1(\ x\ )$ $\qquad FS_o = \frac{437.75}{188.44} = 2.32\ \ > 1.2\ \ (ok)$

- **Block 1:**



∑ **Horizontal Force** = 157.54 +20.98+14.75+59.68+7.25+7.00+26.74
+22.21= 316.14 kN/m

∑ **Vertical Force** = 198.53 +167.156+14.454+24.32 = 404.46 kN/m

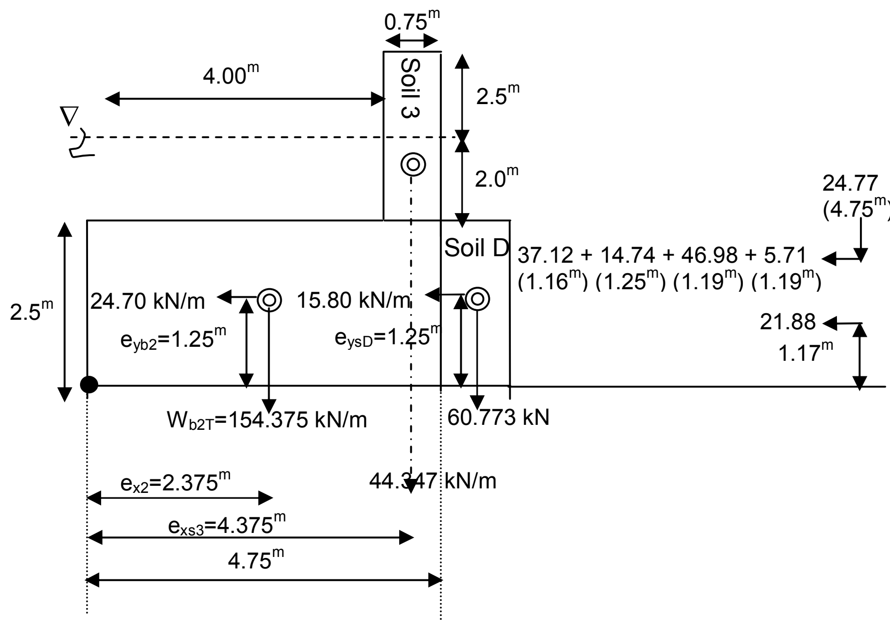
Overturning Moments =188.44 +(157.54)(2.5)+(20.98)(1.09) +(14.75)(1.25)
+(59.68)(1.21) +(7.25)(1.21) +(7.00)(0.667) +(26.74)(1.44)+(22.21)(1.25)
= 775.54 kNm/m

Resisting Moments = (128.8)(0.65)+(32.2)(2.75)+ (18.396)(2.75) +(19.13)(3.45)
+(167.156)(1.76)+(14.454)(3.725)+(24.32)(4.0) = 734.17 kNm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.50)(404.46)}{316.14} = 0.64 < 1.1 \ \ (\ x\ ) \qquad FS_o = \frac{734.17}{775.74} = 0.95 \ \ < 1.2 \ \ (\ x\ )$$

- **Block 2**



∑ **Horizontal Force** = 316.14 +104.56+ 24.70+15.80+21.88= 483.08 kN/m

∑ **Vertical Force** = 404.46 +154.375+44.347+24.77 = 627.95 kN/m

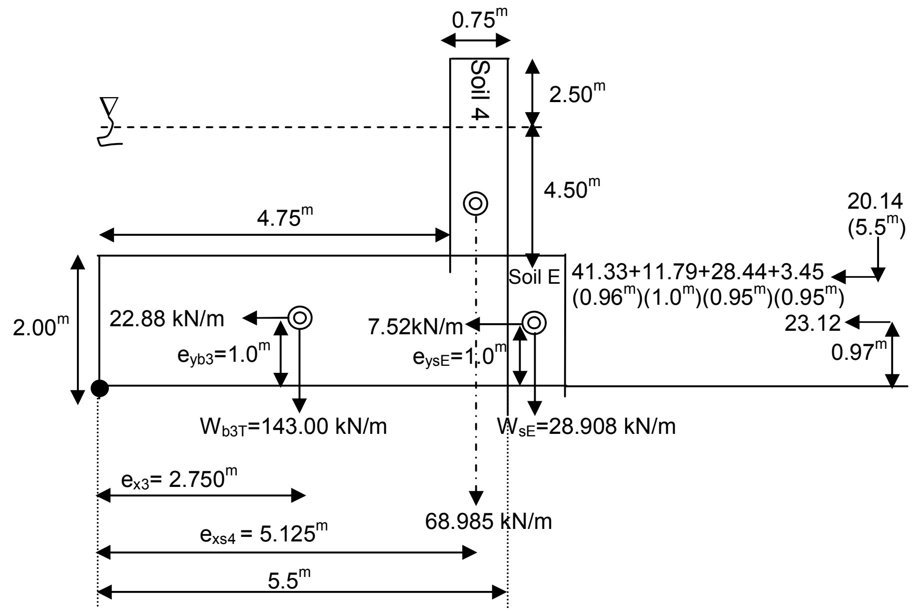
Overturning Moments = 775.54 + (316.14)(2.5)+(37.12)(1.16) +(14.74)(1.25) +(46.98)(1.19)+(5.71)(1.19) +(21.88)(1.17) +(24.70)(1.25) +(15.80)(1.25)
= 1766.21 kNm/m

Resisting Moments = 729.36 + (154.375)(2.375)+(44.347)(4.375)+(24.77)(4.75)
=1412.49 kNm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.50)(627.95)}{483.08} = 0.65 < 1.1 \quad (\ x\ ) \qquad FS_o = \frac{1412.49}{1766.21} = 0.80 < 1.2 \quad (\ x\ )$$

- **Block 3**



$\sum$ **Horizontal Force** = 483.08 + 85.02+23.12+22.88+7.52 = 621.62 kN/m

$\sum$ **Vertical Force** = 627.95 + 143.00+68.985+20.14= 860.08 kN/m

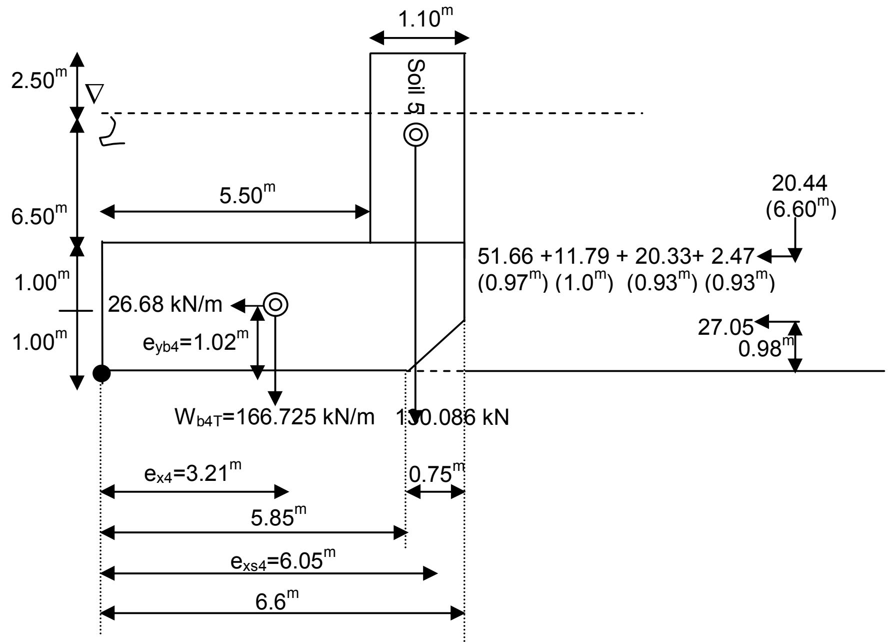
Overturning Moments = 1766.21 + (465.40)(2.00)+(41.33)(0.96) +(11.79)(1.00) +(28.44)(0.95)+(3.45)(0.95) +(23.12)(0.97) +(22.88)(1.00) +(7.52)(1.00)
= 2866.98 kNm/m

Resisting Moments = 1412.49 +(143.00)(2.75)+(68.985)(5.125) +(20.14)(5.50)
= 2270.06 kNm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.50)(860.08)}{621.62} = 0.69 < 1.1 \text{ (x)} \qquad FS_o = \frac{2270.06}{2866.98} = 0.79 < 1.2 \text{ (x)}$$

- **Block 4**



∑ **Horizontal Force** = 621.62 +86.25+27.05+26.68 = 761.60 kN/m

∑ **Vertical Force** = 860.08 +166.725+130.086+20.44 = 1177.33 kN/m

Overturning Moments =2866.98 + (621.62)(2.00)+ (51.66)(0.97) +(11.79)(1.00)+(20.33)(0.93)+(2.47)(0.93) +(27.05)(0.98) +(26.68)(1.02) = 4247.01 kNm/m

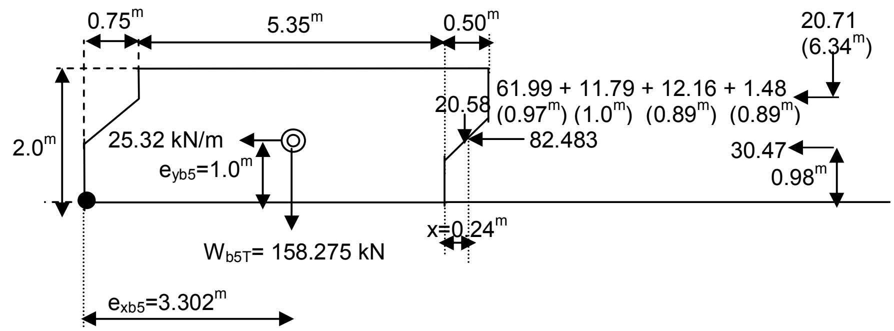
Resisting Moments = 2270.06 + (166.725)(3.21)+(130.086)(6.05) +(20.44)(6.60) = 3726.91 kNm/m

FACTOR of SLIDING and OVERTURNING

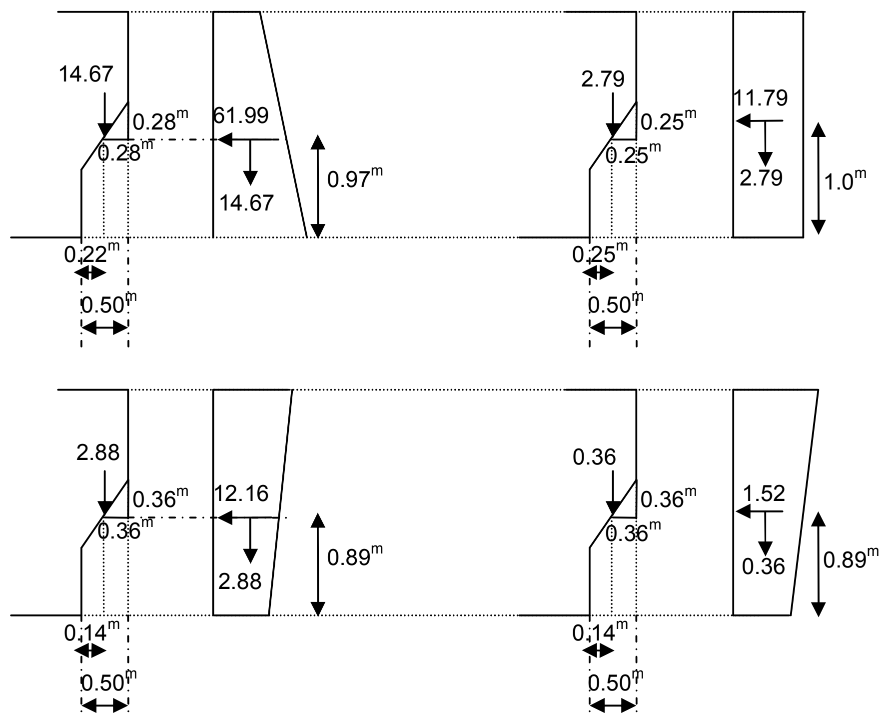
$$FS_s = \frac{(0.50)(1177.33)}{761.60} = 0.77 < 1.1 \quad (\text{ x })$$

$$FS_o = \frac{3726.91}{4247.01} = 0.88 \quad < 1.2 \quad (\text{ x })$$

- **Block 5**



In order to find the effect point of vertical force;



$$x = \frac{(14,67)(0,22)+(2,79)(0,25)+(2,88)(0,14)+(0,36)(0,14)}{20,70} = 0,21^m$$

∑ **Horizontal Force** = 761.60 +87.42+30.47+25.32 = 904.81 kN/m

∑ **Vertical Force** = 1177.33 +158.275+20.71=1356.32 kN/m

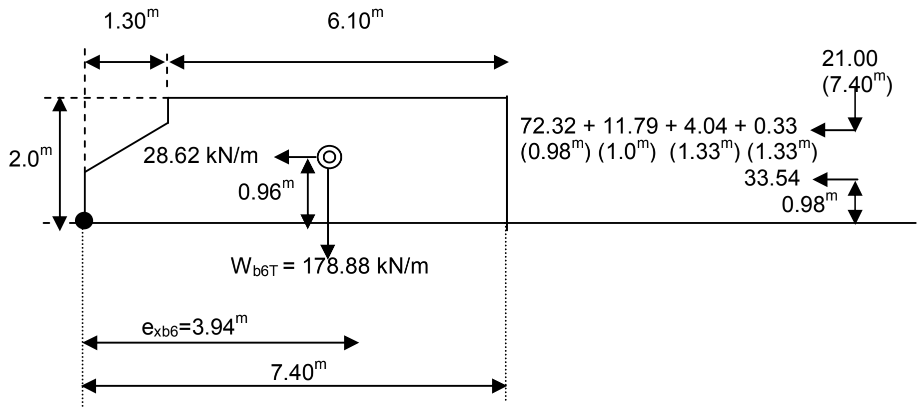
Overturning Moments = 4247.01 + (761.60)(2.00)+(61.99)(0.97) +(11.79)(1.00)+(12.16)(0.89)+(1.52)(0.89) +(30.47)(0.98) +(25.32)(1.00) = 5909.45 kNm/m

Resisting Moments = 3726.91 + (1177.33)(0.75)+(158.275)(3.302) +(20.71)(5.35+0.75+0.21)= 5263.21 kNm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.50)(1356.32)}{904.81} = 0.75 < 1.1\ (\ x\ ) \quad FS_o = \frac{5263.21}{5909.45} = 0.89 \quad < 1.2\ (\ x\ )$$

- **Block 6**



∑ **Horizontal Force=** 904.81 +88.63+33.54+28.62= 1055.60 kN/m

∑ **Vertical Force** = 1356.32 + 178.88+ 21.00 = 1556.20 kN/m

Overturning Moments = 5909.45 + (904.81)(2.00)+(72.32)(0.98) +(11.79)(1.00)+(4.04)(1.33)+(0.33)(1.33)+(33.54)(0.98) +(28.62)(0.96) = 7867.89 kNm/m

Resisting Moments = 5263.21 +(1356.32)(1.3)+(178.88)(3.94) +(21.00)(7.40)=7886.61 kNm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.60)(1556.20)}{1055.60} = 0.88 < 1.1(\ x\ ) \quad FS_o = \frac{788661}{7867.89} = 1.00 \quad < 1.2\ (\ x\ )$$

Tables 5.3-Table 5.10 show all the results for Approach 1 which are calculated above

Table 5.3: Static forces due to the soil and surcharge and exerting point of these forces

STATIC						
	H (m)	Earth (kN/m)	Surcharge (kN/m)	TOTAL (kN/m)	EP. Earth (m)	EP. Surcharge (m)
Crown	2	5.31	12.12	17.43	0.67	1.00
Block1	4.5	21.56	15.15	36.71	1.09	1.25
Block2	7	38.15	15.15	53.30	1.16	1.25
Block3	9	42.47	12.12	54.59	0.96	1.00
Block4	11	53.09	12.12	65.21	0.97	1.00
Block5	13	63.70	12.12	75.82	0.97	1.00
Block6	15	74.32	12.12	86.44	0.98	1.00

Table 5.4: Dynamic forces because of the soil and surcharge and exerting point of these forces

DYNAMIC							
BLOCK	H(m)	AREA(m^2)	P_{ae} (kN/m)	S (kN/m)	TOTAL (kN/m)	EP. of Earth (m)	EP. of Surc. (m)
Crown	2	13.65	58.44	7.10	65.54	0.98	0.98
Block1	4.5	14.33	61.33	7.45	68.78	1.21	1.21
Block2	7	11.28	48.28	5.87	54.15	1.19	1.19
Block3	9	6.83	29.23	3.55	32.78	0.95	0.95
Block4	11	4.88	20.89	2.54	23.43	0.93	0.93
Block5	13	2.92	12.50	1.52	14.02	0.89	0.89
Block6	15	0.97	4.15	0.50	4.65	1.33	1.33

Table 5.5: The total thrust and the components of total thrust

BLOCK	Total Trust (TT) (kN/m)	(TT)*cos(δ) (kN/m)	(TT)*cos(δ) (kN/m)
Crown	82.97	80.73	19.13
Block1	105.49	102.65	24.32
Block2	107.45	104.56	24.77
Block3	87.37	85.02	20.14
Block4	88.64	86.25	20.44
Block5	89.84	87.42	20.71
Block6	91.09	88.63	21.00

Table 5.6: The hydrodynamic forces and exerting point of these forces

		WATER		
BLOCK	**H (m)**	**P_{dw} (kN/m^2)**	**Force (kN/m)**	**EP of P_{dw} (m)**
Crown	0.000	0.000	0.000	**0.00**
Block1	2.000	7.00	7.00	**0.67**
Block2	4.500	10.50	21.88	**1.17**
Block3	6.500	12.62	23.12	**0.97**
Block4	8.500	14.43	27.05	**0.98**
Block5	10.500	16.04	30.47	**0.98**
Block6	12.500	17.50	33.54	**0.99**

Table 5.7: The inertia forces and exerting point of these forces

BLOCK	W*k_h (kN)	C (m)	SOIL EFFECT	W*k_h (kN)	CG (m)
Crown	25.76	0.90	**Soil 1-5.block1**	**26.304**	1.09
Block1	26.75	1.44	**Soil 3-5.block2**	**22.21**	1.25
Block2	24.70	1.25	**Soil 4-5. block 3**	**15.80**	1.25
Block3	22.88	1.00	**Soil 5.block 4**	**7.52**	1.00
Block4	26.68	1.02			
Block5	25.32	0.99			
Block6	28.62	0.96			

Table 5.8: The weight of the blocks and soils (buoyancy effect)

WEIGHT OF THE BLOCKS & SOIL							
	a(m)	b(m)	**W (kN)**	**Total**	**Block**	**CG(ex)**	**CG(ey)**
Crown-a	2.800	2	128.800			**1.40**	**1.00**
Crown-b	1.400	1	32.200			**1.82**	**0.90**
Soil 1	1.400	1	18.396	**179.394**	**Crown**	**3.50**	**1.50**
Block1	**type 2**		167.156			**1.76**	**1.44**
Soil 2	0.550	2	14.453	**362.222**	**Block 1**	**3.73**	**1.00**
Block2	4.750	2.5	154.375			**2.38**	**1.25**
Soil 3	0.750	4.5	44.344	**560.941**	**Block 2**	**4.38**	**1.25**
Block3	5.500	2	143.000			**2.75**	**1.00**
Soil 4	0.750	7	68.979	**772.920**	**Block 3**	**5.13**	**1.00**
Block 4	**type 3**		166.725			**3.21**	**1.02**
Soil 5	1.100	9	130.075	**1069.720**	**Block 4**	**6.05**	**1.00**
Block 5	**type 4**		158.275	**1227.995**	**Block 5**	**3.30**	**0.99**
Block 6	**type 5**		178.880	**1406.875**	**Block 6**	**3.94**	**0.96**

Table 5.9: Total horizontal and vertical forces, overturning and resisting moments

Block	Total HF	Total VF	Overturning Moment	Resistant Moment
Crown	157.54	198.53	188.44	437.75
Block 1	316.14	404.46	775.54	734.17
Block 2	483.08	627.95	1766.21	1412.49
Block 3	621.62	860.08	2866.98	2270.06
Block 4	761.60	1177.33	4247.01	3726.91
Block 5	904.81	1356.32	5909.45	5263.21
Block 6	1055.60	1556.20	7867.89	7886.61

Table 5.10: Factor of safety values against sliding and overturning

Block	FS_s	FS_o
Crown	0.63	2.32
Block 1	0.64	0.95
Block 2	0.65	0.80
Block 3	0.69	0.79
Block 4	0.77	0.88
Block 5	0.75	0.89
Block 6	0.88	1.00

APPROACH 1

The parameters used in this case defined below:

1. Peak Ground Acceleration (a_{max}) and Seismic Coefficient (k_h)

(a_{max}) and (k_h) are calculated according to TSDC-CRA, 2007;(Eq.4.16, Eq.4.18)

2. Static Pressure Distribution (earth and surcharge)

Triangular distribution is accepted for static earth pressure and rectangular distribution is used for seismic surcharge pressure.

3. Modified Seismic Coefficient

Modified seismic coefficient (k_h^{I}) is used during the dynamic calculations (Eq.4.22).

4. Seismic Pressure Distribution (earth and surcharge)

Inversed triangular distribution is accepted both for the dynamic earth pressure and dynamic surcharge pressure.

5. Forces (Figure 5.8)

5.1 Horizontal Forces

5.1.1 Static Forces

5.1.1.1 Static earth forces (Eq.4.7 - Eq.4.10) (Figure 5.7)

5.1.1.2 Static surcharge forces (Eq.4.11 - Eq.4.12)

5.1.1.3 Hydrostatic forces

5.1.2. Dynamic Forces

5.1.2.1 Dynamic earth forces (Eq.4.13 - Eq.4.14).

5.1.2.2 Dynamic surcharge forces (Eq.4.28 - Eq.4.32)

5.1.2.3 Hydrodynamic forces (Eq.4.39 - Eq.4.40)

5.1.2.4 Inertia forces (Eq.4.41)

5.1.2.4.1 Unit Weight of Soil

Equivalent unit weight (γ_{eq}) is used both for dry and submerged soil (Eq.4.25).

5.1.2.4.2 Weight of the Blocks

Dry weight is used above still water level (SWL) and submerged weight is used below SWL

5.1.2.4.3 Modified Seismic Coefficient

$(k_h^{\prime})$ are used for both dry and submerged soil.

5.2 Vertical Forces

Dry weight of the blocks above SWL and submerged weight of the blocks below SWL are taken into consideration. The weight of the soil is calculated according to (γ_{eq}).

6. Friction Coefficient

The friction coefficient between the blocks is taken as 0.5; the friction coefficient between the block and the foundation is taken as 0.6.

7. Factor of Safety

The factor of safety values against sliding (FS_s) and overturning (FS_o) is computed (Table 5.11). The recommended values against sliding and overturning are FS_s =1.1 and FS_o =1.2 respectively.

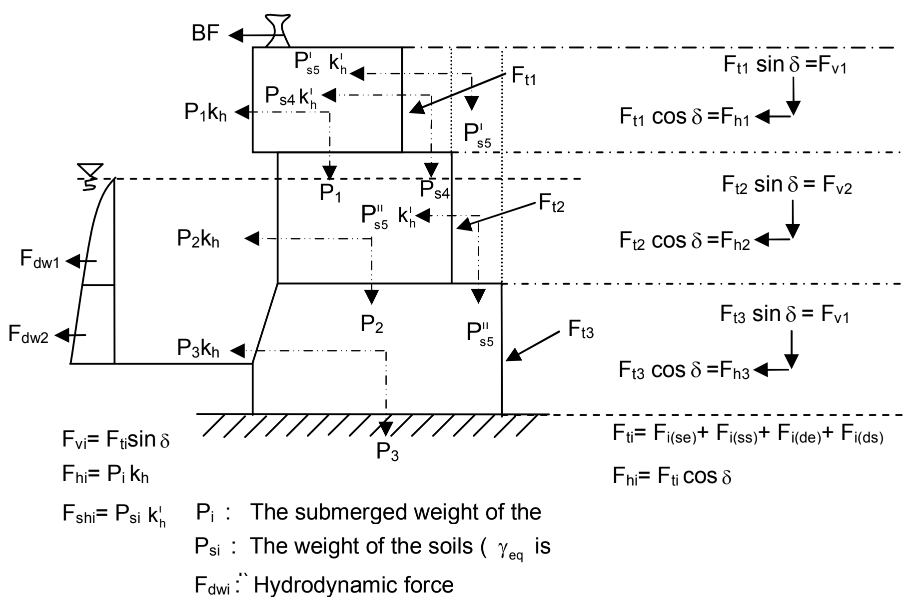


Figure 5.8: Forces acting on block type quay wall for Approach 1

Table 5.11: Factor of safety values for Approach 1

	Approach 1	
Block	**FS_s**	**Fs_o**
Crown	**0.63**	2.32
Block 1	**0.64**	0.95
Block 2	**0.65**	0.80
Block 3	0.69	**0.79**
Block 4	0.77	0.88
Block 5	0.75	0.89
Block 6	0.88	1.00

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges between 0.63 to 0.88 from Crown Wall to Block 6. Therefore considering sliding **Crown Wall** is the most critical. However, **Block 1** and **Block 2** have almost the same FS_s. In the design of crown wall sliding will govern.

3. FS_o ranges between 2.32 to 0.79. Therefore considering overturning, crown wall is safe but **Block 2** and **Block 3** are more critical.

5.2 APPROACH 2

The parameters used in this case defined below:

1. Peak Ground Acceleration (a_{max}) and Seismic Coefficient (k_h)

(a_{max}) and (k_h) are calculated according to TSDC-CRA, 2007; (Eq.4.16 - Eq. 4.18)

2. Static Pressure Distribution (earth and surcharge)

Triangular distribution is accepted for static earth pressure and rectangular distribution is used for seismic surcharge pressure.

3. Modified Seismic Coefficient

Modified seismic coefficient (k_h^{I}) is used during the dynamic calculations (Eq.4.22).

4. Seismic Pressure Distribution (earth and surcharge)

Inversed triangular distribution is accepted both for the dynamic earth pressure and dynamic surcharge pressure.

5. Forces (Figure 5.9)

5.1 Horizontal Forces

5.1.1 Static Forces

5.1.1.1 Static earth forces (Eq.4.7 - Eq.4.10)

5.1.1.2 Static surcharge forces (Eq.4.11 - Eq.4.12)

5.1.1.3 Hydrostatic forces

5.1.2 Dynamic Forces

5.1.2.1 Dynamic earth forces (Eq.4.13 - Eq.4.14)

5.1.2.2 Dynamic surcharge forces (Eq.4.28 - Eq.4.32)

5.1.2.3 Hydrodynamic forces (Eq.4.39 - Eq.4.40)

5.1.2.4 Inertia forces (Eq.4.41)

5.1.2.4.1 Unit Weight of Soil

Equivalent unit weight (γ_{eq}) is used both for dry and submerged soil (Eq.4.25).

5.1.2.4.2 Weight of the Blocks

Dry weight is used both above still water level (SWL) and below SWL.

5.1.2.4.3 Modified Seismic Coefficient

Both for dry and for submerged soil, (k_h^{I}) is used.

5.2 Vertical Forces

Dry weight of the blocks above SWL and submerged weight of the blocks below SWL are taken into consideration. The weight of the soil is calculated according to (γ_{eq}).

6. Friction Coefficient

The friction coefficient between the blocks is taken as 0.5, the friction coefficient between the block and the foundation is taken as 0.6.

7. Factor of Safety

The factor of safety values against sliding (FS_s) and overturning (FS_o) is computed (Table 5.12). The recommended values against sliding and overturning are FS_s =1.1 and FS_o =1.2 respectively.

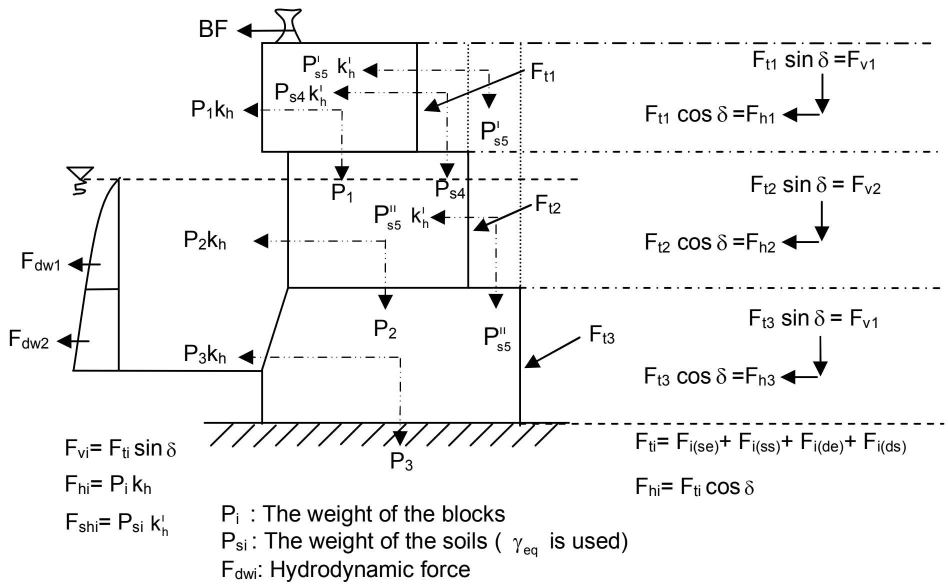


Figure 5.9: Forces acting on block type quay wall for Approach 2

Table 5.12: Factor of safety values for Approach 2

	Approach 2	
Block	$\mathbf{FS_s}$	$\mathbf{Fs_o}$
Crown	**0.63**	2.32
Block 1	**0.62**	0.93
Block 2	**0.61**	**0.77**
Block 3	**0.64**	**0.75**
Block 4	0.71	0.82
Block 5	0.68	0.83
Block 6	0.80	0.93

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges between 0.61 to 0.80. The most critical block for sliding is **Block 2**.

However, **Crown Wall**, **Block 1** and **Block 4** have almost the same FS_s.

3. FS_o ranges between 2.32 to 0.75. Considering overturning, **Block 2** and **Block 3** are more critical.

5.3 APPROACH 3

The parameters used in this case defined below:

1. Peak Ground Acceleration (a_{max})

(a_{max}) and (k_h) are calculated according to TSDC-CRA, 2007;(Eq.4.16 - Eq.4.18)

2. Static Pressure Distribution (earth and surcharge)

Triangular distribution is accepted for static earth pressure and rectangular distribution is used for seismic surcharge pressure.

3. Seismic Coefficient (k_h)

Seismic coefficient (k_h) is used during the dynamic calculations (Eq.4.18) (k_h) is calculated according to TSDC-CRA.2007.

4. Seismic Pressure Distribution (earth and surcharge)

Inversed triangular distribution is accepted both for the dynamic earth pressure and dynamic surcharge pressure.

5. Forces (Figure 5.10)

5.1 Horizontal Forces

5.1.1 Static Forces

5.1.1.1 Static earth forces (Eq.4.7 - Eq.4.10)

5.1.1.2 Static surcharge forces (Eq.4.11 - Eq.4.12)

5.1.1.3 Hydrostatic forces (there is no water level differences between both sides so hydrostatic force is not taken into consideration in this study)

5.1.2 Dynamic Forces

5.1.2.1 Dynamic earth forces (Eq.4.13 - Eq.4.14)

5.1.2.2 Dynamic surcharge forces (Eq.4.28 - Eq.4.32)

5.1.2.3 Hydrodynamic forces (Eq.4.39 - Eq.4.40)

5.1.2.4 Inertia forces (Eq.4.41)

5.1.2.4.1 Unit Weight of Soil

Equivalent unit weight (γ_{eq}) is used both for dry and submerged soil (Eq.4.25).

5.1.2.4.2 Weight of the Blocks

Dry weight is used above still water level (SWL) and submerged weight is used below SWL

5.1.2.4.3 Seismic Coefficient

Both for dry and for submerged soil, (k_h) is used.

5.2 Vertical Forces

Dry weight of the blocks above SWL and submerged weight of the blocks below SWL are taken into consideration. The weight of the soil is calculated according to (γ_{eq}).

6. Friction Coefficient

The friction coefficient between the blocks is taken as 0.5, the friction coefficient between the block and the foundation is taken as 0.6.

7. Factor of Safety

The factor of safety values against sliding (FS_s) and overturning (FS_o) is computed (Table 5.13). The recommended values against sliding and overturning are FS_s =1.1 and FS_o =1.2 respectively.

According to Table 5.13;

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges between 0.80 to 1.00. The most critical block for sliding is **Crown Wall**. However, **Block 1**. **Block 2** and **Block 3** have almost the same FS_s.

3. FS_o ranges between 2.74 to 0.97. Considering overturning, **Block 2** and **Block 3** are more critical.

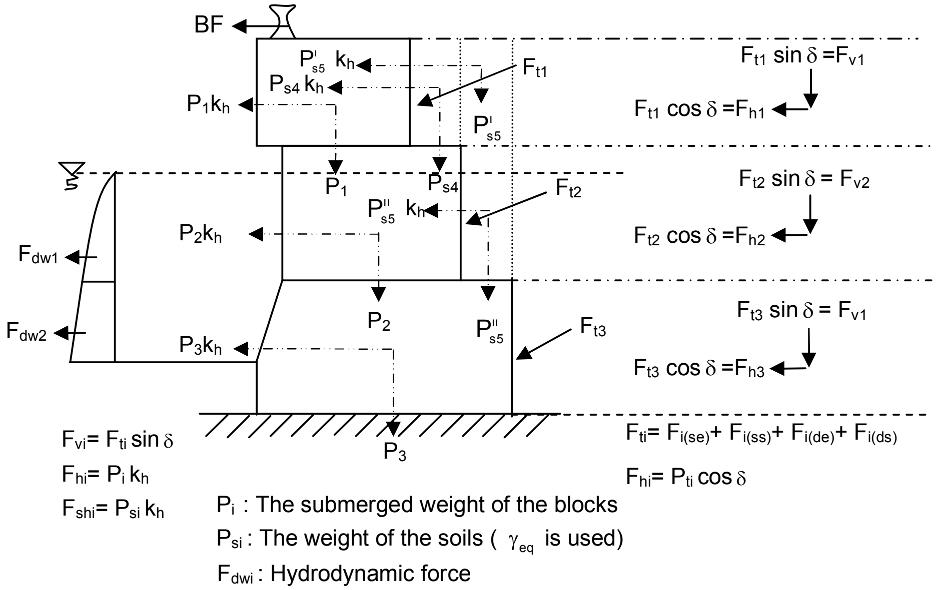


Figure 5.10: Forces acting on block type quay wall for Approach 3

Table 5.13: Factor of safety values for Approach 3

	Approach 3	
Block	**FS_s**	**Fs_o**
Crown	**0.80**	2.74
Block 1	**0.82**	1.15
Block 2	**0.81**	**0.98**
Block 3	**0.84**	**0.97**
Block 4	0.92	1.08
Block 5	0.87	1.08
Block 6	1.00	1.20

5.4 APPROACH 4

The parameters used in this case defined below:

1. Peak Ground Acceleration (a_{max})

(a_{max}) is calculated according to TSDC-CRA, 2007; (Eq. 4.16 - Eq. 4.18)

2. Static Pressure Distribution (earth and surcharge)

Triangular distribution is accepted for static earth pressure and rectangular distribution is used for seismic surcharge pressure.

3. Seismic Coefficient

Seismic coefficient (k_h) is used during the dynamic calculations (Eq.4.18).

(k_h) is calculated according to TSDC-CRA.2007.

4. Seismic Pressure Distribution (earth and surcharge)

Inversed triangular distribution is accepted both for the dynamic earth pressure and dynamic surcharge pressure.

5. Forces (Figure 5.11)

5.1 Horizontal Forces

5.1.1 Static Forces

5.1.1.1 Static earth forces (Eq.4.7 - Eq.4.10)

5.1.1.2 Static surcharge forces (Eq.4.11 - Eq.4.12)

5.1.1.3 Hydrostatic forces (there is no water level differences between both sides so hydrostatic force is not taken into consideration in this study)

5.1.2.Dynamic Forces

5.1.2.1 Dynamic earth forces (Eq.4.13 - Eq.4.14)

5.1.2.2 Dynamic surcharge forces (Eq.4.28- Eq.4.32)

5.1.2.3 Hydrodynamic forces (Eq.4.39- Eq.4.40)

5.1.2.4 Inertia forces (Eq.4.41)

5.1.2.4.1 Unit Weight of Soil

Equivalent unit weight (γ_{eq}) is used both for dry and submerged soil (Eq.4.25).

5.1.2.4.2 Weight of the Blocks

Dry weight is used both above still water level (SWL) and below SWL.

5.1.2.4.3 Seismic Coefficient

Both for dry and for submerged soil, (k_h) is used.

5.2. Vertical Forces

Dry weight of the blocks above SWL and submerged weight of the blocks below SWL are taken into consideration. The weight of the soil is calculated according to (γ_{eq}).

6. Friction Coefficient

The friction coefficient between the blocks is taken as 0.5, the friction coefficient between the block and the foundation is taken as 0.6.

7. Factor of Safety

The factor of safety values against sliding (FS_s) and overturning (FS_o) is computed (Table 5.14). The recommended values against sliding and overturning are FS_s =1.1 and FS_o =1.2 respectively.

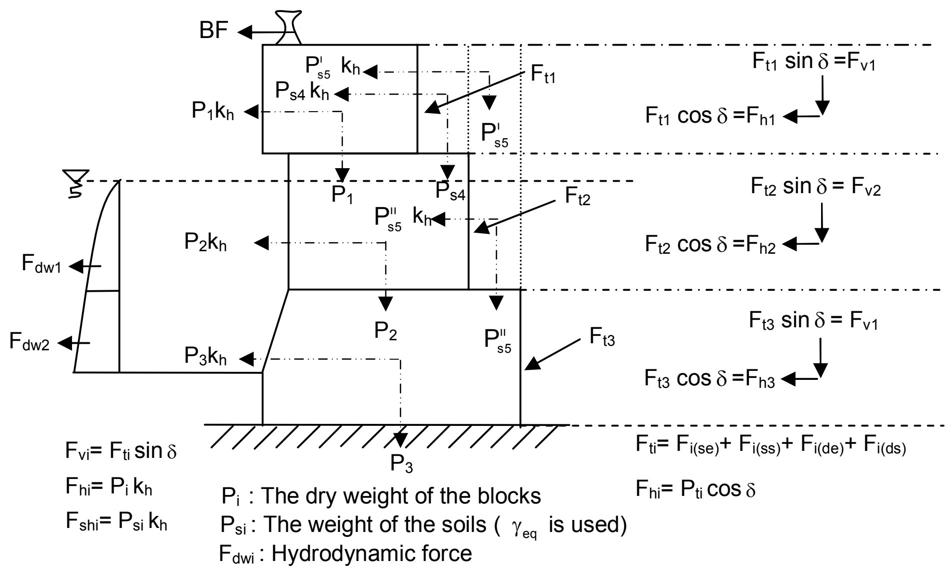


Figure 5.11: Forces acting on block type quay wall for Approach 4

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges between 0.74 to 0.89. The most critical block for sliding is **Block 2**.
3. FS_o ranges between 2.32 to 0.75. Considering overturning, **Block 2** and **Block 3** are more critical.

Table 5.14: Factor of safety values for Approach 4

	Approach 4	
Block	**FS_s**	**Fs_o**
Crown	0.80	2.72
Block 1	0.77	1.11
Block 2	**0.74**	**0.93**
Block 3	0.76	**0.91**
Block 4	0.82	0.99
Block 5	0.78	0.99
Block 6	0.89	1.09

In summary;

- For **Approach 1** and **Approach 3**. the most vulnerable block is **crown wall** with respect to factor of safety against **sliding** (FS_s) (Table 5.11 and Table 5.13)

CrownWall	Approach1	Approach 3
FS_s	0.63	0.80

- For **Approach 2** and for **Approach 4**. the most vulnerable block is **Block 2** with respect to factor of safety against **sliding** (FS_s) (Table 5.12 and Table 5.14)

Block 2	Approach 2	Approach 4
FS_s	0.61	0.74

- For all the approaches, namely, **Approach 1**, **Approach 2, Approach 3** and **Approach 4,** the most vulnerable block is **Block 3** with respect to factor of safety against **overturning** (FS_o) (Table 5.11. Table 5.12. Table 5.13 and Table 5.14).

Block 3	Approach1	Approach2	Approach 3	Approach4
FS_o	0.79	0.75	0.97	0.91

In order to the most critical approaches **Approach 1** and **Approach 2** are compared. Approach 1 uses dry weight and submerged weight, according condition of the blocks either above still water level (SWL) or below SWL to compute inertia forces. However, Approach 2 uses dry weight not only for the blocks above SWL but also for the blocks below SWL. Thus, Approach 2 gives more critical results.

If factor of safety values of Approach 1 and Approach 2 are compared. as dry weight of blocks are greater than submerged weight of blocks. **Approach 2**, in which dry weights are used, gives **more critical** results **than Approach 1**, in which both submerged and dry weights are used. Considering the sliding **Crown Wall** is in risk for **Approach 1**, on the other hand, **Block 2** is critical for Approach 2. And **Block 3** is critical for both approaches according to overturning condition (Table 5.11 and Table 5.12)

Approach	**Critical Block**	
	FS_s	**FS_o**
Approach 1	Crown Wall	Block 3
Approach 2	Block 2	Block 3

New design approach to conventional method is also compared with the TSDC-CRA, 2007. A case study is applied for Derince Port block type quay wall.

In addition to these discussions by using the different kinds of soils stability analysis are made for each block.

- If **soil class C (Table 4.1)** is used, k_h is calculated as 0.22 (k_h =0.22) (TSDC-CRA, 2007) (Eq.4.19). During calculation of inertia forces, submerged unit weights of the blocks are taken into consideration. According to these values factor of safety against sliding and overturning are computed (Table 5.15)

As it seen from Table 5.15;

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.
2. FS_s ranges between 0.65 to 0.83. The most critical block for sliding is Crown wall and Block 1 and Block 2.

3. FS_o ranges between 2.35 to 0.79. Considering overturning, **Block 2** and **Block 3** are more critical.

Table 5.15: Factor of safety against sliding and overturning for soil C (k_h- W_{sub})

k_h - $\mathbf{W_{sub}}$	$\mathbf{FS_s}$	$\mathbf{FS_O}$
Crown Wall	0,65	2,35
Block 1	0,65	0,95
Block 2	0,65	0,80
Block 3	0,68	0,79
Block 4	0,75	0,87
Block 5	0,71	0,87
Block 6	0,83	0,97

Soil Class A is used for Derince Port and the results are discussed above. In addition if different kinds of soils are used, the stability analyses are made for Derince Port Quay Wall and the results are calculated.

- If **soil class C (Table 4.1)** is used, k_h is calculated as, 0.22 (k_h =0.22) and k_h^{I} is calculated as 0.36 (k_h^{I}=0.36) (TSDC-CRA, 2007) (Eq.4.16 - Eq.4.18). During calculations of inertia forces, submerged unit weights of the blocks are taken into consideration. According to these values factor of safety against sliding and overturning are computed (Table 5.16).

As it seen from Table 5.16;

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges from 0.49 to 0.71. The most critical block for sliding is Crown wall and Block 1. However, **Crown Wall**, **Block 1** and **Block 2** have almost the same FS_s.

3. FS_o ranges from 1.95 and 0.62. Considering overturning, **Block 2** and **Block 3** are more critical.

Table 5.16: Factor of safety against sliding and overturning for soil C (k_h^I -W_{sub})

k_h^I - W_{sub}	FS_s	FS_O
Crown Wall	0.49	1.95
Block 1	0.49	0.77
Block 2	0.50	0.64
Block 3	0.54	0.62
Block 4	0.61	0.68
Block 5	0.59	0.70
Block 6	0.71	0.79

- If soil class D is used, k_h is calculated as 0.24 (k_h =0.22) (TSDC-CRA, 2007) (Eq.4.19). During calculation of inertia forces, submerged unit weights of the blocks are taken into consideration. According to these values factor of safety against sliding and overturning are computed (Table 5.17).

Table 5.17: Factor of safety against sliding and overturning for soil D (k_h- W_{sub})

k_h - W_{sub}	FS_s	FS_O
Crown Wall	0,61	2,25
Block 1	0,61	0,90
Block 2	0,61	0,76
Block 3	0,64	0,74
Block 4	0,70	0,82
Block 5	0,67	0,82
Block 6	0,78	0,92

As it seen from Table 5.17;

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges between 0.61 to 0.78. The most critical block for sliding is Crown wall and Block 1 and Block 2.

3. FS_o ranges between 2.25 to 0.74. Considering overturning, **Block 2** and **Block 3** are more critical.

- If soil class D is used, k_h is calculated as, 0.24 (k_h =0.24) and $k_h^{\prime}$ is calculated as 0.39 ($k_h^{\prime}$=0.39) (TSDC-CRA, 2007) (Eq.4.16 - Eq.4.18). During calculation of inertia forces, submerged unit weights of the blocks are taken into consideration. According to these values factor of safety against sliding and overturning are computed (Table 5.18)

Table 5.18: Factor of safety against sliding and overturning soil D ($k_h^{\prime}$ - W_{sub})

$k_h^{\prime}$ - W_{sub}	FS_s	FS_O
Crown Wall	0,46	1,86
Block 1	0,46	0,73
Block 2	0,47	0,60
Block 3	0,50	0,58
Block 4	0,57	0,64
Block 5	0,56	0,65
Block 6	0,67	0,73

As it seen from Table 5.18;

1. FS_o is larger than FS_s. Therefore it can be stated that sliding is more critical for the stability of the blocks.

2. FS_s ranges between 0.46 to 0.67. The most critical block for sliding is Crown wall and Block 1.
However, **Crown Wall**, **Block 1** and **Block 2** have almost the same FS_s.

3. FS_o ranges between 1.86 to 0.58. Considering overturning, **Block 2** and **Block 3** are more critical.

In conclusion; if k_h parameter increases the FS values decrease resulting in block sliding. The above given computations show the importance of decision on correct k_h parameter which is reflecting soil conditions.

5.5 Turkish Seismic Design codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007)

Table 5.19 shows the necessary parameters to calculate the forces acting on block type quay wall.

Table 5.19: Necessary parameters to calculate the forces acting on block type quay wall

Φ (deg)	**δ(deg)**	**β(deg)**	θ **(deg)**	**H (m)**	H_{sub} **(m)**	**q (kN/m²)**
40	13.33	0	0	15	12.5	30

Bollard (kN)	**Cronman Length (m)**	γ_{conc} **(kN/m³)**	γ_{b-conc} **(kN/m³)**	γ_{dry} **(kN/m³)**	γ_{sub} **(kN/m³)**	γ_{w} **(kN/m³)**
800	16.16	23	13.33	18	11	10

1. Coordinates of Derince port is 29.80 longitude and 40.8 latitude for %50 exceedance probability $S_s = 0.761$ (Erdik et.all., 2006)

2. The soil classification is chosen as A and the short period soil coefficient (F_a) is found by using the Table 4.1.

$S_s = 0.76 \Rightarrow F_a = 0.8$

3. $S_{MS} = 0.8 \times 0.76 = 0.61$ (Eq. 4.15)

4. $A_{10} = 0.4 \times 0.61 = 0.243$ (Eq. 4.16)

5. $k_h = (2/3)(0.243) = 0.16$ (Eq. 4.18)

6. Static case; (Eq. 4.7)

$$K_A = \frac{\cos^2(40-0)}{\cos^2(0)\cos(0+13.33)\left[1+\sqrt{\frac{\sin(40+13.33)\sin(40-0)}{\cos(13.33+0)\cos(0-0)}}\right]^2} \textbf{=0.202}$$

It is assumed that soil improvement techniques are used for this site so the internal friction of soil is taken as ϕ=40°. And, the friction angle between the soil and block is taken as $\left(\delta=\frac{\phi}{3}=13.33\right)$. As it is seen from the Figure 4.7, angle between the back of the wall and the vertical plane (θ) and the inclined angle with the horizontal (β) is taken 0 for this study.

6.1. Active earth pressure;

$$p_{ai} = K_{ai}\left[\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^{i}(\gamma_{bj} h_j) + \frac{q_o \cos\alpha}{\cos(\alpha-\beta)}\right]\cos\alpha \quad (5.1)$$

where p_{ai} is the active pressure for soil class i^{th}, $K_{ai.d}$ is active pressure coefficient for soil class i^{th}, γ_j is dry unit weight of soil for soil class j^{th}, γ_{bj} is submerged unit weight of soil for soil class j^{th}, h_j is thickness of the soil class j^{th}, ND is number of the dry soil class according to ground level, q_o is surcharge load, α (or θ) is inclination of the ground surface behind the wall, β is inclined angle with the horizontal (α (or θ) and β are zero)

This formulation is divided in to two parts; 1. active earth pressure (p_{ai}). 2.surcharge ($p_{ai.s}$).

1. Active Earth Pressure; (Figure 5.12)

$p_{ac1} = 0.202(18\times2) = 7.272$ kN/m^2

$p_{ac} = 0.202(18\times2.5) = 9.09$ kN/m^2

$p_{a1} = 0.202(18\times2.5+11\times2) = 13.534$ kN/m^2

$p_{a2} = 0.202(18\times2.5+11\times2+11\times2.5) = 19.089$ kN/m^2

$p_{a3} = 0.202(18\times2.5+11\times2+11\times2.5+11\times2) = 23.533$ kN/m^2

$p_{a4} = 0.202(18 \times 2.5 + 11 \times 2 + 11 \times 2.5 + 11 \times 2 + 11 \times 2) = 27.977 \text{ kN/m}^2$

$p_{a5} = 0.202(18 \times 2.5 + 11 \times 2 + 11 \times 2.5 + 11 \times 2 + 11 \times 2 + 11 \times 2) = 32.421 \text{ kN/m}^2$

$p_{a6} = 0.202(18 \times 2.5 + 11 \times 2 + 11 \times 2.5 + 11 \times 2 + 11 \times 2 + 11 \times 2 + 11 \times 2)$
$= 36.875 \text{ kN/m}^2$

2. Surcharge (Figure 5.13)

$p_{ac1,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{ac,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{a1,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{a2,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{a3,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{a4,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{a5,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

$p_{a6,s} = 0.202 \times 30 = 6.06 \text{ kN/m}^2$

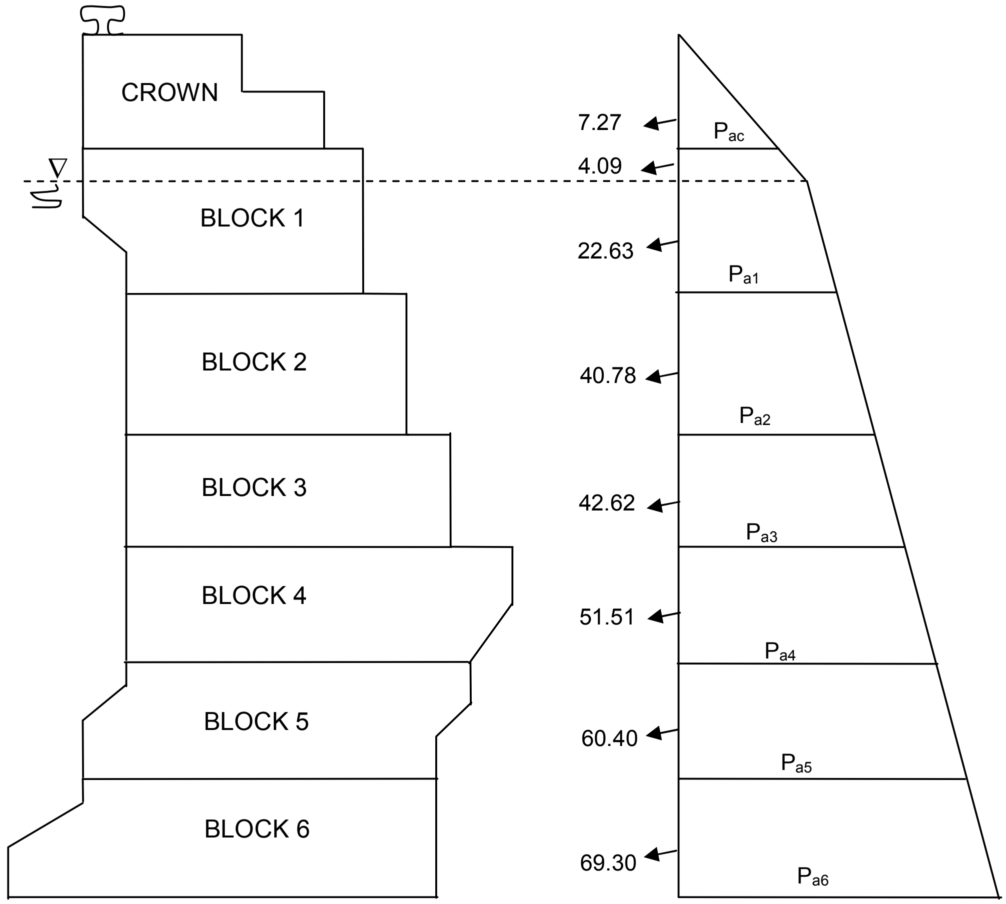


Figure 5.12: The soil effect on the block type quay wall for static case

7. Dynamic Case;

7.1.For dry part seismic inertia angle;

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

$$\psi = \arctan(0.162) = 9.09^o$$

For dry part dynamic active earth pressure coefficient; **(Eq. 4.13)**

$$K_{AE} = \frac{(1-0)\cos^2(40-0-9.09)}{\cos(9.09)\cos^2(0)\cos(0+13.33+9.09)\left[1+\sqrt{\frac{\sin(40+13.33)\sin(40-9.09)}{\cos(13.33+0+9.09)\cos(0-0)}}\right]^2}$$

$$K_{AE} = 0.290$$

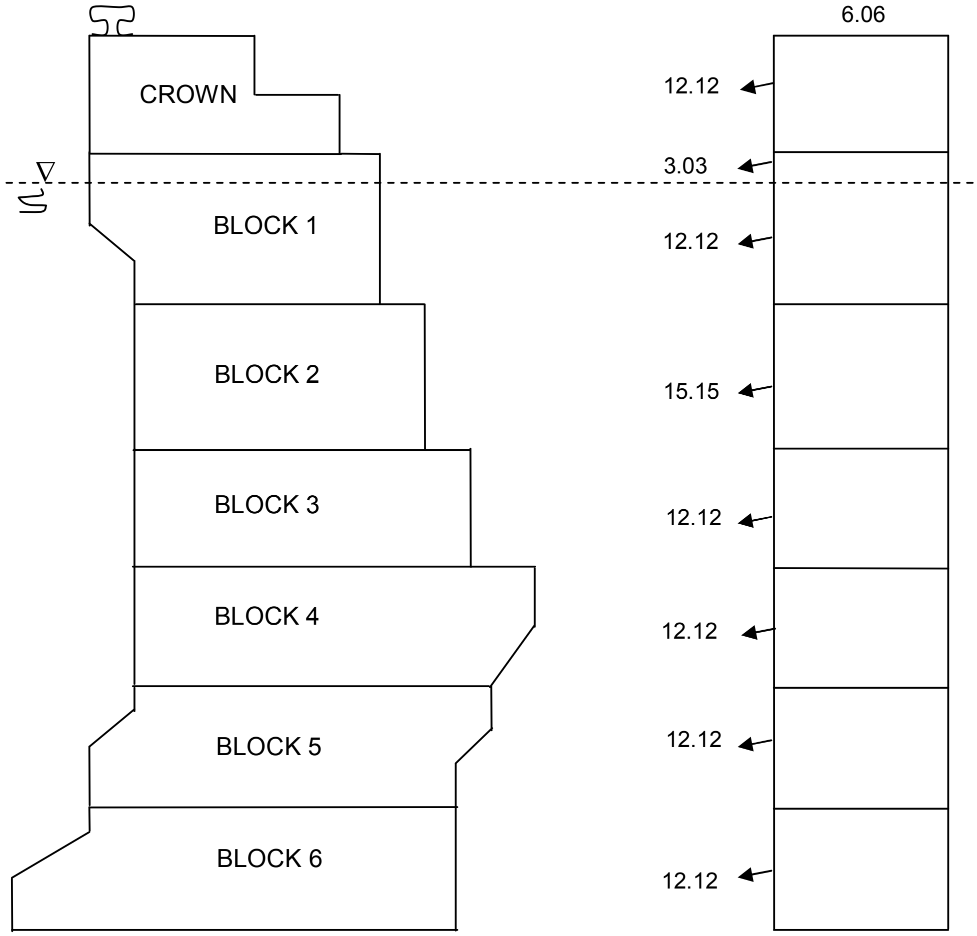


Figure 5.13: The surcharge effect on the block type quay wall for static case

For dry part dynamic coefficient; (Eq. 4.24)

$$K_{d(dry)} = 0.290 - 0.202 = 0.088$$

7.2.For saturated part:

For saturated part modified seismic coefficient; (Eq. 4.26)

$$k_h^{I} = \frac{18\times 2.5 + 21\times 2 + 21\times 2.5 + 21\times 2 + 21\times 2 + 21\times 2 + 21\times 2 + 15}{18\times 2.5 + 11\times 2 + 11\times 2.5 + 11\times 2 + 11\times 2 + 11\times 2 + 11\times 2 + 15}\times 0.16$$

k_h^{I} =0.26

For saturated part seismic inertia angle; (Eq. 4.27)

$$\psi = \arctan(k_h^{I})$$

$\psi = \arctan(0.26) \quad \Rightarrow \quad \psi = 14.57^{o}$

For saturated part dynamic active earth pressure coefficient;

$$K_{AE} = \frac{(1-0)\cos^2(40-0-14.57)}{\cos(14.57)\cos^2(0)\cos(0+13.33/2+14.57)\left[1+\sqrt{\dfrac{\sin(40+13.33/2)\sin(40-14.57)}{\cos(13.33/2+0+14.57)\cos(0-0)}}\right]^2}$$

$K_{AE} = 0.363$

For dry part dynamic coefficient;

$K_{d(sat)} = 0.363 - 0.202 = 0.161$

7.3 Dynamic Earth Pressure (Figure 5.14)

$$p_{ai,d} = K_{ai,d}\left[\sum_{j=1}^{ND}(\gamma_j\, h_j) + \sum_{j=ND+1}^{i}(\gamma_{bj}\, h_j) + \frac{q_o \cos\alpha}{\cos(\alpha-\beta)}\right]\cos\alpha \qquad (5.2)$$

where $p_{ai.d}$ is the dynamic active pressure for soil class i^{th}, $K_{ai.d}$ is dynamic active pressure coefficient for soil class i^{th}, γ_j is dry unit weight of soil for soil class j^{th}, γ_{bj} is submerged unit weight of soil for soil class j^{th}, h_j is thickness of the soil class j^{th}, ND is number of the dry soil class according to ground level. q_o is surcharge load (q/2 is taken), α is inclination of the ground surface behind the wall, β is inclined angle with the horizontal (α and β are zero).

This formulation is divided in to two parts; 1. active dynamic earth pressure ($p_{ai.d}$), 2. surcharge ($p_{ai.ds}$).

$p_{a,d} = 0.088(18\times 2) = 3.17$ kN/m^2

$p_{ac,d} = 0.088(18\times 2.5) = 3.96$ kN/m^2

$p_{a1,d} = 0.088(18\times 2.5) + 0.161(11\times 2) = 7.50$ kN/m^2

$p_{a2,d} = 0.088(18\times 2.5) + 0.161(11\times 2 + 11\times 2.5) = 11.93$ kN/m^2

$p_{a3,d} = 0.088(18\times 2.5) + 0,161(11\times 2 + 11\times 2.5 + 11\times 2) = 15.47$ kN/m^2

$p_{a4,d} = 0.088(18\times 2.5) + 0.161(11\times 2 + 11\times 2.5 + 11\times 2 + 11\times 2) = 19.01$ kN/m^2

$p_{a5,d} = 0.088(18 \times 2.5) + 0.161(11 \times 2 + 11 \times 2.5 + 11 \times 2 + 11 \times 2 + 11 \times 2)$
$= 22.55 \text{ kN/m}^2$

$p_{a6,d} = 0.088(18 \times 2.5) + 0.161(11 \times 2 + 11 \times 2.5 + 11 \times 2 + 11 \times 2 + 11 \times 2$
$+ 11 \times 2) = 26.10 \text{ kN/m}^2$

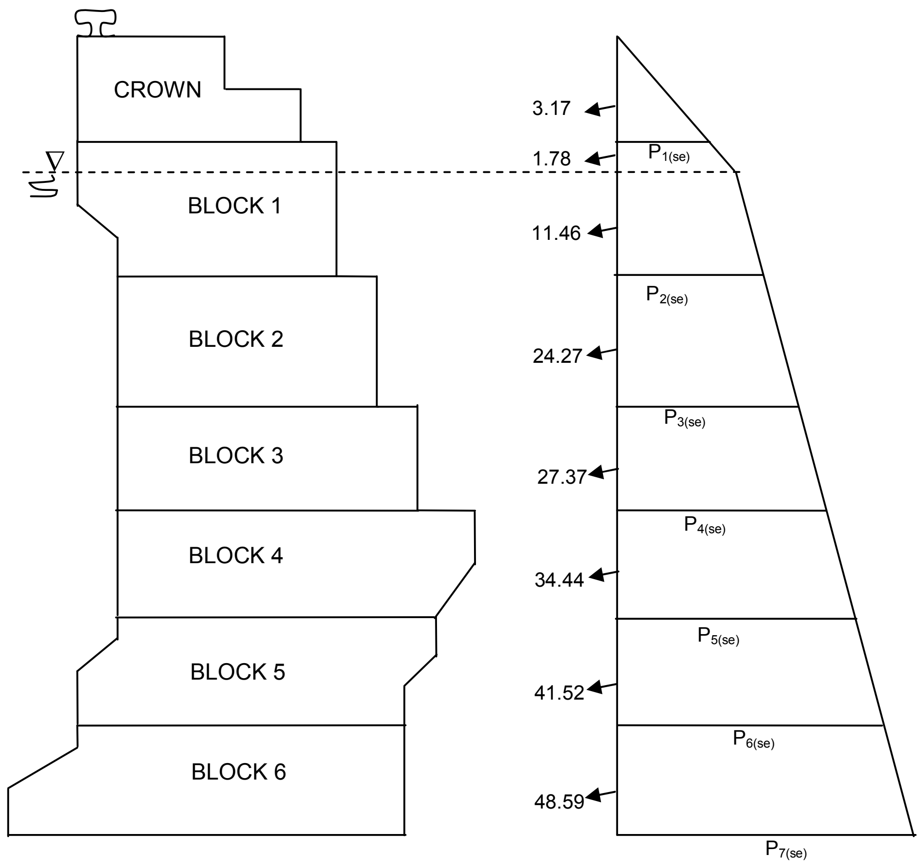


Figure 5.14: The soil effect on the block type quay wall for dynamic case

7.4 Dynamic Surcharge Force (Figure 5.15)

$P_{a1,ds} = 0.088 \times 15 = 1.32 \text{ kN/m}^2$

$P_{a,ds} = 0.088 \times 15 = 1.32 \text{ kN/m}^2$

$P_{a2,ds} = 0.161 \times 15 = 2.42 \text{ kN/m}^2$

$P_{a3,ds} = 0.161 \times 15 = 2.42 \text{ kN/m}^2$

$P_{a4,ds} = 0.161 \times 15 = 2.42$ kN/m^2

$P_{a5,ds} = 0.161 \times 15 = 2.42$ kN/m^2

$P_{a6,ds} = 0.161 \times 15 = 2.42$ kN/m^2

$P_{a7,ds} = 0.161 \times 15 = 2.42$ kN/m^2

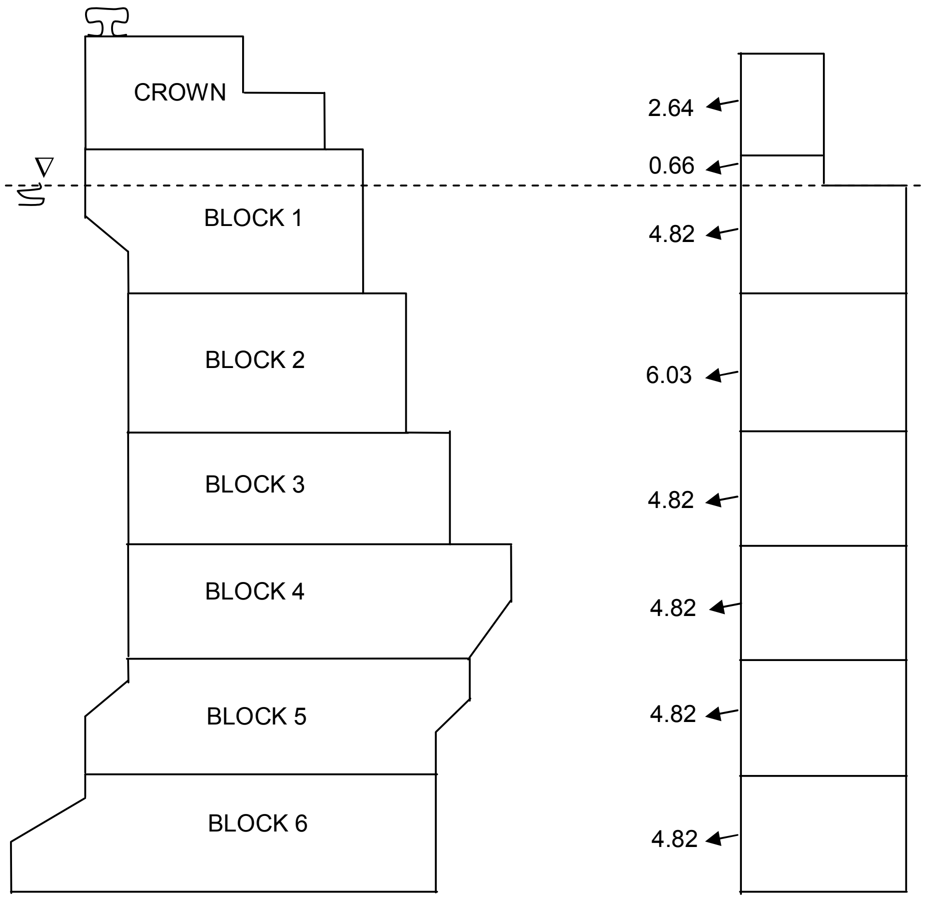


Figure 5.15: The surcharge effect on the block type quay wall for dynamic case

The total thrust (P_{AE}) acting on block type quay wall and the horizontal and vertical components of total thrust are shown in Figure 5.16.

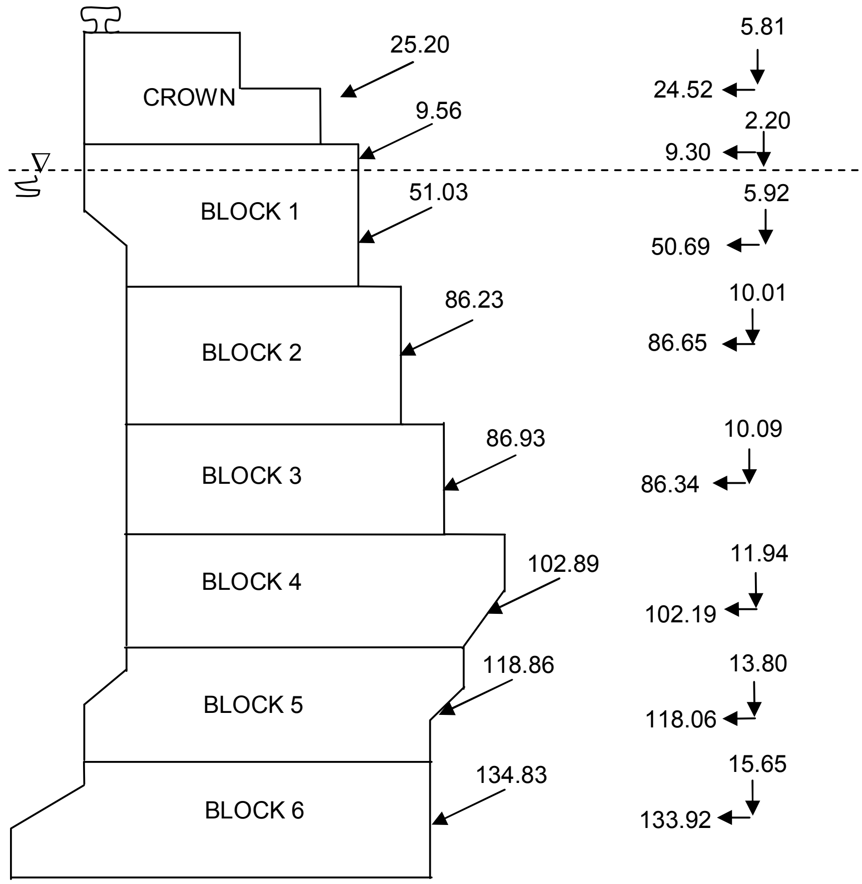


Figure 5.16: The total force on the block type quay wall.

7.5. Hydrodynamic Force

Using Eq.4.40 and Eq.4.41; the hydrodynamic pressures and forces are calculated.

$$P_{dw1} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(2)} = 7.00 \text{ kN/m}^2$$

$$P_{dw2} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(4.5)} = 10.50 \text{ kN/m}^2$$

$$P_{dw3} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(6.5)} = 12.62 \text{ kN/m}^2$$

$$P_{dw4} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(8.5)} = 14.43 \ \text{kN/m}^2$$

$$P_{dw5} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(10.5)} = 16.04 \ \text{kN/m}^2$$

$$P_{dw6} = \frac{7}{8}(0.16)(10)\sqrt{(12.5)(12.5)} = 17.50 \ \text{kN/m}^2$$

The first area is assumed that **triangular**;

$$F_{dw1} = \frac{7{,}00 \times 2}{2} = 7{,}00 \ \text{kN/m}$$

The other areas are assumed that **trapezoidal**;

$$F_{dw2} = \left(\frac{7.00 + 10.50}{2}\right)2.5 = 21.88 \ \text{kN/m}$$

$$F_{dw3} = \left(\frac{10.50 + 12.62}{2}\right)2 = 23.12 \ \text{kN/m}$$

$$F_{dw4} = \left(\frac{12.62 + 14.43}{2}\right)2 = 27.05 \ \text{kN/m}$$

$$F_{dw5} = \left(\frac{14.43 + 16.04}{2}\right)2 = 30.47 \ \text{kN/m}$$

$$F_{dw6} = \left(\frac{16.04 + 17.50}{2}\right)2 = 33.54 \ \text{kN/m}$$

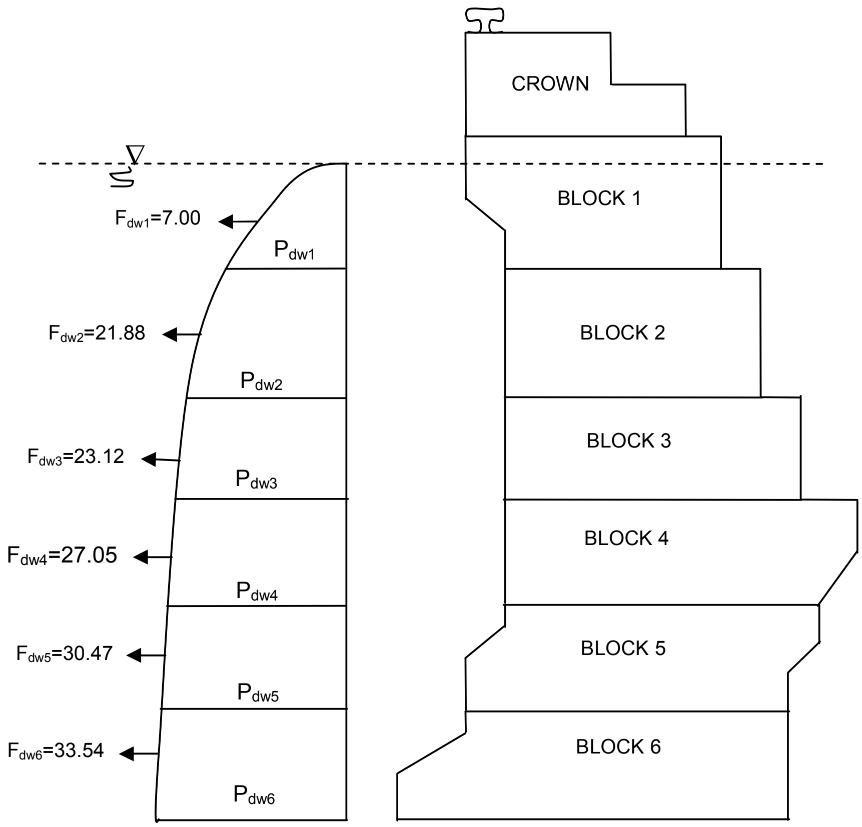


Figure 5.17: The hydrodynamic forces on the block type quay wall

8. VERTICAL FORCES

- **Crown Wall**

W= (a) (b) (γ_c)

W_{CA}=(2.8)(2.0)(23)= 128.8 kN/m

W_{CB}=(1.4)(1.0)(23)= 32.2 kN/m

W_C=161 kN/m

The center of gravity;

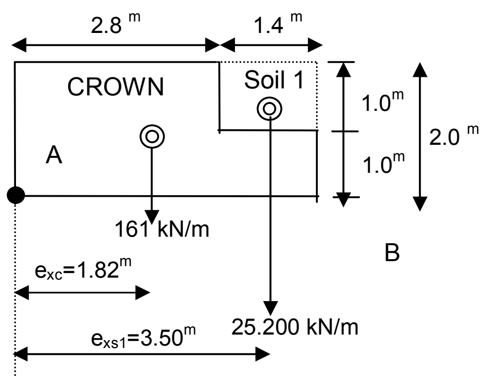
$$e_{xc} = \frac{128.8 \times 1.4 + 32.2 \times (2.8 + 1.4/2)}{128.8 + 32.2} = 1.82^m$$

Soil 1

W_{S1} = (1.4) (1)(18)= 25.20 kN/m

The center of gravity;

e_{xs1} =1.4/2+2.8=3.5^{m}



- **Block 1**

Under the water;

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{1A}=(4.75)(0.5)(23)= 54.63 kN/m

W_{1B}=(0.75)(0.75)/2(13)= 3.656 kN/m

W_{1C}=(0.75)(0.75)(13)= 7.313 kN/m

W_{1D}=(4.75)(2.0)(13)= 123.5 kN/m

W_{1T} = 54.63+123.5-3.656-7.313 =167.16 kN

The center of gravity; (within buoyancy effect)

$$e_{x1} = \frac{(54.63)(4.75/2)+(4.75)(2)(13)(4.75/2)-(0.75)(0.75)(0.75/3)(13)/2}{167.16}$$

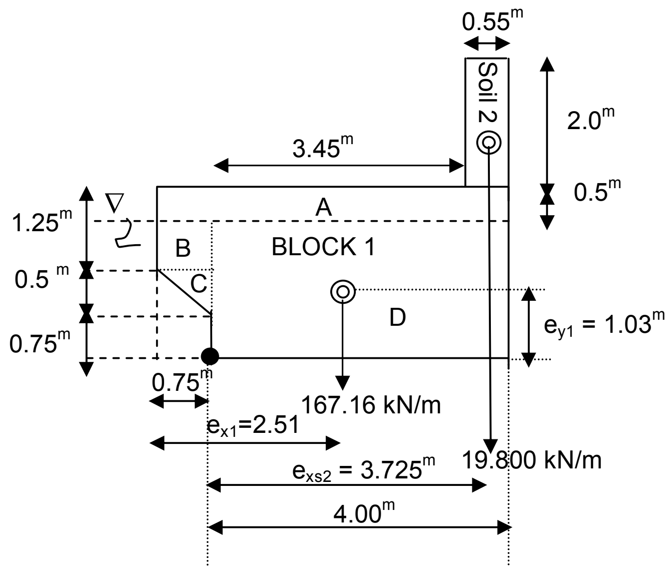
$$\frac{-(0.75)(0.75)(0.75/2)(13)}{167.16} = 2.51$$

Soil 2

W_{S2} = (0.55) (2)(18.00)= 19.800 kN/m

The center of gravity;

e_{xs2} =0.55/2+3.45=3.725 ^{m}



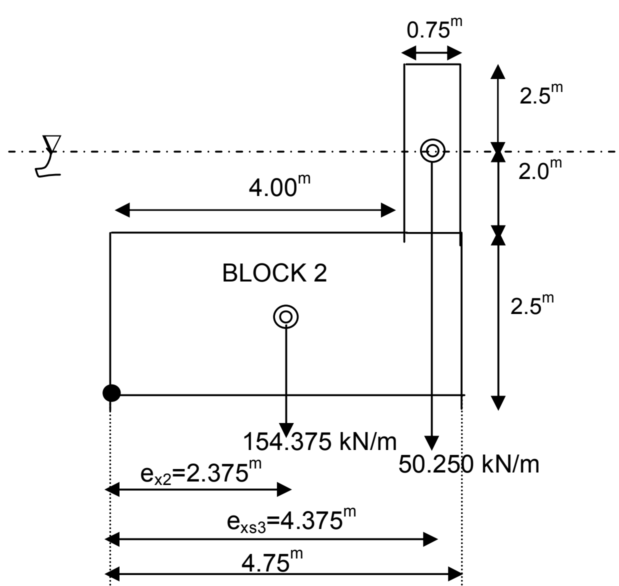
- **Block 2**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_2= (4.75)(2.5)(13)=154.375 kN/m

The center of gravity;

e_{x2}= 4.75/2 +4=2.375^{m}



Soil 3

W_{s3}=(0.75)(2.5)(18.00)+ (0.75)(2.0)(11.00)=50.250 kN/m

The center of gravity;

e_{xs3}= 0.75/2 +4=4.375^m

- **Block 3**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_3= (5.5)(2.0)(13)=143.00 kN/m

The center of gravity;

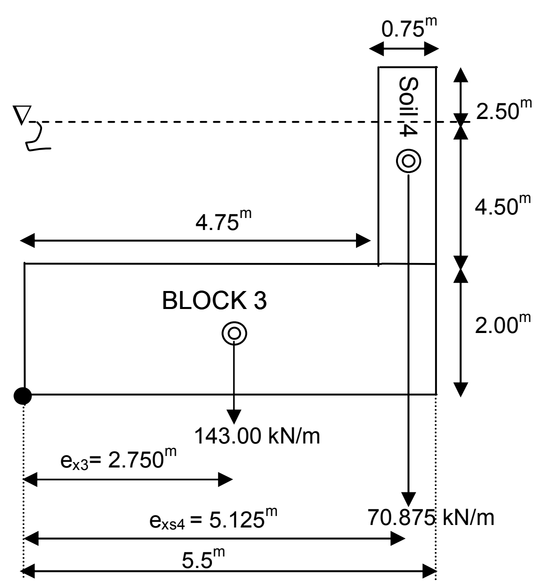
e_{x3}= 5.5/2 =2.75^m

Soil 4

W_{s4}=(0.75)(2.5)(18.00)+(0.75)(4.5)(11.00)=70.875 kN/m

The center of gravity;

e_{xs4}= 0.75/2 +4.75=5.125^m



- **Block 4**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{4A}= (6.6)(2.0)(13)=171.6 kN/m

W_{4B}= $\dfrac{(0.75)(1.0)}{2}$(13) = 4.875 kN/m

W_{4T} = 171.6 - 4.875 =166.725kN/m

The center of gravity;

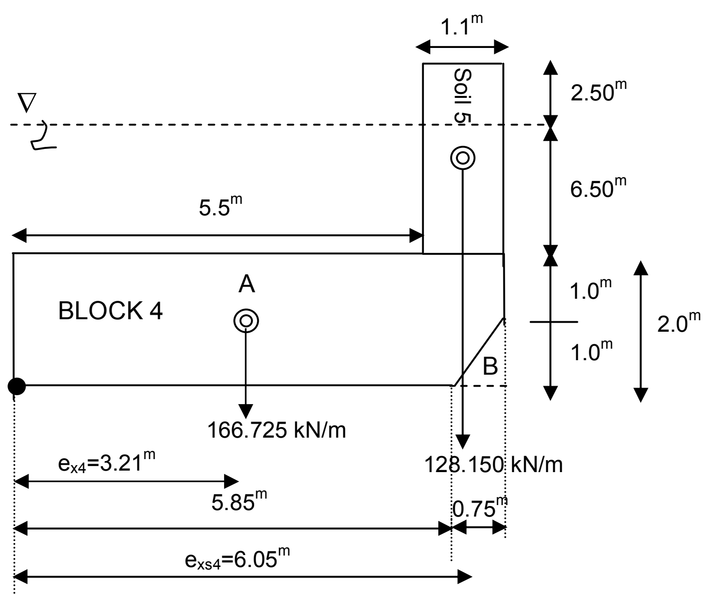
$$e_{x4}=\frac{(171.6)(6.6/2)-(4.875)(5.85+0.5)}{166.725}=3.21^m$$

Soil 5

W_{s5}= (1.10)(2.50)(18.00)+ (1.10)(6.50)(11.00)=128.150 kN/m

The center of gravity;

e_{xs5}= 1.1/2 + 5.5=6.05^m



- **Block 5**

W= (a) (b) (γ_c)   (Under water γ_{cw} is taken)

W_{5A}= (6.6)(2.0)(13.0)= 171.6 kN/m

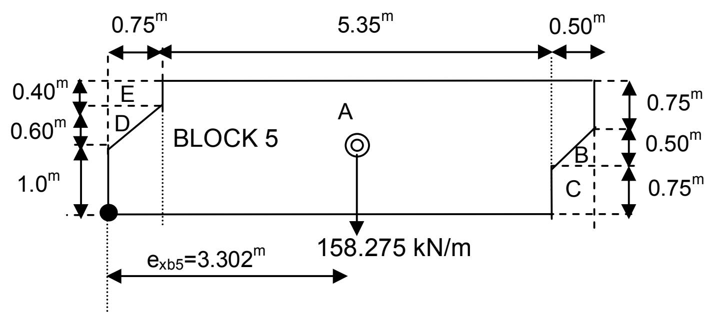
W_{5B}= (0.75)(0.5)(13.0)= 4.875 kN/m

W_{5C}= $\frac{(0.5)(0.5)}{2}(13.0)$ = 1.625 kN/m

W_{5D}= $\frac{(0.6)(0.75)}{2}(13.0)$ = 2.925 kN/m

W_{5E}= (0.4)(0.75)(13.0)= 3.9 kN/m

W_{b5}= 171.6-4.875-1.625-2.925 -3.9 =158.275 kN/m



The center of gravity;

$$e_{x5}=\frac{(139.10)(5.35/2+0.75)+(4.875)(0.5/2+6.10)+(1.625)(0.5/3+6.10)}{158.275}$$

$$\frac{+(2.925)(0.75\times 2/3)+(9.75)(0.75/2)}{158.275}=3.302^{m}$$

- **Block 6**

W= (a) (b) (γ_c) (Under water γ_{cw} is taken)

W_{6A}= (7.4)(2.0)(13.0)= 192.40 kN/m

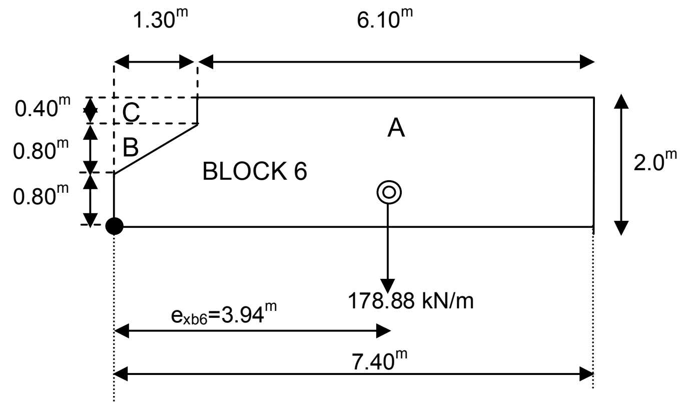
W_{6B}= $\frac{(1.30)(0.8)}{2}(13.0)$ = 6.76 kN/m

W_{6C}= (1.30)(0.4)(13.0)= 6.76 kN/m

W_{b6}= 192.40-6.76-6.76 = 178.88 kN/m

The center of gravity;

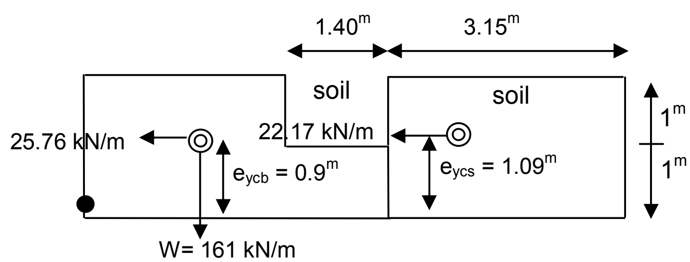
$$e_{xb6}=\frac{(192.40)(7.40/2)-(6.76)(1.30/3)-(6.76)(1.30/2)}{178.88}=3.94^{m}$$



9. INERTIA FORCES

By using the Pseudostatic Analysis (Eq. 4.5);

- **Crown Wall**



From crown wall;

W_C= 161 kN k_h=0.16

F_{hC}= (161)(0.16)=25.76 kN/m

From soil ; (γ=18 kN/m^3)

(3.15)(2.0)(18)(0.16)= 18.14 kN/m

(1.4)(1.0)(18)(0.16)= 4.03 kN/m

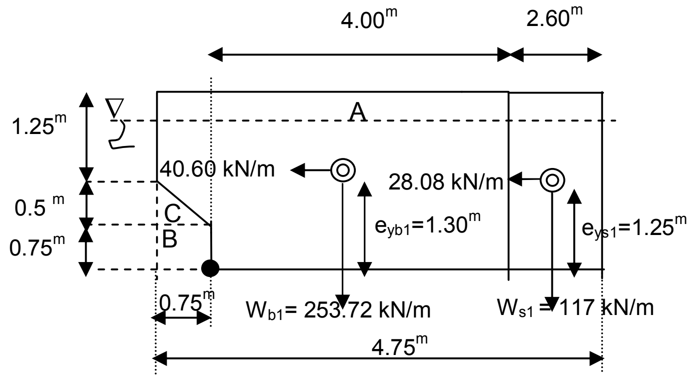
** Total Blocks Force = 128.8+32.2 = 161 kN/m

$$e_{ycb} = \frac{(128.8)(1.0) + (32.2)(0.5)}{161} = 0.9$$

** Total Soils Force = 18.14+4.03= 22.17 kN/m

$$e_{ycs} = \frac{(4.03)(1.0/2 + 1.0) + (18.14)(1.0)}{22.17} = 1.09^m$$

- **Block 1**



Total weight of Block 1:

W= (a) (b) (γ_c)

W_{1A}=(4.75)(2.5)(23)= 273.125 kN/m

W_{1B}=(0.5)(0.75)(23)= 12.94 kN/m

W_{1C}=(0.75)(0.75)/2(23)= 6.47kN/m

W_{b1} =273.125-6.47-12.94 = 253.72 kN/m

$$e_{yb1} = \frac{(273.125)(2.5/2)-(6.47)(0.75/3+0.75)-(12.94)(0.75/2)}{253.72} = 1.30^{m}$$

From block 1 ;

F_{hb1}= (253.72)(0.16)=40.60 kN/m

Total weight of Soil: (γ =18.00 kN/m^3)

W_{s1}=(2.6)(0.5)(18.0) = 23.4 kN/m

W_{s2}=(2.6)(2.5)(18.0) = 93.6 kN/m

$e_{yb1} = 1.30^{m}$

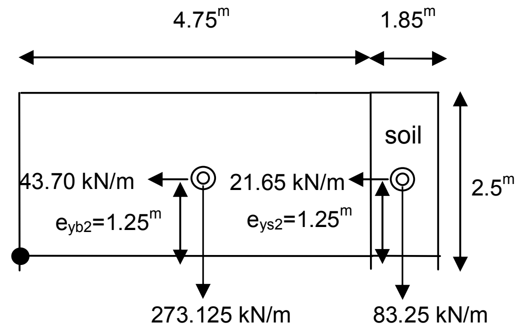
From soil ;

F_{hs1}= (23.4)(0.16)=3.74 kN/m

F_{hs1}= (93.6)(0.26)=24.34 kN/m

$e_{ys1} = (2.5/2) = 1.25^{m}$

- **Block 2**



Total weight of Block 2:

W= (a) (b) (γ_c)

W_{b2}= (4.75)(2.5)(23)=273.125 kN/m

From block 2 ;

F_{hb2}= (273.125)(0.16)=43.70 kN/m

$e_{yb2} = 2,5/2 = 1,25^m$

Total weight of Soil: (γ =18.00 kN/m^3)

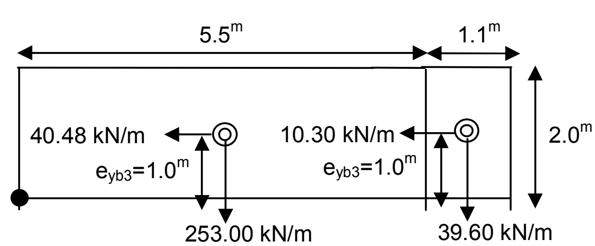
W_{s2}=(1.85)(2.5)(18.0) = 83.25 kN/m

From soil ;

(83.25)(0.26)=21.65 kN/m

$e_{ys2} = 2.5/2 = 1.25^m$

- **Block 3**



Total weight of Block 3:

W= (a) (b) (γ_c)

W_{b3}= (5.5)(2.0)(23)=253 kN/m

From Block 3 ;

F_{hb3}= (253.00)(0.16)=40.48 kN/m

$e_{yb3} = 2/2 = 1{,}0^m$

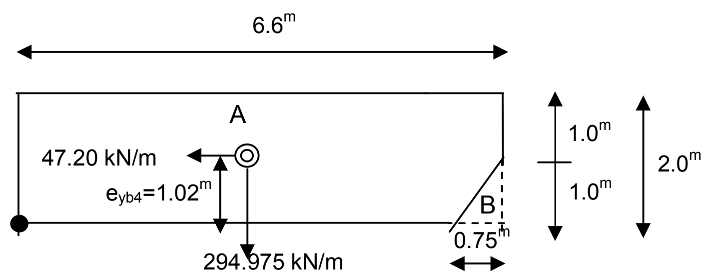
Total weight of Soil: (γ =18 kN/m^3)

W_{s3}=(1.10)(2.0)(18.0) = 39.6 kN/m

From soil ;

(39.6)(0.26)= 10.30 kN/m and $e_{ys3} = 2/2 = 1.0^m$

- **Block 4**



Total weight of Block 4:

W= (a) (b) (γ_c)

W_{4A}= (6.6)(2.0)(23)=303.6 kN/m

W_{4B}= $\frac{(0.75)(1.0)}{2}$(23)=8.625 kN/m

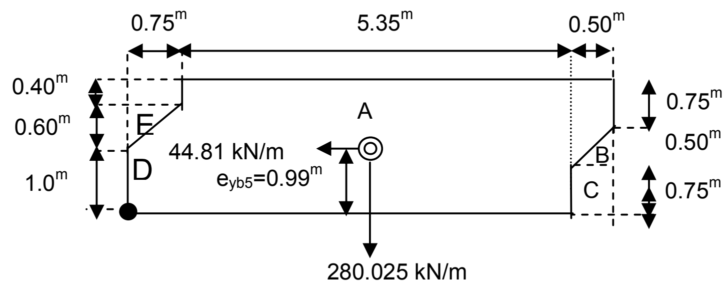
W_{b4}= 303.6-8.625 =294.975 kN/m

From block 4;

F_{hb4}= (294.975)(0.16)=47.20 kN/m

e_{yb4}=$\frac{(303.6)(2/2)-(8.625)(1/3)}{294.975} = 1.02^m$

- **Block 5**



Total weight of Block 5:

W= (a) (b) (γ_c)

W_{5A}= (6.6)(2.0)(23.0)= 303.6 kN/m

W_{5B}= (0.75)(0.5)(23.0)= 8.625 kN/m

W_{5C}= $\frac{(0.5)(0.5)}{2}(23.0)$= 2.875 kN/m

W_{5D}= $\frac{(0.6)(0.75)}{2}(23.0)$ = 5.175 kN/m

W_{5E}= (0.4)(0.75)(23.0)= 6.9 kN/m

W_{b5}= 303.6 - 8.625- 2.875 - 5.175 - 6.9 =280.025 kN/m

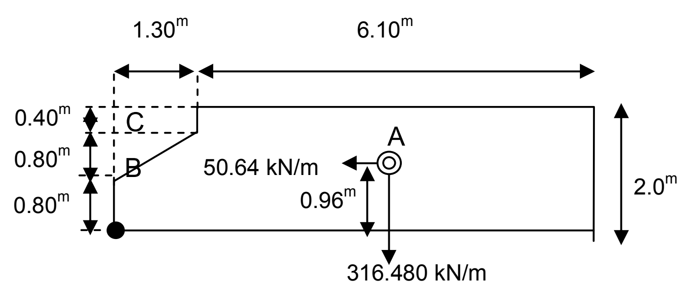
From block 5 ;

F_{hb5}= (280.025)(0.16)=44.81 kN/m

$$e_{yb5} = \frac{(303.6)(1.0) - (8.625)(0.75/2) - (2.875)(0.917) - (5.175)(1.40)}{280.025}$$

$$= \frac{-(6.9)(1.8)}{280.025} = 0.99$$

- **Block 6**



Total weight of Block 6:

W= (a) (b) (γ_c)

W_{6A}= (7.4)(2.0)(23.0)= 340.40 kN/m

W_{6B}= $\frac{(1.30)(0.8)}{2}(23.0)$ = 11.96 kN/m

W_{6C}= (1.30)(0.4)(23.0)= 11.96 kN/m

W_{b6}= 340.40 - 11.96 - 11.96 = 316.48 kN/m

From block 6;

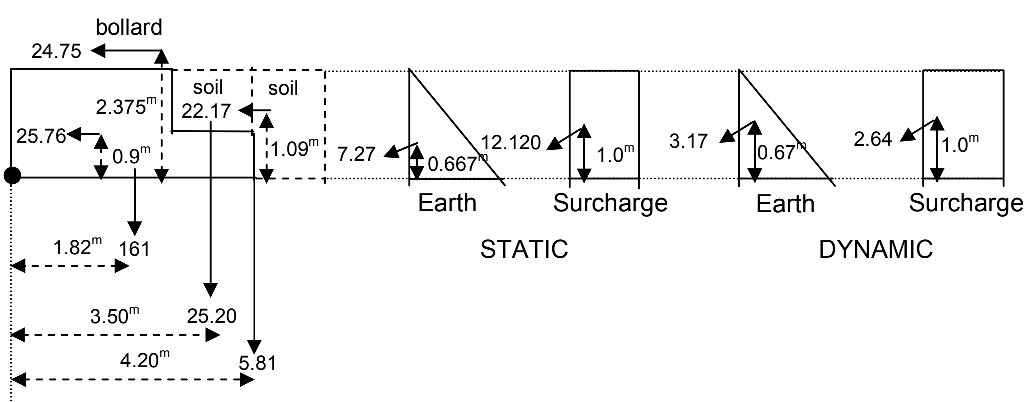
F_{h6}= (316.480)(0.16)=50.64 kN/m

The center of gravity;

$$e_{yb6}=\frac{(340.40)(2.0/2)-(11.96)(0.8\times 2/3+0.8)-(11.96)(0.4/2+1.6)}{316.48}=0.96^m$$

10. Stability Analysis

- **Crown:**



∑ **Horizontal Force=** 25.76 + 22.17+(7.27)cos13.33+(0.5)(12.12)cos13.33 (3.17)cos13.33+(0.5)(2.64)cos13.33 +24.75 =90.02 kN/m

∑ **Vertical Force** = 161+25.20+5.81 =192.01 kN/m

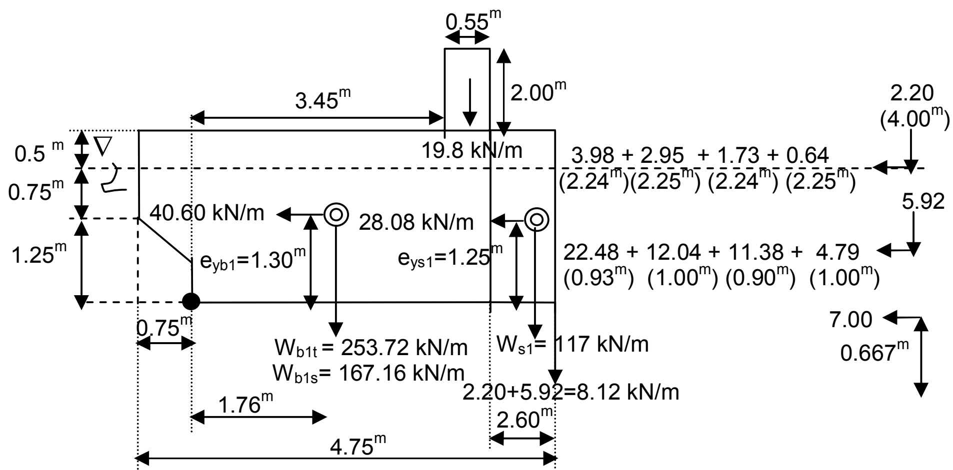
Overturning Moments= (25.76)(0.90)+(22.17)(1.09)+(7.07)(2/3) +(11.79)(1.00)+(3.08)(2/3)(1.5)+(2.57)(1.00)(1.50)+(24.75)(2.375)=129.57kNm/m

Resisting Moments = (161)(1.82) +(25.20)(3.50)+(5.81)(4.20) =405.62 kNm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.50)(192.01)}{90.02} = 1.07 \;\; <1.1(\;x\;) \qquad FS_o = \frac{405.62}{129.57} = 3.13 \;\; > 1.2\;(OK\;)$$

- **BLOCK 1:**



∑Horizontal Force= 90.02 +40.60+28.08+((4.09)cos13.33+(22.63) cos(13.33/2)) +(0.5(3.03)cos13.33+0.5(12.12) cos(13.33/2)) +((1.78)cos13.33+(11.46) cos(13.33/2)) +(0.5(0.66)cos13.33+0.5(4.82) cos(13.33/2)) +7.00= 215.48 kN/m

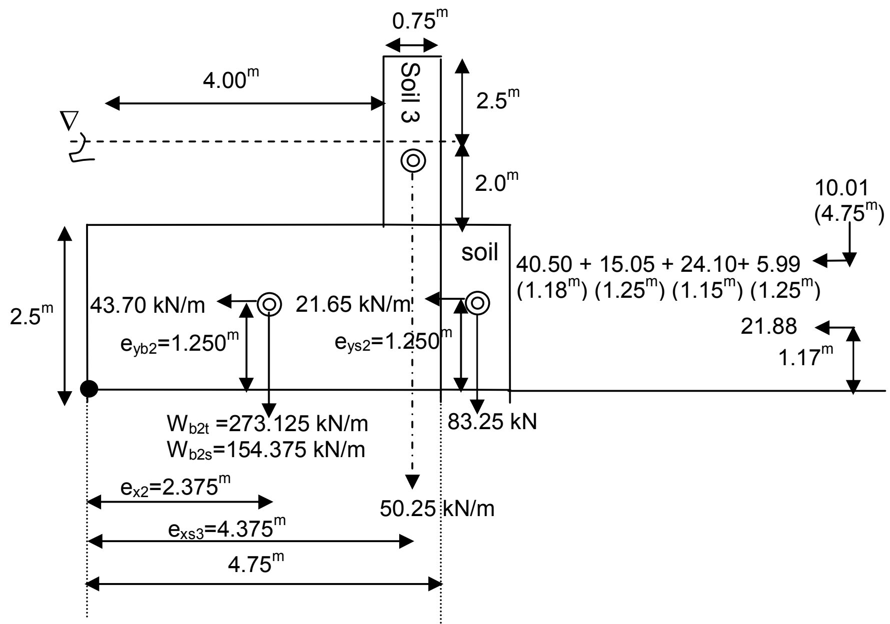
∑ **Vertical Force** = 192.01 + 167.16 + 19.8+ 8.12= 387.09 kN/m

Overturning Moments= 129.57 +(90.02)(2.5)+ (40.60)(1.30) + (28.08)(1.25) +((3.98)(0.24+2) +(22.48)(0.93)) +((2.95)(2.25)+(12.04)(1.00)) +((1.73)(2.24)+(11.38)(0.90))(1.50) + ((0.64)(2.25)+(4.79)(1.00))(1.50) + (7.0)(2/3) = 526.19 kNm/m

Resisting Moments = (128.8)(1.40-0.75)+(32.2)(2.8-0.75+0.7) +(25.20)(2.75) +(5.81)(3.45)+(167.16)(1.76)+(19.8)(3.725) +(8.12)(4.00)= 662.05 kNm/m

$$FS_s = \frac{(0.50)(387.09)}{215.48} = 0.90 < 1.1(\;x\;) \qquad FS_o = \frac{662.05}{526.19} = 1.26 \;\; >1.2 \quad (ok)$$

- **BLOCK 2**



$\sum$ **Horizontal Force=** 215.48+ 43.70 + 21.65+ 40.50 + 0.5 (15.05) +24.10 + 0.5 (5.99) + 21.88 = 377.83 kN/m

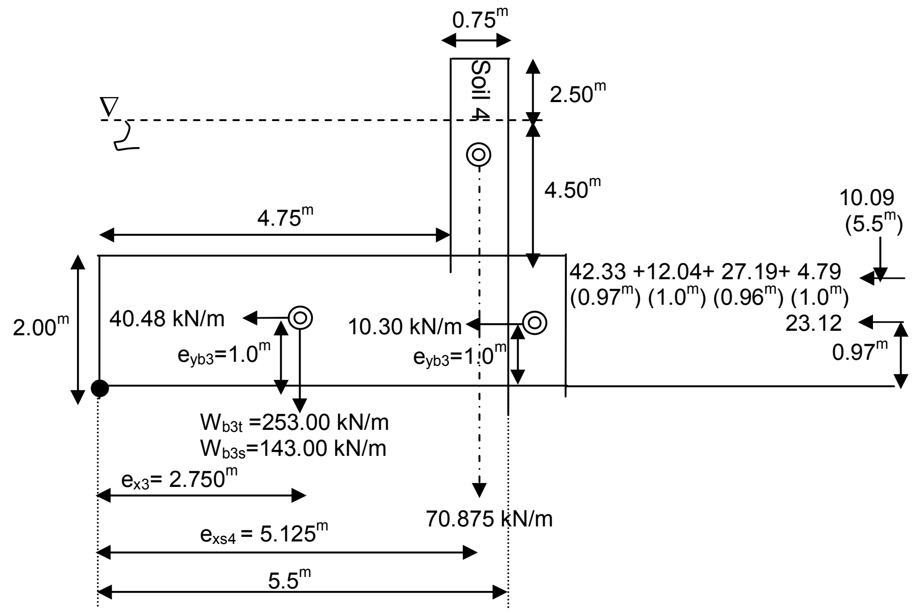
$\sum$ **Vertical Force** = 387.09 + 154.375 + 50.25 +10.01= 601.73 kN/m

Overturning Moments= 526.19 + (215.48)(2.50)+ (43.70)(1.25) +(21.65)(1.25) +(40.50)(1.18) +(15.05)(1.25)+ (24.10)(1.15)(1.50) +(5.99)(1.25)(1.50) + (21.88)(1.17) = 1291.58 kNm/m

Resisting Moments = 662.05 + (154.375)(2.375) +(50.25)(4.375) +(10.01)(4.75) = 1296.08 kNm/m

$FS_s = \frac{(0.50)(601.73)}{377.83} = 0.79 < 1.1(\ x\ )$ $FS_o = \frac{1296.08}{1291.58} = 1.00 \quad < 1.2\ (x)$

- **BLOCK 3**



$\sum$ **Horizontal Force=** 377.83 + 40.48 + 10.30 + 42.33 + 0.5(12.04) +27.19 + 0.5(4.79) + 23.12 =529.67 kN/m

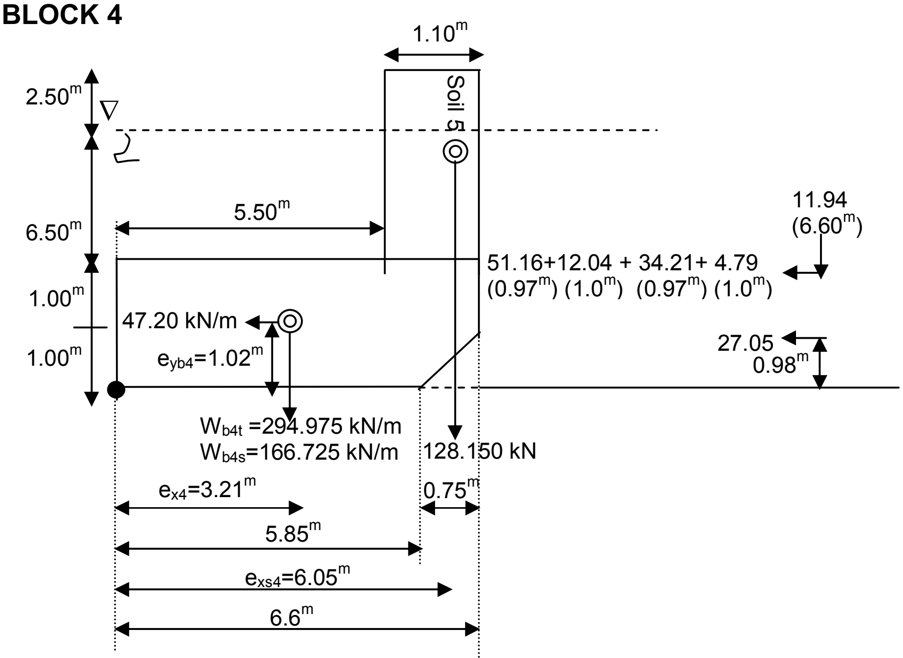
$\sum$ **Vertical Force** = 601.73 + 143.00 + 70.875 + 10.09 = 825.70 kN/m

Overturning Moments= 1291.58 + (377.83)(2.00) +(40.48)(1.00) +(10.30)(1.00) +(42.33)(0.97)+(12.04)(1.00) +(27.19)(0.96)(1.50) +(4.79)(1.00)(1.50) +(23.12)(0.97) = 2219.89 kNm/m

Resisting Moments = 1296.08 + (143.00)(2.75) +(70.875)(5.125) +(10.09)(5.50) = 2108.06 kNm/m

$FS_s = \frac{(0.50)(825.70)}{529.67} = 0.78 < 1.1(\text{ x })$ $\quad FS_o = \frac{2108.06}{2219.89} = 0.95 \ < 1.2\ (\text{x})$

- **BLOCK 4**



∑ **Horizontal Force**= 529.67 + 47.20 + 51.16 + 0.5(12.04) + 34.21 + 0.5(4.79) + 27.05 = 697.71 kN/m

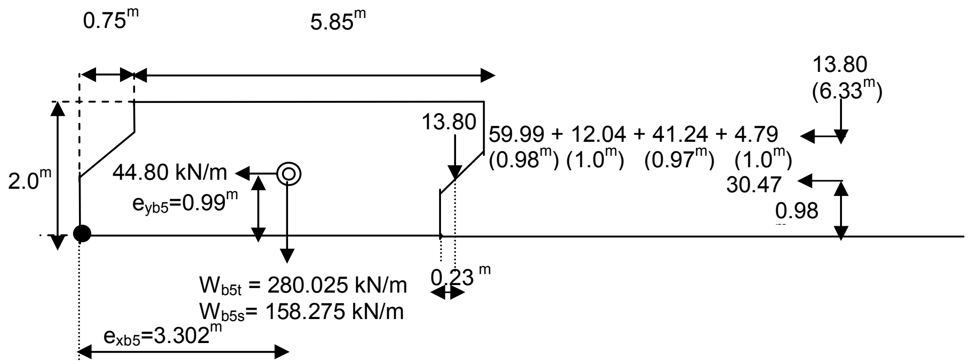
∑ **Vertical Force** = 825.70 +166.725+128.15+11.94 = 1132.52 kN/m

Overturning Moments= 2219.89 + (529.67)(2.00) + (47.20)(1.02) + (51.16)(0.97) + (12.04)(1.00) + (34.21)(0.97)(1.50) +(4.79)(1.00)(1.50) + (27.05)(0.98) = 3472.51 kNm/m

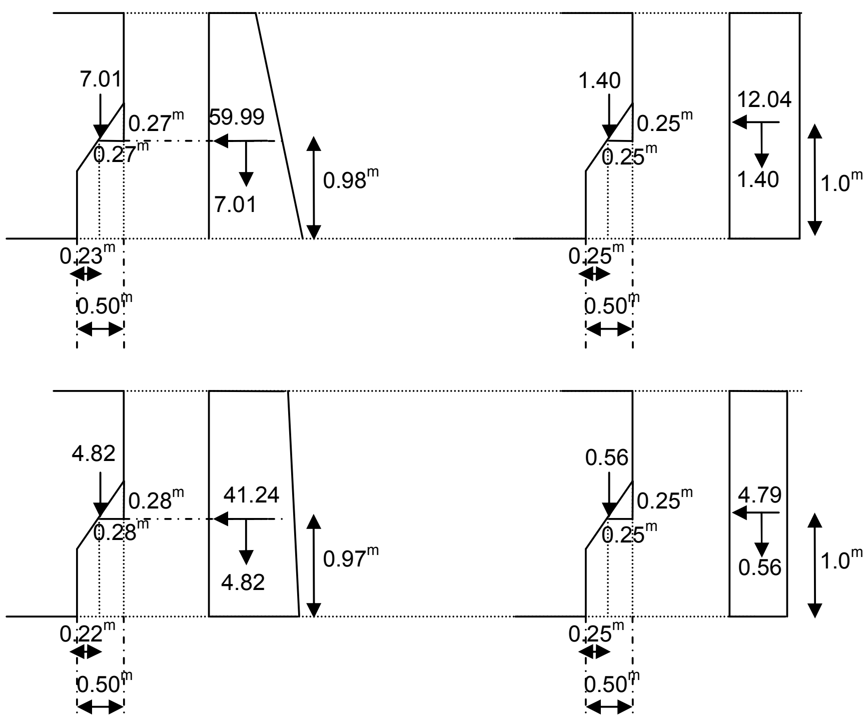
Resisting Moments = 2108.06 + (166.725)(3.21) + (128.15)(6.05) + (11.94)(6.60) = 3497.36 kNm/m

$$FS_s = \frac{(0.50)(1132.52)}{697.71} = 0.81 < 1.1(\ x\ ) \qquad FS_o = \frac{3497.36}{3472.51} = 1.00 \ \ < 1.2\ (x)$$

- **BLOCK 5**



In order to find the effect point of vertical force;



$$x = \frac{(7.01)(0.23) + (1.40)(0.25) + (4.82)(0.22) + (0.56)(0.25)}{13.80} = 0.23^{m}$$

∑ **Horizontal Force**= 697.71 + 44.80+ 59.99 + 0.5(12.04) + 41.24+ 0.5(4.79) + 30.47 = 882.63 kN/m

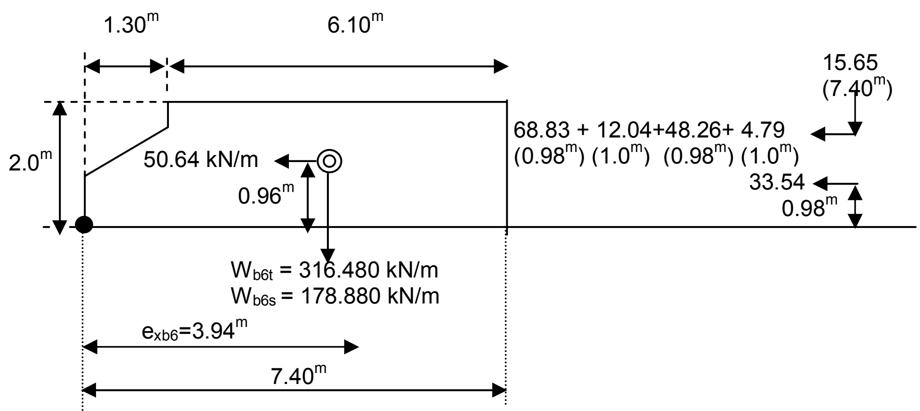
∑ **Vertical Force** = 1132.52 + 158.275 + 13.80 = 1304.60 kN/m

Overturning Moments= 3472.51+(697.71)(2.00)+(44.80)(0.99) +(12.04)(1.00) +(41.24)(0.97)(1.50) +(4.79)(1.00)(1.50) + (30.47)(0.98) = 5080.16 kNm/m

Resisting Moments = 3497.36 + (1132.52) (0.75) + (158.275)(3.302) + (13.80)(6.33) = 4956.73 kNm/m

$$FS_s = \frac{(0.50)(1304.60)}{882.63} = 0.74 < 1.1(\ x\ ) \quad FS_o = \frac{4956.73}{5080.16} = 0.98 \ < 1.2\ (x)$$

- **BLOCK 6**



∑ **Horizontal Force =** 882.63 + 50.64 + 68.83 + 0.5(12.04) + 48.26 + 0.5 (4.79) + 33.54 = 1092.32 kN/m

∑ **Vertical Force** = 1304.60 + 178.88 + 15.65 = 1499.13 kN/m

Overturning Moments= 5080.16 + (882.63)(2.00) + (50.64)(0.96) + (68.83)(0.98) + (12.04)(1.00) + (48.26)(0.98)(1.50) + (4.79)(1.00)(1.50) + (33.54)(0.98) = 7084.52 kNm/m

Resisting Moments = 4956.73 + (1304.60)(1.30) + (178.880)(3.94) + (15.65)(7.40) = 7473.31 kNm/m

$$FS_s = \frac{(0.60)(1499.13)}{1092.32} = 0.82 < 1.1(\ x\ ) \quad FS_o = \frac{7473.52}{7084.52} = 1.06 \ < 1.2\ (x)$$

Table 5.20: Factor of safety values against sliding and overturning for (TSDC-CRA, 2007)

TSDC-CRA, 2007 (W_{dry})		
Block	**FS_s**	**Fs_o**
Crown	1.07	3.13
Block 1	0.90	1.26
Block 2	0.80	1.00
Block 3	0.78	0.95
Block 4	0.81	1.01
Block 5	0.74	0.98
Block 6	0.82	1.06

5.5.1. DISCUSSION ON Turkish Seismic Design codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007)

New design approach to conventional method is also compared with the TSDC-CRA, 2007. The formulations in order to define the peak ground acceleration (a_{max}) and the seismic coefficient (k_h) are taken from the TSDC-CRA, 2007. Thus, (a_{max}) and (k_h) values are same for all approaches defined in this study (Table 5.19).

For the static case; active earth pressure coefficient and active earth pressure is calculated by using the equations given in TSDC-CRA, 2007. The formulation of active earth pressure is given for both cohesion and cohesionless for dry and saturated soil. These formulations also contain surcharge parameter. TSDC-CRA, 2007 assumes a triangular distribution in order to define static earth effect. Dry and submerged unit weights of the soils being either above the water level or below the water level are taken into consideration. Surcharge load is not influenced from the unit weight of the soil as shown in Figure 5.18.

For **dynamic case**, M-O Method is used in order to define dynamic active earth pressure coefficient (K_{AE}) for two different seismic inertia angles. Thus, by using these inertia angles, two different K_d values are obtained. One of the K_d values is used for dry part and other K_d is used for saturated part. As it is mentioned before, while using the M-O Method during the dynamic analysis the forces acting on an active wedge in a dry cohesionless backfill are taken into account. However, TSDC-CRA, 2007 defines two dissimilar K_{AE} values for dry and saturated soil; this approach is not in agreement with the M-O Method's principal.

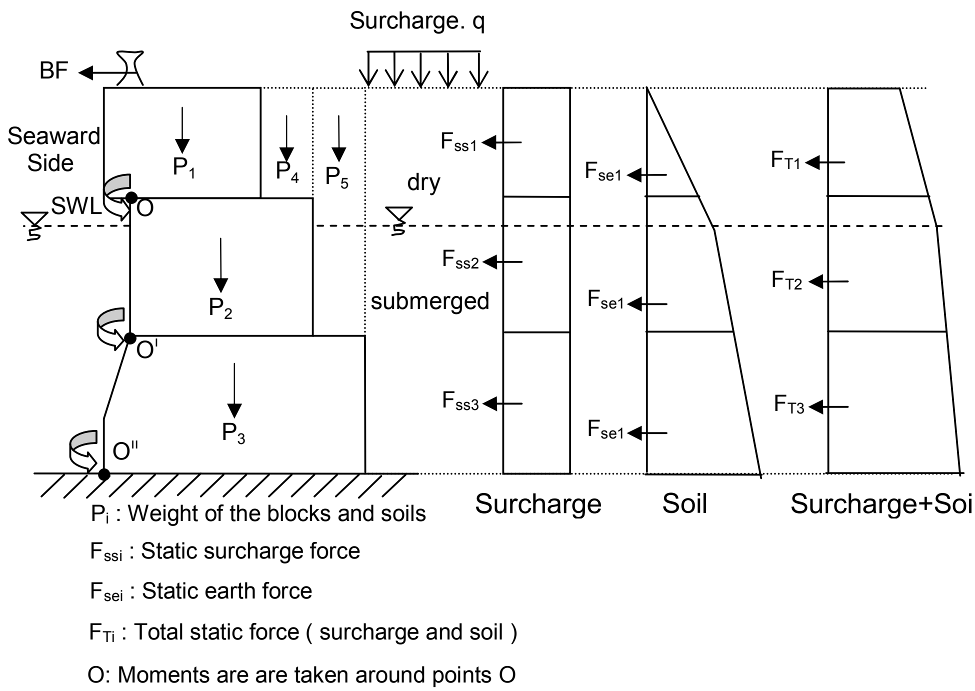


Figure 5.18: Static forces acting on block type quay wall (TSDC-CRA. 2007)

TSDC-CRA, 2007 does not give the distribution of pressure to define the dynamic earth effect on the blocks. But, after computing the pressure values, it is realized that lower factor of safety for sliding and overturning at the bottom blocks are obtained. However, this result is not in agreement with actual damage observation where upper blocks are more vulnerable during earthquake according to sliding.

Moreover, TSDC-CRA, 2007 states that the moment values due to the soil and surcharge should be multiplied with the 1.5 without convincing distributions.

In addition to these points, the seismic distribution of the forces due to the surcharge is accepted like rectangular shape as shown in Figure 5.19. However, it is believed that an inversed triangular force distribution can provide more reliable solution based on field measurement (Yüksel et al., 2002).

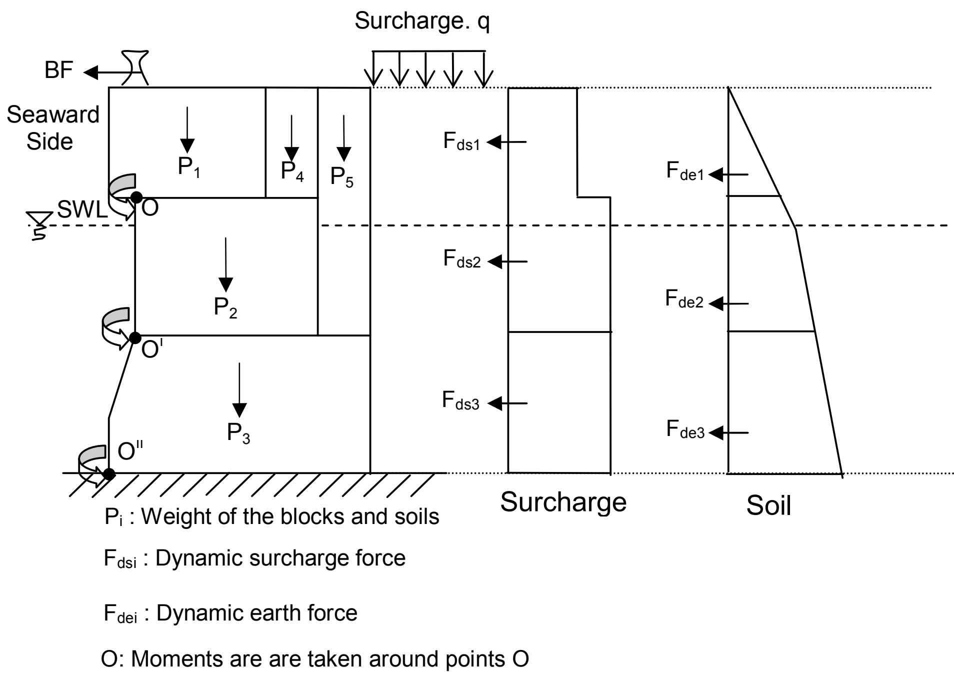


Figure 5.19: Dynamic forces acting on block type quay wall

TSDC-CRA, 2007 offers the usage of dry weight of blocks to define the inertia forces.

Table 5.21 and Table 5.22 show factor of safety values for Approach 1 and Approach 2 TSDC-CRA, 2007 for comparison.

As it is seen from Table 5.16 and Table 5.17, the blocks at the bottom have higher factor of safety against sliding for Approach 1 and Approach 2. On the other hand, the blocks at the bottom have smaller factor of safety against sliding for the TSDC-CRA, 2007. The most vulnerable block is **Crown Wall** and **Block 2** for **Approach 1** and **Approach 2** respectively according to **sliding**. On the other

Table 5.21: Factor of safety values both Approach 1 and TSDC-CRA, 2007 for comparison

Approach 1			TSDC-CRA. 2007	
Block	**FS_s**	**FS_O**	**FS_s**	**FS_O**
Crown	**0.63**	2.32	1.07	3.13
Block 1	0.64	0.95	0.90	1.26
Block 2	0.65	0.80	0.80	1.00
Block 3	0.69	**0.79**	0.78	**0.95**
Block 4	0.77	0.88	0.81	1.01
Block 5	0.75	0.89	**0.74**	0.98
Block 6	0.88	1.00	0.82	1.06

Table 5.22: Factor of safety values both Approach 2 and TSDC-CRA, 2007 for comparison

Approach 2			TSDC-CRA. 2007	
Block	**FS_s**	**FS_O**	**FS_s**	**FS_O**
Crown	0.63	2.32	1.07	3.13
Block 1	0.62	0.93	0.90	1.26
Block 2	**0.61**	0.77	0.80	1.00
Block 3	0.64	**0.75**	0.78	**0.95**
Block 4	0.71	0.82	0.81	1.01
Block 5	0.68	0.83	**0.74**	0.98
Block 6	0.80	0.93	0.82	1.06

hand, **Block 5** is the most critical block according to **sliding** for TSDC-CRA, 2007. **Block 3** is in risk with respect to **overturning** for **all conditions**. In addition, Table 5.16 points out that factor of safety value against sliding are calculated as 1.07 according to TSDC-CRA, 2007. This result implies that it should not be observed any motion on block type quay wall. However, scientific researches designated that 0.7 m horizontal displacement was observed in Derince Port Quay Wall (PIANC, 2001).

CHAPTER 6

CONCLUSION, DISCUSSION and SUGGESTIONS

Within the development of the new design approach, the following assumptions are used,

- Soil improvement techniques are used for the site where the existing soil conditions are expected not to lead unsatisfactory performance, such as liquefaction.

- Equivalent unit weight (γ_{eq}) is used during the both static and dynamic calculations.

- In this study for the **first time**, the distributions of the dynamic pressures are applied by assuming **inversed triangular** distribution.

- Peak ground acceleration (a_{max}) and seismic coefficient (k_h) are calculated according to the equations given in Turkish Seismic Design codes for Coastal Structures, Railways and Airport Structures, 2007 (TSDC-CRA, 2007).

- Due to the water in backfill, it is assumed that seismic coefficient (k_h) could be modified. So, during the seismic calculations, both modified seismic coefficient (k_h^{I}) and seismic coefficient (k_h) are used for comparison.

Dynamic forces are classified into four parts; dynamic earth forces, dynamic surcharge forces, hydrodynamic forces and inertia forces.

- **Dynamic Earth Forces:** Mononobe-Okabe Method is used to define dynamic active earth pressure coefficient (K_{AE}) and total active thrust

(P_{AE}). It is accepted that upward blocks are vulnerable during the earthquake and to enhance this assumption, inversed triangular distribution is used for the distribution of the dynamic component of the total thrust.

- **Dynamic Surcharge Forces:** For the dynamic surcharge force, $q_{sur}/2$ is taken. The distribution of the surcharge load is also assumed to be an inversed triangular distribution.

- **Hydrodynamic Forces:** It is clear that during seismic shaking, the free water in front of the structure exerts a cyclic dynamic loading on the wall and when suction pressure is applied on the wall the critical mode occurs. Thus, the hydrodynamic forces are taken into consideration according to Westergaard, 1933.

- **Inertia Forces:** Inertia forces are computed not only for the submerged weight of the blocks but also dry weight of the blocks as two different approaches. Vertical forces due to the weights of the blocks (dry or submerged) are multiplied with the horizontal seismic coefficient (k_h) and vertical forces due to the weights of the soils are multiplied with the modified seismic coefficient (k_h^{I}).

After defining the static and dynamic forces stability checks are performed for the each block.

- As for the stability analysis, factor of safety against sliding and overturning values for each block are computed. The static and dynamic forces due to soil and surcharge act on each block with an angle (δ), so in the stability computations both the vertical and horizontal components are taken into consideration.

- In general, sliding of the blocks is found to be more critical compared to overturning. For severe seismic shaking condition, overturning is found to be much more destructive than the sliding. In view of this, a higher safety factor for overturning (FS_o) than sliding (FS_s) is recommended.

- In the stability computations, the friction coefficients are taken as 0,5 and 0,6 for block-block and block-soil respectively.

For the case studies Derince Port is selected where peak ground acceleration is taken as 0.24 g (a_{max}=0.24 g).

- 4 different approaches are implemented where seismic coefficients and weights are defined differently (Table 5.7).

- **Approach 2** which uses modified seismic coefficient (k'_h) and dry weight of blocks (W_{dry}) during the calculation of inertia forces gives the most critical condition that is minimum factor of safety for both sliding and overturning (Table 5.4). In this case, including crown wall and upper section blocks namely, **Block 1-3**, are the **most critical** blocks with **FS_s** ranging between 0,61-0,64, **Block 2** and **Block 3** are the **most critical** with **FS_o** ranging between 0,77-0,75 respectively.

- Application of the numerical model in a case study at Derince Port proves that model results are qualitatively consistent with the real observations (Yüksel et al., 2002).

In addition, new design approach to conventional method is compared with the TSDC-CRA, 2007.

- Derince Port is selected to apply TSDC-CRA, 2007 where a_{max}=0.24 g.

- Considering the sliding, opposite to Approach 1-4, crown wall and block 2 are not the most critical blocks but **Block 5** is **critical**. Crown wall with factor of safety 1.07 and 3.13 for sliding and overturning respectively found stable. This result implies that it should not be observed any motion on block type quay wall. Since, it is known that during 1999 eastern Marmara earthquake, upper blocks were horizontally displaced.

Therefore, the factor of safety values are obtained using the Seismic Design Standards, 2007 are not in agreement with actual damage observation.

Based on these explanations and discussions new approaches to conventional method can be suggested to be used in the seismic design of block type walls.

Under light of the conclusions and discussions presented, it can be said that new design approaches to conventional method are more reliable and simple methods among the other methods since the results can be considered to be in agreement with the field data.

Obviously, due to fact that seismic events are rather unpredictable and field conditions are often characterized with significant uncertainties so laboratory tests and field measurements remain as an effective tool in developing the design methods. Therefore, further detailed experimental measurements could be performed to study the complex dynamic-soil-pore fluid-structure interaction problems.

New design approach presented has the objectives to provide safety against overturning and sliding. For a defined performance of a structure in terms of a state of damage, strain and deformation give better indicators of damage than stresses. Therefore, it is recommended that performance based design should be applied in the design of block type quay walls. Even if the force balance exceeds the limit values, it can be possible to get some information about the performance of a structure.

Also, in the future studies, crane loads and combinations together with different soil conditions can be included to check most critical stability conditions.

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APPENDIX A

CONVENTIONAL METHOD, 1975

A.1 FORCES

1. Active Earth Pressure

Active earth pressure coefficient K_A;

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2(\theta)\cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right]^2} \tag{1.1}$$

Where; K_A is active earth pressure coefficient. P_A is active earth pressure, θ is angle between the back of the retaining wall and the vertical plane, δ is friction angle between wall and soil, β is inclined angle with the horizontal, ϕ internal friction angle.

Horizontal component of active earth pressure coefficient, K_{Ah};

$$K_{Ah} = K_A \cos\delta = K_A \cos\left(\frac{\phi}{3}\right) \tag{1.2}$$

Vertical component of active earth pressure coefficient, K_{av};

$$K_{av} = K_a \cos\delta = K_a \sin\left(\frac{\phi}{3}\right) \tag{1.3}$$

Vertical component of earth pressure is neglected.

- If the earthquake is considered ϕ is reduced 6^o. This new value is used in order to calculate K_A value.

where, K_A is active earth pressure coefficient, ϕ is internal friction angle, β is inclined angle with the horizontal, δ is friction angle between wall and soil, θ is angle between the back of the retaining wall and the vertical plane (deg).

2. Horizontal Forces

A live load surcharge should be applied when traffic loads are located and this force usually uniformly distributed on the back-fill soil to a certain distance from the top of the wall.

The horizontal forces are calculated by using the e_i values, such as;

$$e_1 = (\text{surcharge})(K_{Ah}) \quad (1.4)$$

$$e_2 = (\gamma\, h_2\, K_{Ah}) + e_1 \quad (1.5)$$

$$e_n = (\gamma\, h_i\, K_{Ah}) + e_{i-1} \quad (1.6)$$

where, e_i is pressure values, h is the height from the still water level (SWL), K_{Ah} is the horizontal active earth pressure coefficient, γ is unit weight of the soil.

This pressure values are converted to forces (E_i) and these values are used for stability computations for each blocks.

- If the earthquake is considered additional horizontal forces (Z_i), occurred due to the weight of the blocks and soil, should be calculated by multiplying these values with the earthquake coefficient value (C). The earth pressure thrust values are added to these values (E_i+Z_i) and stability computations are made by using these new values for each block.

3. Vertical Forces

The weight of the blocks and soils are taken as vertical loads. During these calculations buoyancy force is taken into consideration.

4. Stability Computation

In stability calculations of gravity type quay wall, the following items should be examined;

1. Sliding of the wall
2. Overturning of the wall
3. Bearing capacity of the foundation

(Weber, Richard P., Personal Course Notes, 2000)

4.1 Sliding of the wall: A retaining structure has a tendency to move away from the backfill surface because of the horizontal driving forces resulting from the soil backfill and other forces such as surcharge. Generally, the wall resists sliding by the frictional resistance developed between the foundation of the wall and foundation soil. Although other horizontal forces act opposite to the driving force such as passive soil pressure in the fill in front of the wall. it is often ignored. The factor of safety with respect to sliding equals the resisting force divided by the driving force and is shown in Eq.(1.7). In conventional method, a minimum factor of safety of 1.5 is taken if the earthquake is not considered and a minimum factor of safety of 1.2 is taken if the earthquake is considered.

If the factor of safety against sliding is insufficient, increase resistance by either increasing the width of the base or lowering the base elevation. If the wall is founded on clay, the resistance against sliding should be based on short-term analysis and for long-term analysis (**TM 5-818-1 / AFM 88-3. Chap. 7)**

4.2 Overturning of the wall: A retaining structure also has a tendency to rotate outward around the toe of the wall. The moment resulting from the earth pressure force (as well as other lateral forces such as surcharge) must be resisted by the moments resulting from the vertical forces produced by the wall including any vertical component (P_{av}) of the earth pressure force. Thus, the factor of safety with respect to overturning is the resisting moment divided by the overturning moment as shown in Eq.(1.8). In conventional method, a minimum factor of safety of 1.5 is taken if the earthquake is not considered and also is considered.

For each block;

$$FS_S = \frac{\mu F_v}{F_H} \quad (1.7) \qquad\qquad FS_o = \frac{\sum M_r}{\sum M_o} \quad (1.8)$$

Sliding,

For ordinary condition ≥ 1.5

For extraordinary condition ≥ 1.2

Overturning,

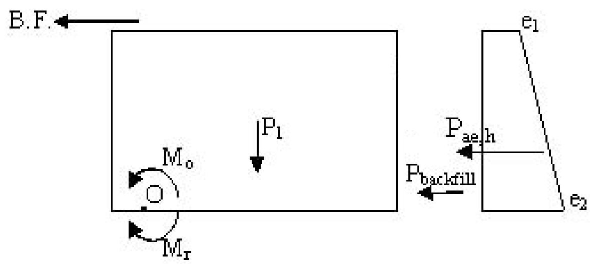
For ordinary condition ≥ 1.5

For extraordinary condition ≥ 1.5

where, μ is coefficient of friction between the bottom of the wall body and the foundation, F_v is resultant vertical force acting on the wall (kN/m), F_h is resultant horizontal force acting on the wall (kN/m), FS_s is factor of safety against sliding. FS_o is factor of safety against overturning, $\sum M_r$ is the sum of the resisting moments around the toe of the wall, $\sum M_o$ is the sum of the overturning moments around the toe of the wall.

The factor of safety against sliding and the factor of safety against overturning are calculated for each block.

For Crown Wall



$$M_o = \sum_{i=0}^{n} E_i r_i \qquad M_r = \sum_{i=0}^{n} P_i r_i$$

For Block 1

$$M_{o.Total} = M_{o.crown} + \sum M_o$$

$$M_{r.Total} = M_{r.crown} + \sum M_r$$

For Block 2

$$M_{o.Total} = M_{o.(crown+Block\ 1)} + \sum M_o$$

$$M_{r.Total} = M_{r.(crown+Block\ 1)} + \sum M_r$$

4.3 Bearing capacity of the foundation: As with any structure. the bearing capacity of the soil must be adequate to safely support the structure. The ultimate bearing capacity of the foundation soil (q_u) is calculated using theoretical bearing capacity methods. The resultant of all forces acting along the base of the wall from earth pressure and the weight of the wall result in a non-uniform pressure below the base of the wall with the greatest pressure below the toe of the base and the least pressure below the heel of the base.

The maximum and minimum pressure below the base of the wall (B) is:

q_{max} = (∑V / B) (1 + 6e / B) (5.0) (1.9)

q_{min} = (∑V / B) (1 - 6e / B) (6.0) (1.10)

Where e is eccentricity; e = (B / 2) - (∑M_r - ∑M_o) / ∑V (1.11)

Eccentricity is an important consideration when proportioning the wall. Consider the eccentricity (e) in relationship to the minimum pressure (q_{min}).

Substituting for (e);

If e = B / 6 then q_{min} = (∑V / B) (1 - 6e / B) = 0 (1.12)

If e < B / 6 then q_{min} = (∑V / B) (1 - 6e / B) > 0 (1.13)

If e > B / 6 then q_{min} = (∑V / B) (1 - 6e / B) < 0 (1.14)

Eq.(1.12) and Eq.(1.13) give acceptable results since the pressure at the heel is zero or greater (positive). Thus the entire base lies in contact with the soil. If Eq.(1.14) were true, then the pressure at the heel is negative indicating the heel of the base is tending toward lifting off the soil, which is unacceptable. If this condition occurs, then the wall must be re-proportioned (Weber, Richard P., Personal Course Notes, 2000).

A.2 EXAMPLE ON BLOCK TYPE QUAY WALL

Figure A.1 shows the section of block type quay wall

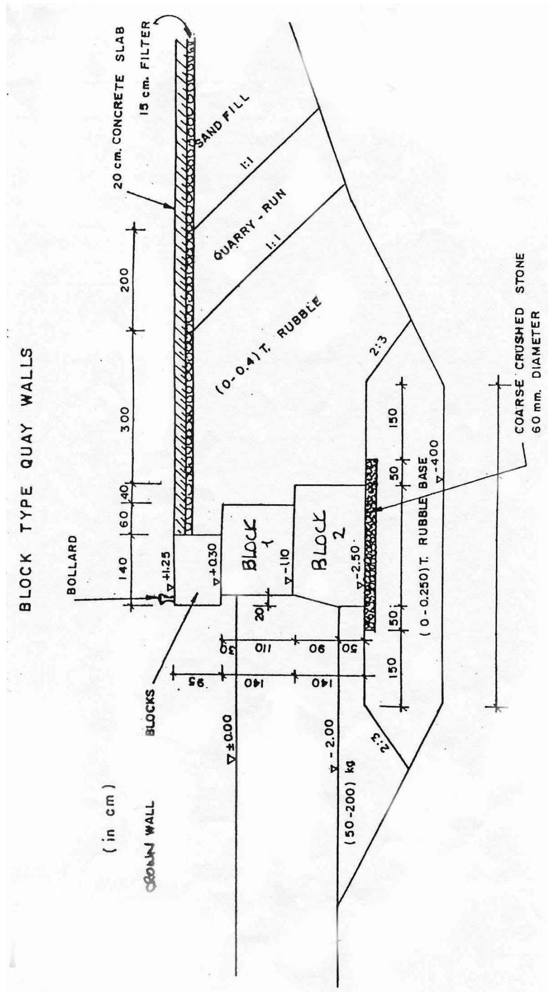


Figure A.1: Section of block type quay wall

A.2.1 EARTHQUAKE IS NOT CONSIDERED

DATA

1. Structure

Construction Depth	:	-2.00 m
Elevation above SWL	:	+1.25 m
Surcharge	:	1.50 t/m^2
Bollard Force	:	5t
Longitudinal length of crown wall	:	10 m

2. Backfill

Unit Weights: a) above GWT γ = 1.8 t/m^3

b) below GWT γ' = 1.1 t/m^3 (submerged)

Internal friction angle: ϕ=40^o

Friction angle between wall and soil: δ=1/3ϕ=13.33^o

3. Friction coefficients

a) crown wall + block	μ= 0.6
b) block + block	μ= 0.52
c) block + ruble base	μ= 0.52

4. Earth pressure coefficients

Active earth pressure coefficient K_A.

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

$$K_A = \frac{\cos^2(40-0)}{\cos^2(0)\cos(0+13.33)\left[1+\sqrt{\frac{\sin(40+13.33)\sin(40-0)}{\cos(13.33+0)\cos(0-0)}}\right]^2} = 0.202$$

Horizontal component of active earth pressure coefficient K_{Ah};

$$K_{Ah} = K_A \cos\delta = K_A \cos\left(\frac{\phi}{3}\right)$$

$$K_{Ah} = 0.202\cos 13.33 = 0.197$$

Vertical component of active earth pressure coefficient, K_{av};

$$K_{av} = K_a \cos\delta = K_a \sin\left(\frac{\phi}{3}\right)$$

Vertical component of earth pressure is neglected.

5. Vertical Forces

Table A.1 shows vertical forces acting on each block

$P = h\ b\ \gamma$

$P_1 = (0.95)(1.40)(2.30) = 3.059$ t/m

$P_2 = (0.30)(1.50)(2.3) + (1.10)(1.50)(1.30) = 3.180$ t/m

$P_3 = (0.95)(0.30)(1.80) = 0.513$ t/m

$P_4 = (0.90)\left(\frac{2.00 + 1.80}{2}\right)(1.30) + (0.50)(2.00)(1.30) = 3.523$ t/m

$P_5 = (1.25)(0.30)(1.80) + (1.10)(0.30)(1.10) = 1.038$ t/m

Table A.1: Vertical forces acting on each block

Vertical Forces	Block
$P_1 = 3.059$ t/m	Crown Wall $P_C = 3.059$ t/m
$P_2 = 3.180$ t/m	
$P_3 = 0.513$ t/m	Block 1 $P_{B1} = 6.752$ t/m
$P_4 = 3.523$ t/m	
$P_5 = 1.038$ t/m	Block 2 $P_{B2} = 11.313$ t/m

6. Horizontal Forces

e_1= surcharge (K_{Ah}) = (1.50)(0.197) = 0.296 t/m^2 h=0

$e_2 = \gamma\ h\ K_{Ah} + e_1 = (1.80)(0.95)(0.197)+0.296 = 0.633$ t/m^2 h=0.95

$e_3 = (1.80)(1.25)(0.197)+0.296 = 0.739$ t/m^2 h=1.25

$e_4 = (1.10)(1.10)(0.197)+0.739 = 0.977$ t/m^2 h=1.25 + 1.10^m

$e_5 = (1.10)(2.50)(0.197)+0.739 = 1.281$ t/m^2 h=1.25 + 2.50^m

Table 1.2 shows horizontal forces acting on each block

E_o= Bollard Force/ Crown Length = 5/10 = 0.5 t/m

$E_1 = e\ h = (0.296)(0.95) = 0.281$ t/m

$E_2 = (0.633-0.296)(0.95)/2 = 0.160$ t/m

$E_3 = \left(\frac{0.633 + 0.739}{2}\right)(0.30) = 0.206$ t/m

$E_4 = (0.739)(1.10) = 0.813$ t/m

$E_5 = (0.977-0.739)(1.10)/2 = 0.131$ t/m

$E_6 = (0.977)(1.40) = 1.368$ t/m

E_7= (1.281-0.977)(1.4)/2= 0.213 t/m

Table A.2: Horizontal forces acting on each block

Horizontal Forces		Block
E_o	0.500 t/m	
E_1	0.281 t/m	
E_2	0.160 t/m	Crown. E_c = 0.941 t/m
E_3	0.206 t/m	
E_4	0.813 t/m	
E_5	0.131 t/m	Block 1. E_{b1}= 2.091 t/m
E_6	1.368 t/m	
E_7	0.213 t/m	Block 2 .E_{b2}= 3.672 t/m

Figure A.2 shows forces acting on block type quay wall. if earthquake is not considered.

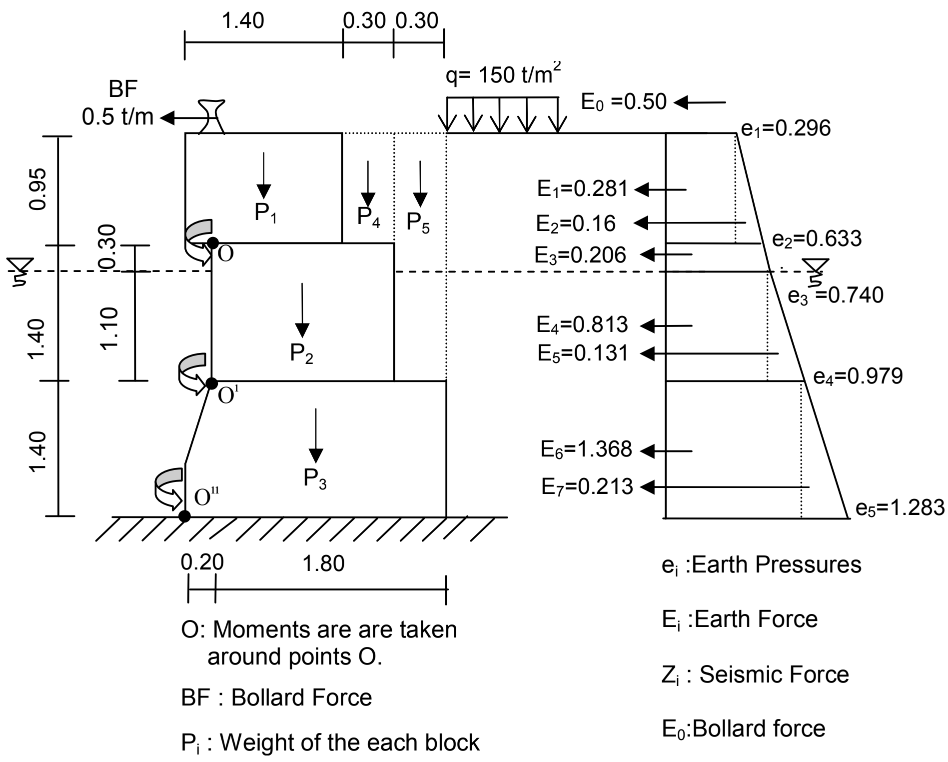


Figure A.2: Forces acting on block type quay wall, if earthquake is not considered (TSS, 1975)

7. STABILITY COMPUTATION

7.1 CROWN WALL

$\sum$ **Horizontal Force** = $E_0 + E_1 + E_2$

= 0.500+0.281+0.160 = 0.941 t/m

$\sum$ **Vertical Force** = P_C=3.059 t/m

Overturning Moment = $\sum_{i=0}^{2} E_i r_i$

=(0.500)(0.95+0.25)+(0.281)(0.95/2)

+(0.160)(0.95/3) = 0.784 tm/m

Resisting Moment = (3.059)(1.40/2-0.20)=1.530 tm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s=\frac{(0.6)(3.059)}{0.941}=1.95 > 1.20 \quad \text{(ok)}$$

$$FS_o=\frac{1.530}{0.780}=1.96 > 1.50 \quad \text{(ok)}$$

$$e=\frac{1.20}{2}-\frac{1.530-0.784}{3.059}=0.36<\frac{1.20}{3}=0.40 \quad \text{(ok)}$$

7.2 BLOCK 1

$\sum$ **Horizontal Force** = $E_C + E_4 + E_5$
= 0.941+0.206+0.813+0.131=2.091 t/m

$\sum$ **Vertical Force** = $P_C + P_2 + P_3 = P_{B1}$

= 3.059+3.180+0.513 = 6.752 t/m

Overturning Moment = $M_{o.crown} + \sum_{i=3}^{5} E_i r_i$

= 0.784+(0.941)(1.40)+(0.206)(1.25)+(0.813)(0.55)

+(0.131)(0.367)= 2.854 tm/m

Resisting Moment = $M_{s.crown} + \sum_{i=2}^{3} P_i r_i$

= 1.530 + (3.180)(0.75)+(0.513)0.30/2+1.20)

= 4.607 tm/m

FACTOR of SLIDING and OVERTURNING

$FS_s = \frac{(0.52)(6.752)}{2.091} = 1.68 > 1.20$ (ok)

$FS_o = \frac{4.607}{2.854} = 1.61 > 1.50$ (ok)

$e = \frac{1.50}{2} - \frac{4.607 - 2.854}{6.752} = 0.49 < \frac{1.50}{3} = 0.50$ (ok)

7.3 BLOCK 2

$\sum$ **Horizontal Force** = $E_{b1} + E_6 + E_7$

= 2.091+1.368+0.213=3.672 t/m

$\sum$ **Vertical Force** = $P_{B1}+P_4+P_5 = P_{B2}$

= 6.752+3.523+1.038=11.313 t/m

Overturning Moment = $M_{o.1} + \sum_{i=6}^{7} E_i r_i$

= 2.854+(2.091)(1.40)+(1.368)(0.70)+(0.213)(0.467)

= 6.838 tm/m

Resisting Moment = $M_{s.1} + \sum_{i=4}^{5} P_i r_i$

= 4.607+(6.752)(0.20)+ (3.523)(1.03)

+(1.038)(2.00-0.30/2)= 11.506 tm/m

FACTOR of SLIDING and OVERTURNING

$FS_s = \frac{(0.52)(11.313)}{3.672} = 1.60 > 1.20$ (ok)

$$FS_o = \frac{11.506}{6.838} = 1.68 > 1.50 \text{ (ok)}$$

$$e = \frac{2.00}{2} - \frac{11.506 - 6.838}{11.313} = 0.59 < \frac{2.00}{3} = 0.67 \quad \text{(ok)}$$

7.4 FOUNDATION

Check for maximum allowable bearing capacity

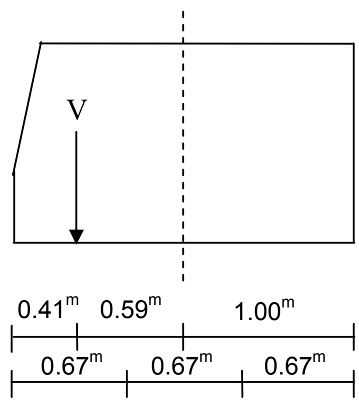
e<b/6

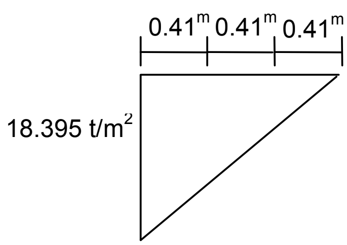
$$e = 0.59 > b/6 = \frac{2.00}{6} = 0.33$$

$$a = \frac{M_s - M_o}{V} = \frac{11.506 - 6.838}{11.313} = 0.41$$

a=0.41 m → Resultant is not within the core!

Distribution of the pressure is assumed to be triangular.





$$\text{Area} = \frac{(18.395)(0.41)(3)}{2} = 11.313 m^2$$

Assume $\sum V$ = area of triangle

$$V = \frac{1}{2} q_{max} 3a \Rightarrow 11.313 = \frac{1}{2}(q_{max})(3)(0.41) \Rightarrow q_{max} = 18.395 t/m^2$$

Dense sand bearing capacity = (32.2-64.5) t/m^2

Medium dense sand bearing capacity = (10.7 -32.2) t/m^2

Average bearing capacity can be taken as : (32.2+10.7)$\cong$21.5 t/m^2

q_{max}= 18.395 t/m^2 < 21.5 t/m^2 (ok)

A.2.2 EARTHQUAKE IS CONSIDERED

DATA

1. Structure

Construction Depth	:	-2.00 m
Elevation above SWL	:	+1.25 m
Surcharge	:	1.50 t/m^2
Bollard Force	:	5t
Longitudinal length of crown wall	:	10 m

2. Backfill :

Unit Weights : a) above GWT γ = 1.8 t/m^3

b) below GWT γ' = 1.1 t/m^3 (submerged)

Internal friction angle: ϕ=40^o

Friction angle between wall and soil: δ=1/3 ϕ=13.33^o

3. Friction coefficients

a) crown wall + block	μ= 0.6
b) block + block	μ= 0.52
c) block + ruble base	μ= 0.52

4. Earthquake Force Coefficients

C= C_0 K S L ; where C_0 : Regional earthquake coefficient

K : Structure type coefficient

S : Structure dynamics coefficient

L : Structure priority (importance) coefficient

Take C_0 = 0.1. K = 1.0 . S = 1.0 . L = 1.0

C= (0.1)(1.0)(1.0)(1.0) =0.1

-Reduce φ by 6^o ; Bollard Force= 2.5 t (due to the earthquake)

$$\phi = 40^o - 6^o = 34^o \qquad \delta = \frac{34^o}{3} = 11.33^o$$

5. Earth pressure coefficients

Active earth pressure coefficient K_A.

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

$$K_A = \frac{\cos^2(34-0)}{\cos^2(0)\cos(0+11.33)\left[1+\sqrt{\dfrac{\sin(34+11.33)\sin(34-0)}{\cos(11.33+0)\cos(0-0)}}\right]^2} = 0.262$$

Horizontal component of active earth pressure coefficient, K_{Ah}; $K_{Ah} = K_A \cos\delta$

$$K_{Ah} = 0.262\cos 11.33 = 0.257$$

Vertical component of active earth pressure coefficient, K_{av};

$$K_{Av} = K_A \cos\delta$$

Vertical component of earth pressure is neglected.

6. Vertical Forces (Table A.3)

P = h b γ

P_1 = (0.95)(1.40)(2.30) = 3.059 t/m

P_2 = (0.30)(1.80)(2.3) + (1.10)(1.80)(1.30) =3.816 t/m

P_3 = (0.95)(0.60)(1.80) = 1.026 t/m

P_4= $(0.90)\left(\dfrac{2.40+2.20}{2}\right)(1.30)+(0.50)(2.40)(1.30) =$ 4.251 t/m

P_5=(1.25)(0.40)(1.80)+(1.10)(0.40)(1.10)= 1.384 t/m

Table A.3: Vertical forces acting on each block

Vertical Forces	Block
P_1= 3.059 t/m	Crown Wall P_C = 3.059 t/m
P_2 = 3.816 t/m	
P_3 = 1.026 t/m	Block 1 P_{B1} = 7.901 t/m
P_4= 4.251 t/m	
P_5= 1.384 t/m	Block 2 P_{B2} = 13.536 t/m

7. Horizontal Forces (Table A.4)

e_1= surcharge (K_{Ah}) = (1.50)(0.257) = 0.386 t/m^2 h=0

e_2= γ h K_{Ah} + e_1 = (1.80)(0.95)(0.257)+0.386 = 0.825 t/m^2 h=0.95

e_3 = (1.80)(1.25)(0.257)+0.866 = 0.964 t/m^2 h=1.25

e_4 = (1.10)(1.10)(0.257)+0.964 = 1.275 t/m^2 h=1.25 + 1.10^m

e_5 = (1.10)(2.50)(0.257)+0.964 = 1.671 t/m^2 h=1.25 + 2.50^m

E_o= Bollard Force/ Crown Length = 2.5/10 = 0.25 t/m

E_1= e h = (0.386)(0.95) = 0.367 t/m

E_2= (0.826-0.386)(0.95)/2 =0.209 t/m

E_3= $\left(\dfrac{0.964+0.825}{2}\right)(0.30) = 0.268$ t/m

E_4= (0.964)(1.10)=1.060 t/m

E_5=(1.275-0.964)(1.10)/2= 0.171 t/m

E_6= (1.275)(1.40)=1.785 t/m

E_7= (1.671-1.275)(1.4)/2= 0.277 t/m

Table A.4: Horizontal forces acting on each block

Horizontal Forces		Block
E_o	0.250 t/m	
E_1	0.367 t/m	
E_2	0.209 t/m	Crown, E_C = 0.826 t/m
E_3	0.268 t/m	
E_4	1.060 t/m	
E_5	0.171 t/m	Block 1, E_{b1}= 2.325 t/m
E_6	1.785 t/m	
E_7	0.277 t/m	Block 2, E_{b2}= 4.387 t/m

8. Earthquake Forces

Earth Forces:

$P^{I} = (0.95)(0.60 + 0.40)(1.8) = 1.710 \text{ t/m}$

$P^{II} = (0.30)(0.40)(1.8) = 0.216 \text{ t/m}$

$P_3 = (1.10)(0.40)(1.1) = 0.484 \text{ t/m}$

Earthquake Forces:

$Z_1 = C\ (P_1 + P^{I}) \Rightarrow Z_1 = (0.10)(3.059 + 1.710) = 0.477 \text{ t/m}$

$Z_2 = C\ (P_2^{dry} + P^{II}) \Rightarrow Z_2 = (0.10)((0.30)(1.80)(2.3) + 0.216)) = 0.146 \text{ t/m}$

$Z_3 = C\ (P_2^{sat} + P^{II}) \Rightarrow Z_2 = (0.10)((1.10)(1.80)(1.3) + 0.484)) = 0.306 \text{ t/m}$

$Z_4 = C\ (P_4) \Rightarrow Z_4 = (0.10)(4.251) = 0.425 \text{ t/m}$

9. Total Horizontal Forces

Table A.5 shows total horizontal forces acting on blocks

Table A.5: Total horizontal forces acting on blocks

Earth Force (t/m)	Z (t/m)	∑ Horizontal Force	
E_0=0.250	-	H_0=0.250	
E_1=0.367	Z_1=0.477	H_1=0.844	
E_2=0.209		H_2=0.209	$\sum_{crown\ wall}$=1.303 t/m
E_3=0.268	Z_2=0.146	H_3=0.414	
E_4=1.060	Z_3=0.306	H_4=1.366	
E_5=0.171		H_5=0.171	$\sum_{block\ 1}$=3.254 t/m
E_6=1.785	Z_4=0.425	H_6=2.210	
E_7=0.277		H_7=0.277	$\sum_{block\ 2}$=5.741 t/m

Figure A.3 shows forces acting on block type quay wall. if earthquake is considered.

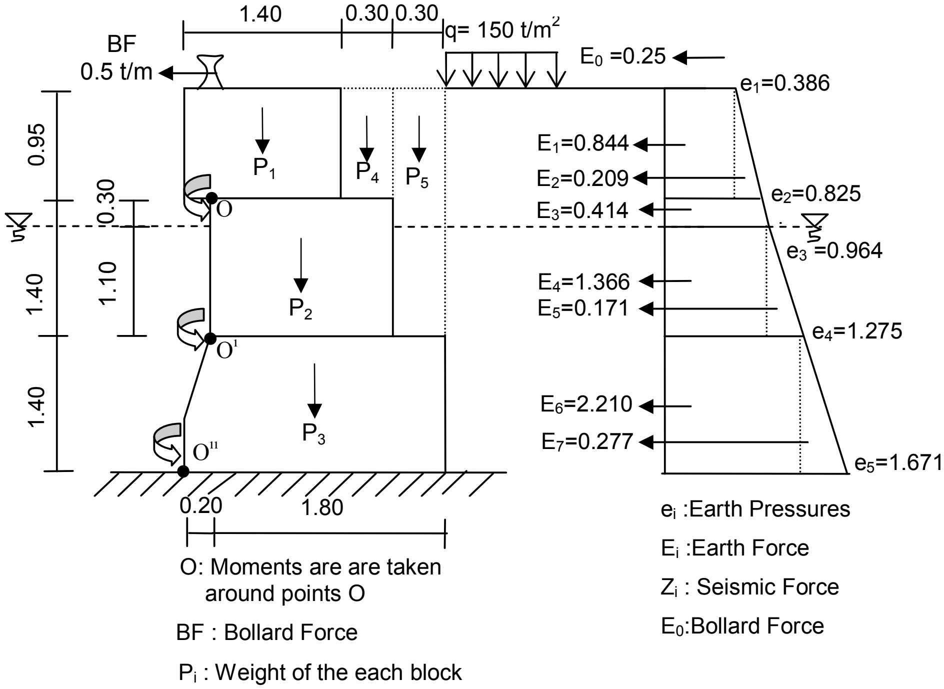


Figure A.3: Forces acting on block type quay wall. if earthquake is considered.

10. STABILITY COMPUTATION

10.1 CROWN WALL

$\sum$ **Horizontal Force** = $\sum_{\text{crown wall}} = H_0 + H_1 + H_2$

$= 0.250+0.844+0.209 = 1.303$ t/m

$\sum$ **Vertical Force** = $P_C = P_1 = 3.059$ t/m

Overturning Moment $= M_{oc} = \sum_{i=0}^{2} H_i r_i$

$=(0.250)(0.95+0.25)+(0.844)(0.95/2)$

$+(0.209)(0.95/3) = 0.767$ tm/m

Resisting Moment $= M_{rc} = \sum_{i=1}^{1} P_i r_i$

$=(3.059)(1.40/2-0.20)=1.530$ tm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s=\frac{(0.6)(3.059)}{1.303}=1.41 > 1.20 \quad \text{(ok)}$$

$$FS_o=\frac{1.530}{0.767}=1.99 \quad > 1.50 \quad \text{(ok)}$$

$$e=\frac{1.20}{2}-\frac{1.530-0.767}{3.059}=0.35<\frac{1.20}{3}=0.40 \quad \text{(ok)}$$

10.2 BLOCK 1

$\sum$ **Horizontal Force** = $\sum_{\text{block 1}} = \sum_{\text{crown wall}} + H_3 + H_4 + H_5$

$= 1.303+0.414+1.366+0.171=3.254$ t/m

$\sum$ **Vertical Force** = $P_{B1} = P_1+P_2+P_3$

$=3.059+3.816+1.026= 7.901$ t/m

Overturning Moment = $M_{o1} = M_{oc}+(\sum_{\text{crown wall}} h)+\sum_{i=3}^{5} H_i r_i$

$= 0.767+(1.303)(1.40)+((0.414)(1.25)$

$+(1.366)(0.55)+(0.171)(1.10/3) = 3.923$ tm/m

Resisting Moment = $M_{r1} = M_{rc}+\sum_{i=2}^{3} P_i r_i$

$= 1.530+(3.816)(1.80/2)+(1.026)(0.60/2+1.20)$

$= 6.503$ tm/m

FACTOR of SLIDING and OVERTURNING

$$FS_s=\frac{(0.52)(7.901)}{3.254}=1.26 \quad > 1.20 \quad \text{(ok)}$$

$$FS_o=\frac{6.503}{3.923}=1.66 \quad > 1.50 \quad \text{(ok)}$$

$$e=\frac{1.80}{2}-\frac{6.503-3.923}{7.901}=0.57<\frac{1.80}{3}=0.60 \quad \text{(ok)}$$

10.3 BLOCK 2

$\sum$ **Horizontal Force** = $\sum_{block\ 2} = \sum_{block1} + H_6 + H_7$

$= 3.254 + 2.210 + 0.277 = 5.741$ t/m

$\sum$ **Vertical Force** = $P_{B2} = P_{B1} + P_4 + P_5$

$= 7.901 + 4.251 + 1.384 = 13.536$ t/m

Overturning Moment = $M_{o2} = M_{o1} + (\sum_{block1} h) + \sum_{i=6}^{7} H_i r_i$

$= 3.923 + (3.254)(1.40) + (2.210)(1.40/2)$

$+ (0.277)(1.40/3) = 10.155$ tm/m

Resisting Moment = $M_{r2} = M_{r1} + \sum_{i=1}^{3}(P_i)(0,20) + \sum_{i=4}^{5} P_i r_i$

$= 6.503 + (7.901)(0.20) + (4.251)(1.23)$

$+ (1.384)(2.20) = 16.356$ tm/m

There is 0.20m difference between the moment points' of block1 and block2 !!

FACTOR of SLIDING and OVERTURNING

$$FS_s = \frac{(0.52)(13.536)}{5.741} = 1.23 \ > 1.20 \quad \text{(ok)}$$

$$FS_o = \frac{16.356}{10.155} = 1.61 \ > 1.50 \quad \text{(ok)}$$

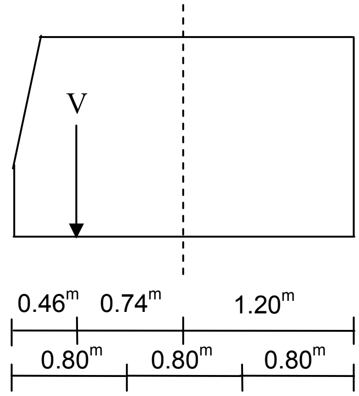
$$e = \frac{2.40}{2} - \frac{16.356 - 10.155}{13.536} = 0.74 < \frac{2.40}{3} = 0.80 \quad \text{(ok)}$$

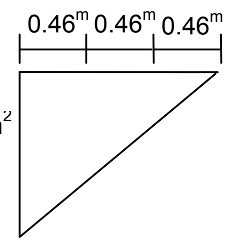
10.4 FOUNDATION BEARING CAPACITY

$$e = 0.74 > b/6 = \frac{2.40}{6} = 0.4$$

$$a = \frac{M_s - M_o}{V} = \frac{16.356 - 10.155}{13.536} = 0.46$$

a=0.46 m → Resultant is outside the core!





$19.617\ t/m^2$

$$\text{Area} = \frac{(19.167)(0.46)(3)}{2} = 13.536 m^2$$

Assume $\sum V$ = area of triangle

$$V = \frac{1}{2} q_{max} 3a \Rightarrow 13.536 = \frac{1}{2}(q_{max})(3)(0.46) \Rightarrow q_{max} = 19.617 t/m^2$$

Medium dense sand bearing capacity = (10.7 -32.2) t/m^2

Average bearing capacity can be taken as : (32.2+10.7) $\cong$ 21.5 t/m^2

q_{max}= 19.617 t/m^2 < 21.5 t/m^2 (ok)

APPENDIX B

CONVENTIONAL METHOD, 1997

B.1. FORCES

1. Active Earth Pressure

Coulomb Theorem:

$$K_A = \frac{\cos^2(\phi-\theta)}{\cos^2(\theta)\cos(\theta+\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right]^2} \tag{2.1}$$

where K_A is active earth pressure coefficient, P_A is active earth pressure, θ is angle between the back of the retaining wall and the vertical plane, δ is friction angle between wall and soil, β is inclined angle with the horizontal, ϕ internal friction angle.

- If the earthquake is considered K_{AE} value should be calculated by using the Mononobe Okabe Method.

2. Active Dynamic Earth Pressure

Mononobe-Okabe Method:

$$K_{AE} = \frac{(1-k_v)\cos^2(\phi-\theta-\psi)}{\cos\psi\cos^2(\theta)\cos(\theta+\delta+\psi)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\psi)}{\cos(\delta+\theta+\psi)\cos(\beta-\theta)}}\right]^2} \tag{2.2}$$

where K_{AE} is the dynamic active earth pressure coefficient, P_{AE} is the total active thrust (kN/m), γ is the unit weight of soil (kN/m^3), ϕ is the internal friction angle of soil (deg), θ is the angle between the back of the retaining wall and the vertical plane (deg), ψ is the seismic inertia angle, δ is the friction angle between wall and soil, k_v is the vertical seismic coefficient and H is the height of the structure.

- Seismic angle (ψ) is necessary in order to calculate K_{AE}. And to find the ψ value, horizontal earthquake coefficient (k_h) should be evaluated.

$$\psi = \arctan\left[\frac{k_h}{(1 \pm k_v)}\right] \text{ for dry soil} \tag{2.3}$$

$$\psi = \arctan\left[\frac{\gamma_s}{\gamma_d}\frac{k_h}{(1 \pm k_v)}\right] \text{ for saturated soil} \tag{2.4}$$

Horizontal earthquake coefficient (k_h);

$$k_h = 0,2(I+1)A_0 \tag{2.5}$$

where, I is structure importance coefficient, A_0 is effective ground acceleration coefficient

$$k_v=2/3\ k_h \tag{2.6}$$

$k_v \cong 0$ (Too small)

So, two different K_{AE} values are available, one of them is for dry soil and the other one is for saturated soil. These two K_{AE} values are utilized to compute two different K_d values for dry and saturated soil.

$$K_{d1}= K_{AE1}-K_A \quad \text{(for dry soil)} \tag{2.7}$$

$$K_{d2}= K_{AE2}-K_A \quad \text{(for saturated soil)} \tag{2.8}$$

where K_d is dynamic active pressure coefficient

3. Horizontal Loads

Static earth thrust is calculated using the K_A value.

- Additional dynamic active earth pressure and earth thrust is computed using the Eq.(2.9) and Eq.(2.10)

$$p_{ad}(z) = 3K_d(1-z/H)p_v(z) \tag{2.9}$$

$$P_{ad}=0,5\,\gamma\,K_d H^2 \tag{2.10}$$

$$z_{cd}=H/2 \tag{2.11}$$

If the soil is dry and uniform, $p_v(z)=\gamma z$

If the soil under the water , $p_v(z)=\gamma_b z$

If the soil is saturated , $p_v(z)=\gamma_s z$

$p_{ad}(z)$ is the exchange function of dynamic active pressure due to the weight of the soil, P_{ad} is the dynamic active force due to the weight of the soil, γ is the unit weight of dry soil, γ_b is the unit weight of soil under the water and γ_s is the unit weight of saturated soil.

- Additional exchange of dynamic active pressure due to the uniform surcharge load throughout the height of the structure and dynamic active force are computed using the Eq.(2.12) and Eq.(2.13)

$$q_{ad}(z) = 2q_o K_d (1 - z/H)\cos\theta / \cos(\theta - \beta) \tag{2.12}$$

$$Q_{ad} = q_o K_d H \cos\theta / \cos(\theta - \beta) \tag{2.13}$$

$$z_{cd}=H/3 \tag{2.14}$$

$q_{ad}(z)$: Exchange function of dynamic active pressure due to the uniform surcharge load throughout the height of the structure
Q_{ad} : Dynamic active force due to the uniform surcharge load

4. Vertical Forces

The weight of the blocks and soils are taken as vertical loads. During these calculations buoyancy force is taken into consideration.

5. Stability Computation

In stability calculations of gravity type quay wall, the following items should be examined;

According to TSS, 1997 the factor of safety against *sliding and overturning* for normal condition are taken as;
FS_s=1.5, FS_o= 1.5

According to TSS, 1997, the factor of safety against *sliding and overturning* for seismic condition are taken as;

FS_s =1.0, FS_o= 1.0