DEVELOPMENT OF A STABILITY ANALYSIS PROGRAM FOR BLOCK TYPE QUAY WALLS AND COMPARISON OF BLOCK PLACING METHODS

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ABSTRACT

DEVELOPMENT OF A STABILITY ANALYSIS PROGRAM
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COMPARISON OF BLOCK PLACING METHODS

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Block type quay walls are commonly used as berthing structures both in Turkey and worldwide. In this study, stability analysis of block type quay wall is carried out using pseudo-static method. A computer program named QSAP (using Excel spreadsheet) has been developed for the design of block type quay walls. QSAP has been prepared based on the rules of Turkish Seismic Design Codes for Coastal Structures, 2008. Reliability of this program is verified by a comparative study of Derince Port block type quay wall, damaged in Marmara earthquake (1999), with manual solution and field measurements. A newly introduced placement methodology "Knapsack" is also studied with QSAP and the results are compared with the conventional placement method.

Keywords: Block type quay walls, stability analysis, knapsack section, conventional section, pseudo-static method.

ÖZ

BLOK TİPİ RIHTIM DUVARLARININ STABİLİTE ANALİZİ İÇİN BİLGİSAYAR PROGRAMI GELİŞTİRİLMESİ VE BLOK YERLEŞTİRME METOTLARININ KARŞILAŞTIRILMASI

NERGİZ, Cengiz
Yüksek Lisans, İnşaat Mühendisliği
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Blok tipi rıhtım duvarları Türkiye'de ve dünyada bağlanma yapısı olarak sıkça kullanılan duvarlardır. Bu çalışmada blok tipi rıhtım duvarlarının stabilite analizi pseudo-statik metot kullanılarak yapılmıştır. Blok tipi rıhtım duvarlarının tasarımı için QSAP (Excel programı kullanılarak) adında bir program geliştirilmiştir. QSAP, Kıyı ve Liman Yapıları Deprem Teknik Yönetmeliğine (2008) uygun çözüm almaktadır. Programın güvenilirlik kontrolleri için 1999 Marmara depreminde zarar gören Derince Limanı blok tipi rıhtım duvarı için yapılmış elle çözüm ve saha ölçümleri kullanılmıştır. Yeni bir yerleştirme metodu olarak önerilen "sırtçantası" metodu da QSAP kullanılarak incelenmiş ve geleneksel şekilde yerleştirilmiş blok tipi rıhtım duvarı ile karşılaştırması yapılmıştır.

Anahtar Kelimeler: Blok tipi rıhtım duvarlar, stabilite analizi, sırtçantası kesiti, geleneksel kesit, pseudo-statik metot.

To My Wife and My Parents...

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a_{max}	Peak ground surface acceleration
a_h	Horizontal pseudo-static acceleration
A	Area of the block
A_{10}	Effective seismic coefficient (a_{max} / g) for L1
A_{20}	Effective seismic coefficient (a_{max} / g) for L2
B	Base length of the wall
BF	Bollard force
c	Distance between total vertical force application point and edge of the structure
e	Eccentricity
e_{xi}	Moment arm for horizontal force
e_{yi}	Moment arm for vertical force
$\sum F$	Total force
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
$\sum F_h$	Total horizontal force
$F_{i(de)}$	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

$\sum F_v$ Total vertical force

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 of block i

h_j Thickness of the soil class j

H Wall height

H_w Total water depth

H_{sub} Submerged soil height

k_h Horizontal seismic coefficient

k_v Vertical seismic coefficient

$k_h^{'}$ Modified horizontal seismic coefficient

K_A Static active pressure coefficient

K_{AE} Total active pressure coefficient

K_{ai} Active pressure coefficient for soil class i

$K_{ai,d}$ Dynamic active pressure coefficient for soil class i

K_d Dynamic active pressure coefficient

$\sum M_r$ Sum of the resisting moments around the toe of the wall

$\sum M_o$ Sum of the overturning moments around the toe of the wall

ND Number of the dry soil class according to ground level

P_A Active earth pressure

P_{AE} Total active thrust

p_{ai} Active pressure for soil class i

$p_{ai,s}$ Static active earth pressure for soil class i

$p_{ai,d}$ Dynamic active earth pressure for soil class i

$p_{ai,ds}$ Dynamic surcharge earth pressure for soil class i

P_{dwi} Hydrodynamic pressure for block i

P_i Weight of 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

q_{max} Maximum foundation reaction

q_{min} Minimum foundation reaction

q_o Surcharge load

QSAP Block type quay wall stability analysis program

S_{MS} Spectral acceleration value with respect to soil classification

S_s Short period spectral acceleration

SWL Still water level

W Weight of the failure mass

α Angle between the back of the retaining wall and the vertical plane

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_b Submerged unit weight of soil

γ_{bj} Submerged unit weight of soil for soil class j

γ_d Dry unit weight of soil (kN/m^3)

γ_j Dry unit weight of soil for soil class j

γ_s Submerged unit weight of soil (kN/m^3)

γ_{sj} Saturated unit weight of soil for soil class j

γ_{sat} Saturated unit weight of backfill soil (kN/m^3)

γ_w Unit weight of seawater

ϕ Internal friction angle

δ Friction angle between wall and soil

μ Coefficient of friction between the bottom of the wall body and the foundation

$\mu_{block1-2}$ Coefficient of friction between block-1 and block-2

$\mu_{block2-N}$ Coefficient of friction between block-2 and block-N

$\mu_{block-found}$ Coefficient of friction between block and foundation

λ Seismic inertia angle

λ_{dry} Dry seismic inertia angle

λ_{sat} Saturated seismic inertia angle

CHAPTER 1

INTRODUCTION

From early eras, sailing over water has become important issue for people. Not only the economical reasons but also curiosity motivated human to explore new lands. Traders and explorers sailing the ocean needed places to berth their ships safely. Historically, port developments and their evolution have begun from that necessity. Ports and harbors provide economical and cultural growth for the population living there. On the other hand these water structures are vulnerable to different kinds of hazards. Earthquakes, tsunamis and landslides are the major hazard sources for these structures.

Although, probability of experiencing a large earthquake in the lifetime of the structure is low, effects may be devastating both economically and socially in the case of occurring this natural event. Adopting seismic provisions to design practice has been realized since mid-twentieth century. However, lack of introducing seismic risks to the design practice caused seriously damaged port structures from earthquakes in the history. For example, Kushiro Port, Kobe Port, Oackland Port, Port Villa, Derince Port, etc.[1]

Marmara earthquake occurred on 17 August 1999 with an Mw=7.4 and İzmit Bay and north-west Turkey had been seriously affected from this earthquake. Especially, earthquake caused crucial damage mostly on bock type quay walls at Derince Port in İzmit [2].

Due to large investment needs and critical infrastructure characteristics, the design and construction of a quay wall, which is earth retaining berth structure, becomes more interesting and complicated day by day. Concrete block quay wall, which is the most common and simplest gravity type quay wall, is composed of concrete blocks with various shapes sitting on each other to form the wall section. Stability is maintained by friction among blocks and between the bottom block and seabed.

In the block type quay wall design, Pseudo-static Method is used. In this method seismic force is 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 [1].

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

In Chapter 3, important studies and researches on block type quay wall stability are evaluated.

In Chapter 4, Turkish seismic design code for coastal structures and its theoretical background are explained in detail.

In Chapter 5, block type Quay wall Stability Analysis Program (QSAP) is presented with its assumptions and program directions for use.

In Chapter 6, a case study is carried out in order to verify the program output data with manual solution of a block type quay wall structure.

In Chapter 7, by using QSAP, stability comparison study is performed between knapsack shape and conventional shape block type quay wall.

In Chapter 8, conclusion is presented and future studies are recommended.

CHAPTER 2

BERTH STRUCTURES

2.1 Historical Background of Berth Structures

A berth structure is a vertical front, which allows ships to berth safely. A quay wall is the oldest example of an earth retaining berth structure with a deck on top, serving both purposes of protecting the shoreline and providing a landing site for cargo and passengers. Earth is usually placed behind the wall so that the deck can be built to the required height from the water level. Whereas dredging to some extent may be necessary in front of the wall so that the required water depth can be obtained. The structure, including foundation, is to be capable of providing stability and meeting various requirements such as soil conditions, water levels, size of ships and loads in addition to specific requirements of the users and quay managers. The growth of the navigation will likely result in increased importance of quay walls in the future design and construction of quays, which is already not simple [3].

Traders and explorers have been sailing, an economical means of transportation, over the oceans to explore and conquer the world. Harbors have developed from the need for places to berth their ships, and provided the sailors with increased wealth and also in knowledge on new technologies. The oldest port near Lothal, India, was built in 2000 BC to serve the trade among Asian countries. Alexandria port was the main trade center in the Mediterranean during three ages before Christ, when the Romans used concrete of some kind to construct quay walls.

The Vikings sailed on fast ships over the West European waters in the Middle Ages when two major problems, namely siltation of harbors and inadequate equipment to hoist the cargo where encountered. Neither quay walls nor any equipment to transship the cargo in harbor were available. In the course of time, cranes were used to serve the purpose but causing another problem of requirement of firm subsoil to erect the cranes. This requirement played major role in the development of quay walls with vertical bearing capacity [3].

Introduction of steam engine in the 19^{th} century paved the way to a large growth in the ship tonnage in the 20^{th} century, resulting in sailing with larger draught, which greatly affects the retaining height of the quay wall. Other results are the increased possibility of self-berthing and additional scour because of propeller currents. Transshipment method is also changed, allowing transshipment of heavier loads at the quay and necessitating construction of larger quay walls. Therefore, quay walls piled up by stones on the basis of a complex design were constructed.

2.2 Functions of Berth Structures

Berth structures of various types serve as a mooring site for ships further to such functions as soil-retaining function, providing bearing capacity for crane loads, goods and storage, and occasionally a water retaining function.

Figure 2.1: Roman ship transporting from Egypt to port of Ostia [3]

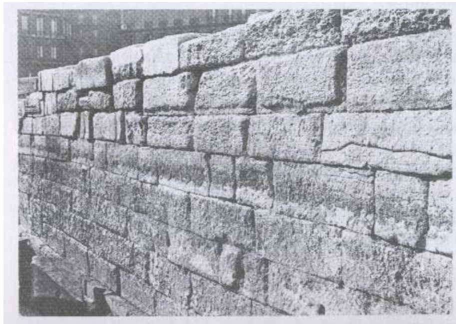


Figure 2.2: The exposure of quay wall close to Marseilles, constructed BC [3]

2.3 Features of Berth Structures

Requirements of a berth structure vary depending on requirements of the user as for freight handling, adequate storage area, adequate bearing capacity for future storage and transport and adequate draught for the largest ships to berth [3].

In addition, the following requirements must be taken into consideration during design and construction:

- Optimum quality at reasonable cost,
- Low maintenance and long-lifetime,
- Sufficient elevation to keep the structure dry at tide,
- Optimum design based on vast experience, ingenuity and creativity taking into account the prevailing water levels, tidal influence, soil properties and climatic conditions.

In order to serve as an effective berthing facility for ships, the structure should retain the earth fill behind the quay or have load-bearing slab on columns, provide bearing capacity adequate to carry the loads imposed by cranes and freight storage facilities, further to potential use as a retaining wall during high tide.

2.4 Main Types of Berth Structures

The berth fronts are constructed according to one of the following two main principles, as illustrated in Figure 2.3.

A diagrammatic classification of berth structures according to type and construction method is shown in Figure 2.4.

Even if it is difficult to provide precise guidelines for selecting the optimum berth type for individual case, the factors mentioned in the following paragraphs should be taken into consideration.

Design and construction should be such that the structure resist safely to vertical loads resulting from live loads, vehicle traffic, cranes etc. in addition to horizontal loads resulting from earthquake, impacts of ships, wind, earth-fill behind the structure etc.

In general, solid berth structures are assumed to be more resistant to both vertical and horizontal loads than open berth structures. Solid structures are less sensitive to overloading, considering the dead weight of open structures. On the other hand, the safety factor for solid structures is normally lower than that for open structures [4].

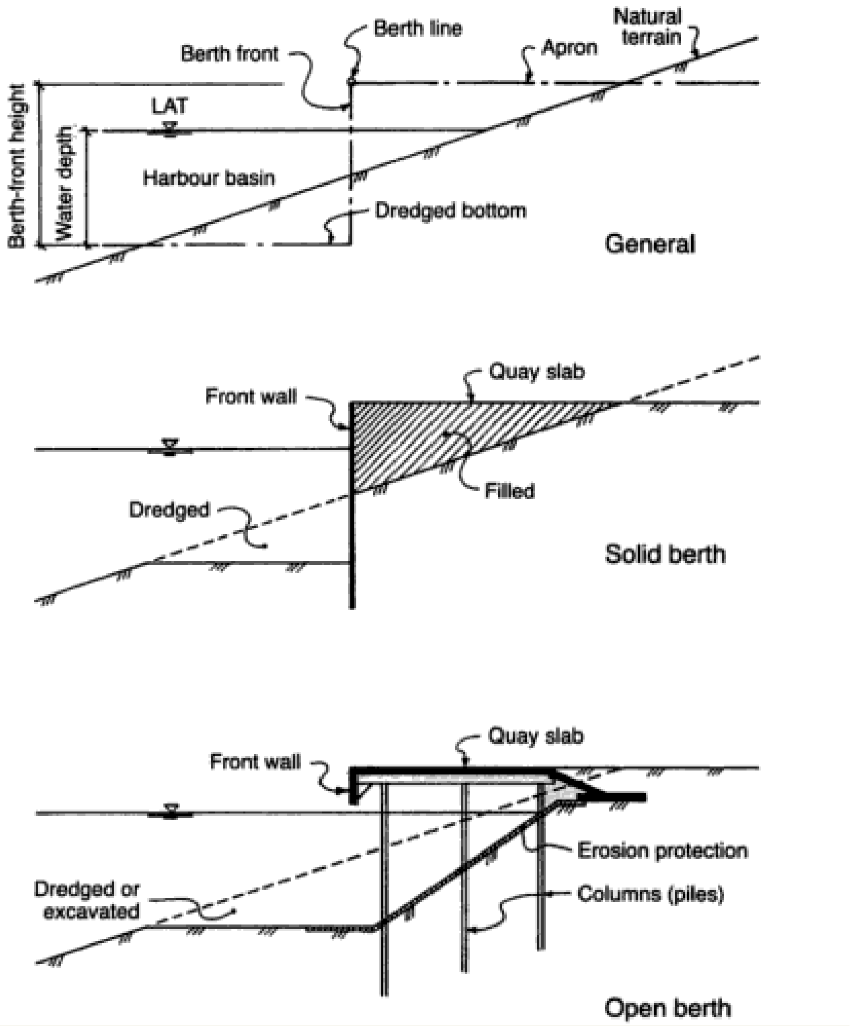


Figure 2.3: Terminology [4]

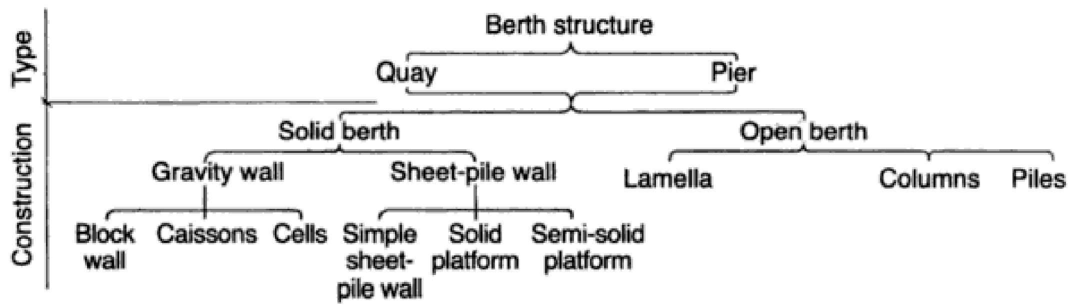


Figure 2.4: Types of berth structures [4]

Solid structures are generally more resistant to impact from ships than open structures. For instance, a block wall wharf is much more resistant than a pier built as an open berth on piles, except for an open berth on wooden piles where the entire structure is flexible and yields sufficiently to absorb most of the impact when ships come to berth alongside [4].

2.4.1 Solid Berth Structures

In order to resist any horizontal load from the earth fill and any live load on the apron, the earth fill must be extended to the berth front where a vertical front wall is built. Depending on the principle on which construction of front wall is based to provide adequate stability, solid berth structures can be studied in two groups [4].

2.4.1.1 Gravity Wall Structures (Quay Walls)

The front wall, including its deadweight and bottom friction, should be self-sufficient to resist the loadings from backfill, useful load and other horizontal and vertical loads acting on the berth wall. According to the type of structural design, gravity wall structures can be sub-divided into;

- Block walls
- L-walls
- Caisson walls
- Cellular walls

In general, gravity wall structure is adopted where seabed or ground conditions are favorable or improved and settlement poses low risk.

2.4.1.1.1 Block Walls

Consisting of large blocks placed in a masonry wall pattern, as illustrated in Figure 2.5, is the oldest type of berth structure. Such walls are built on firm ground, using high quality natural stone or concrete. They have long life and require less maintenance. Today, only concrete blocks are deemed economical due to high costs of natural stone mining [4].

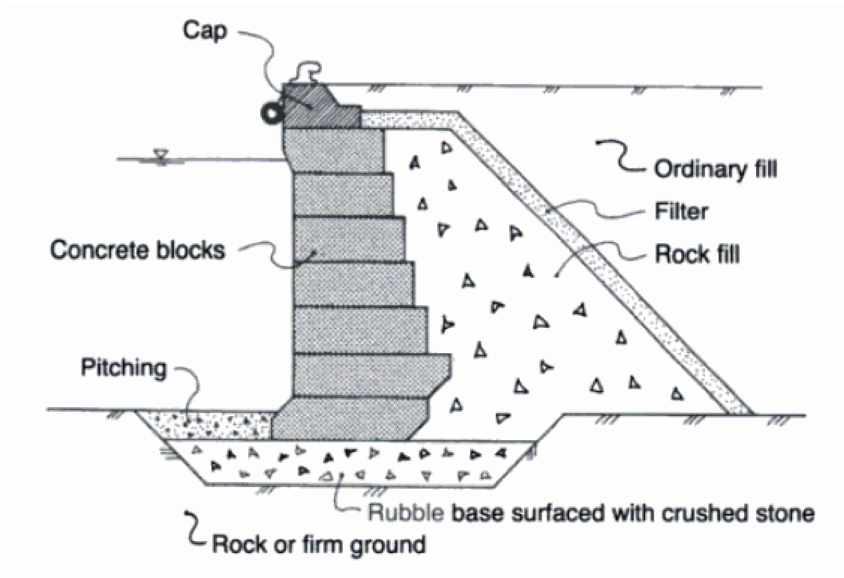


Figure 2.5: Typical section of block wall [4]

2.4.1.1.2 L-Walls

L-walls are walls constructed using reinforced concrete blocks instead of using concrete blocks. The reinforced concrete blocks are transferred and installed by cranes side by side on a gravel and/or rubble base prepared at the sea bed. Figure 2.6 shows the typical section of a L-wall.

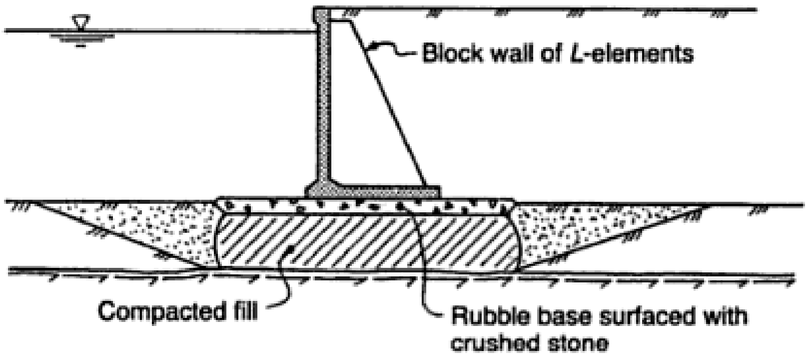


Figure 2.6: Typical section of L-wall [4]

2.4.1.1.3 Caisson Walls

Caisson walls consist of precast concrete caissons placed in a row to form the berth front. Caissons may vary in shape and design according to site conditions and available construction equipment [4]. The most common type is the rectangular caisson. Figure 2.7 illustrates typical section of a caisson wall.

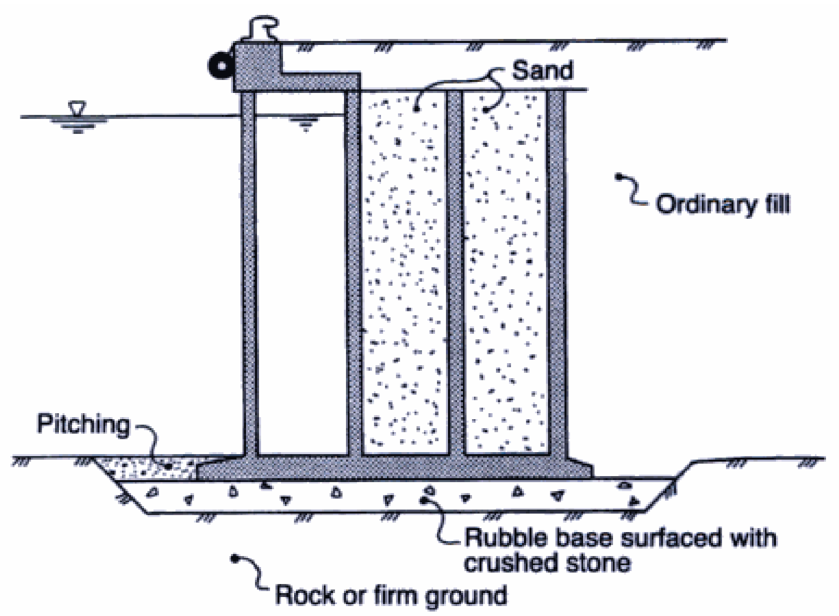


Figure 2.7: Typical section of caisson wall with three chambers [4]

2.4.1.1.4 Cellular Walls

Cellular walls consist of cells, which are constructed in water or on land and then filled with sand or any other material. Arched cells connect the circular main cells as the most common construction method. Advantage of circular cells is that each cell can be constructed and filled individually to become independently stable [4]. Figure 2.8 shows the commonly used design although cellular walls in different geometric configurations are used.

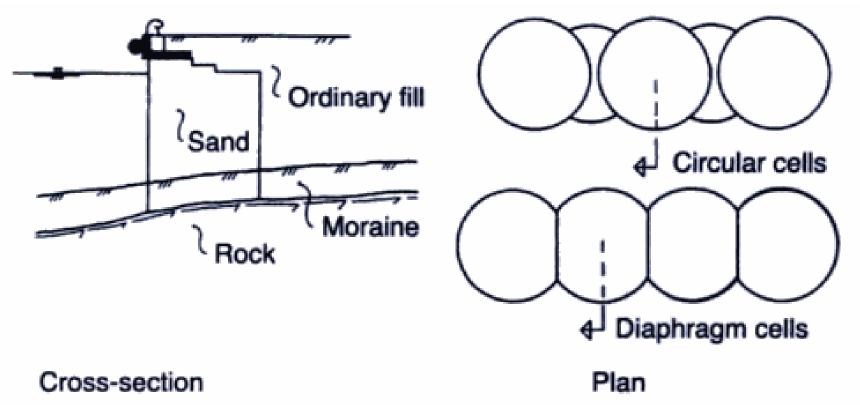


Figure 2.8: Typical section of cellular wall [4]

2.4.1.2 Sheet Pile Wall Structures

Since pile front walls cannot resist any horizontal load acting on the structure, they must be anchored to anchoring plates, walls or rocks behind the berth. As it is with gravity walls, sheet pile walls are sub-divided into;

- simple sheet pile walls
- solid platform
- semi-solid platform

according to the design [4].

2.4.1.2.1 Simple Sheet Pile Walls

Simple sheet pile wall is the most economical solution, as shown in Figure 2.9, where water is not very deep at the berth front and soil conditions are favorable.

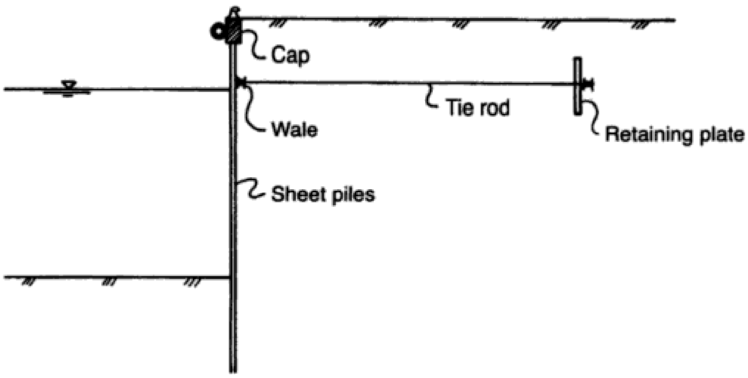


Figure 2.9: A simple sheet pile wall [4]

2.4.1.2.2 Solid Platform Walls

Solid platform wall is the most economical solution where the berth front exceeds about 8 – 10 m in height [4]. This type of structure, as shown in figure 2.10, consists of a relieving plate, which is placed on the wall and on piles behind the wall, to transmit to the wall and piles the useful and axial loads, e.g. the weight of the fill on top, and to reduce the horizontal load acting on the wall.

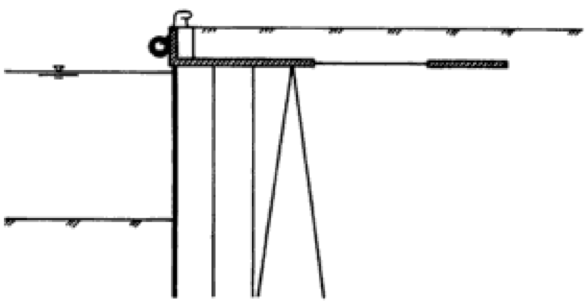


Figure 2.10: A solid platform wall [4]

2.4.1.2.3 Semi-Solid Platform Walls

Semi-solid platform walls may be best solution where a quay front greater than about 14 – 18 m in height is required [4]. This type of structure, as shown in Figure 2.11, is similar to solid platform type, except that the soil pressure on the sheet pile wall is much more reduced by decreasing the fill under the platform.

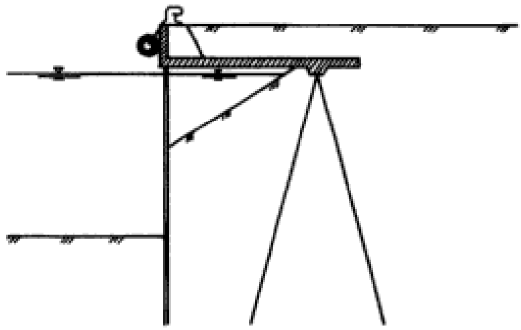


Figure 2.11: A semi-solid platform wall [4]

2.4.2 Open Berth Structures

An open berth structure consists of a load-bearing slab, lying between the top of a dredged or filled slope and the berth front. The slab is constructed on columns or lamella (plate) walls.

2.4.2.1 Lamella Berths

Vertical lamellas constitute a basis for the berth platform and berth front, providing the loaded berth structure with adequate stability. Figure 2.12 shows the typical section of a lamella berth.

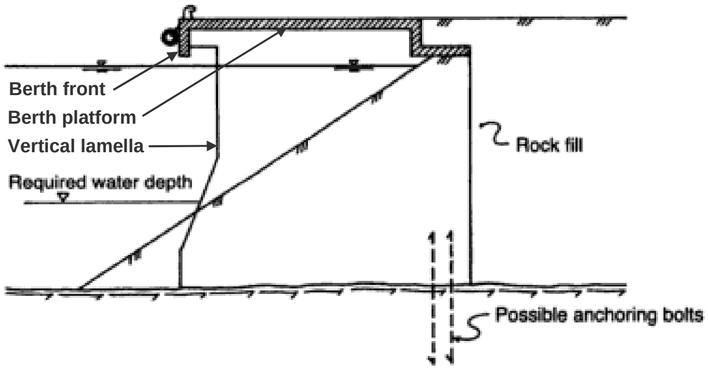


Figure 2.12: A lamella berth [4]

2.4.2.2 Column Berths

In-situ cast concrete columns serve as basis to the berth platform and front wall. Since these columns do not have adequate stability against external forces, the berth structure must be anchored in the filling, using any device such as friction plate [4]. Figure 2.13 shows the typical section of a column berth.

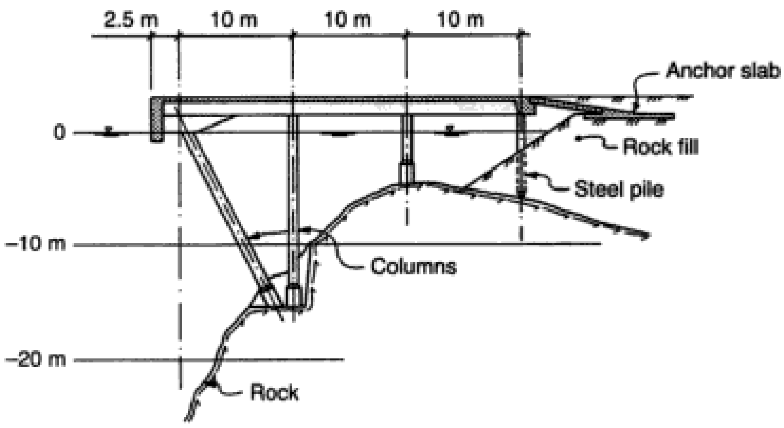


Figure 2.13: A column berth [4]

2.4.2.3 Pile Berths

Tubular steel pipes instead of concrete piles are used to drive into harder ground and to serve as basis to the berth platform and berth front [4]. As it is with column berths, the pile berths must be anchored using a friction plate in the fill. Figure 2.14 shows the typical section of a pile berth.

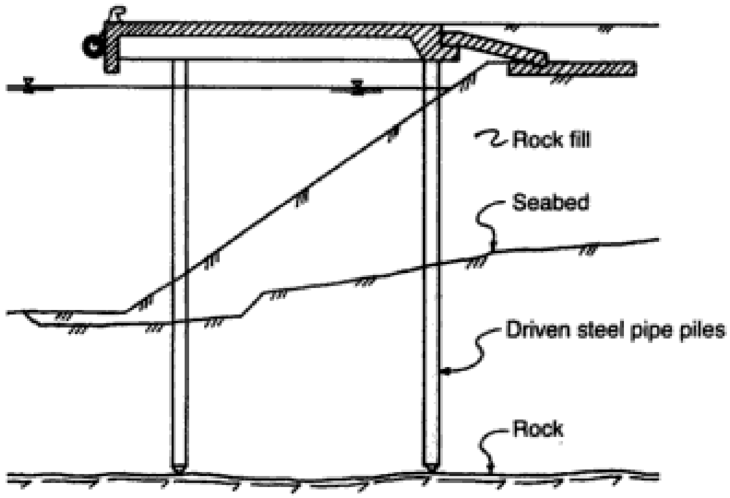


Figure 2.14: A pile berth [4]

2.5 Failure Types of Block Type Gravity Quay Walls

Figure 2.15 and Figure 2.16 show failure modes of block type gravity walls. A gravity quay wall resting on a firm foundation moves toward the sea when earth pressure from the backfill increases in addition to the effect of an inertia force on the wall body (see Figure 2.15). Tilt may also occur if the ratio of width to height of the wall is small.

Larger seaward movement, including tilt, settlement and relative movement of blocks, may occur if the subsoil under the gravity wall is loose and pore water pressure increases in the subsoil, resulting in significant deformation in the foundation soil (see Figure 2.16).

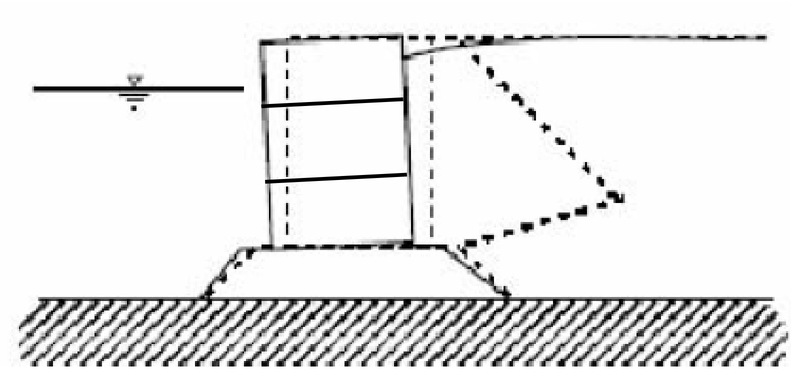


Figure 2.15: Deformation/failure modes of block type gravity quay wall on firm foundation [1]

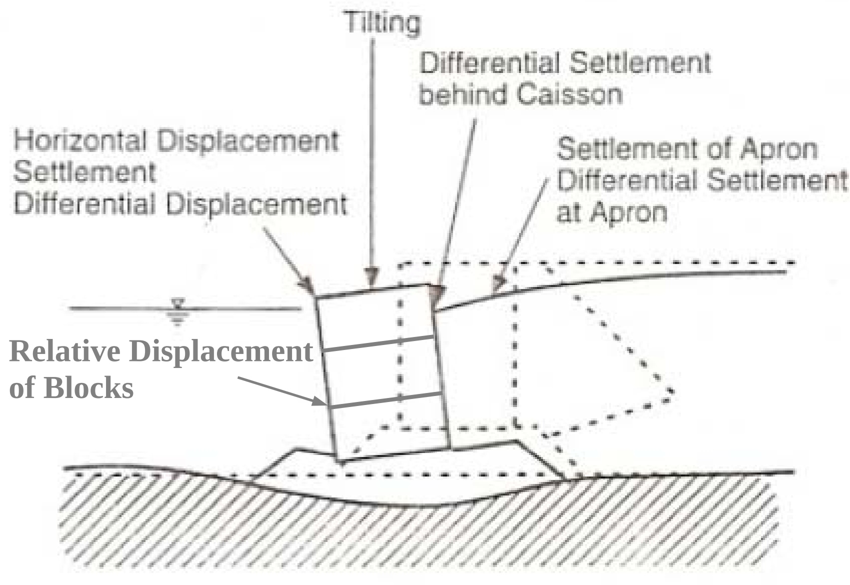


Figure 2.16: Deformation/failure modes of block type gravity quay wall on loose sandy foundation [1]

CHAPTER 3

LITERATURE SURVEY

Block type gravity quay walls are the commonly used and one of the oldest types of docks due to their durability, ease of construction and modest maintenance requirements. Instead of natural stone blocks, concrete blocks are being used nowadays as they are more economical considering the high costs of mining natural stone blocks.

Concrete block types of gravity quay walls have been used world wide with various wall heights. Derince Port (Turkey), San Antonio Port (Chile) Kalamata Port (Greece), Porto Arabia (Qatar), Sohar Port (Oman), Pars Petrochemical Port (Iran) are some of the examples of this kind. Jebel Ali Port, which is the largest commercial port in the Arabian Gulf is another major example. In Jebel Ali Port, maximum depth of 20.2 m has been reached with 13 no of concrete blocks with dimensions ranging between 6.2 m to 9.6 m in width, 1.25 m to 1.75 m in height [5].

In general, the gravity quay walls are designed in line with three design criteria and these can be listed as; sliding, overturning and over stressing. Together with the design calculations; geotechnical conditions and construction methodologies are also considered to determine the block dimensions to be used. Although the design steps of gravity quay walls seem straightforward, external forces increase as the depth of the quay wall increases. The trend of deep water depth vessels has made it difficult to satisfy equilibrium conditions for quite deep quay walls as well. In this case, the stability of the wall may be quite sensitive to many factors; depth of the wall, pulling force, soil characteristics; and base stratum characteristics. In their study, El-Sharnouby et al. [6] have investigated the factors affecting equilibrium condition and their influence on the final design by means of a computer program that has been developed and implemented for direct analysis of gravity quay wall.

One of the important outcomes of their study is that the backfill characteristics have a great influence on the stability of gravity quay wall. Increasing the width of the coarse backfill material to be similar to the wall height may be recommended which decreases the lateral pressures by a percentage of 25 %. The improper construct of the backfill or failing to do so, for any reason, may double the horizontal forces [6].

Other important outcome is that the ship draft is a quite essential factor because lateral pressures and instability moments dramatically increase with depth. A ship load and dimensions also affect the berth in terms of bollard force and magnitude of live load. Increasing the unit weight of the concrete also participates into the stability of the wall. Therefore, it is recommended to use blocks with unit weight not less than 2.4 t/m^3 particularly for deep walls [6].

It is also concluded that the dimensions of each block can be determined by checking sliding first then checking for overturning and stresses. If stresses on the rock base are not appropriate, the center of the last block can be moved toward the sea by reshaping the block. If this step is not enough, the second block from bottom can be enlarged and checked against shear failure. If more steps are needed we may increase the length of third block from the bottom to be (1m) longer than the length of second block from the bottom [6].

Another study has been carried out by Jalili et al. [7] on wall stability where the blocks are positioned in a different way called Knapsack placing. In general, placing of blocks can be done in two ways; conventional (vertical front face) (Figure 3.1) and knapsack (Figure 3.2) formation. In conventional formation each block will be placed on the lower block in such a way that the front face of blocks makes a vertical wall face and back face has stepped-like shape. As an alternative, especially in seismic regions, in order to reduce active soil pressure and increase the stability of the wall, the hunchback section is formed by placing a protruding knapsack on the landside [8].

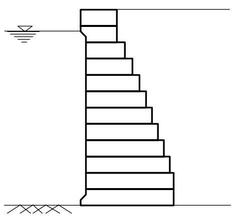


Figure 3.1 Conventional placing

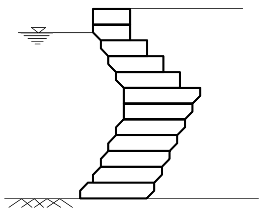


Figure 3.2 Knapsack placing

In the knapsack formation, due to the backward tendency of blocks, resistance moments and safety against overturning and eccentricity of resultant loads under earthquake forces will be increased. Special conditions such as trimming the block faces, more accuracy in setting the blocks and consequent increases in total construction costs should be considered [7].

In their study, Jalili et al. have evaluated a relative comparison of required concrete volume between knapsack and conventional sections of block quay walls by conducting pseudo-static analysis for different levels of earthquakes with different peak ground accelerations and for various quay wall heights.

One of the most important outcome of their study is that the proper section for various depths and seismic coefficients should be chosen with regards to relative comparisons between concrete volumes, required construction equipment, transportation and installation of blocks limitations and construction problems related to block formation. In cases of using powerful cranes for moving heavy blocks, low concrete costs and accessibility in project location, using conventional formation in lower depths (less than -7 m CD) will be a good choice in comparison to knapsack formation which needs more attention to blocks placement. On the other hand, in deeper elevations because of the considerable increment in the number of blocks, dimensions for conventional sections and necessity of particular equipments to transport and form the blocks, using knapsack formation is preferable [7].

Knapsack shape is the suitable section for block quay walls in seismic zones. However, it is necessary to make a comparison between proportions of concrete volumes of knapsack formation and the conventional formation for economic and feasibility evaluation. In other words, if the study indicates that only a small reduction in concrete volume will be achieved, then it is more feasible to build the section with conventional formation. In their study Jalili et al. [7] concluded that the use of knapsack shape for more than 7 m water depth strongly recommended for whole level of seismicity considering construction limitations such as required equipment for portage of large blocks. Other conclusions of their study can be listed as;

- In all seismic zones, with seismic coefficient values from 0.12 to 0.225, using knapsack shape results in decreasing the required concrete volume of the quay wall section.
- In the low seismicity earthquake zones with (k_h=0.12) reduction in blocks dimensions and volume of concrete with increasing in quay wall water depth in knapsack section is 2.5 times of that in high seismicity earthquake zones with (k_h =0.225).
- Percent of reduction in concrete volume of knapsack shape in relation to conventional section increases with quay wall water depth increasing.

Sadrekarimi et al. [9] have studied the static and dynamic behavior of hunchbacked gravity quay walls by considering the back-face shape of the wall which is one of the parameters that can affect the lateral pressures behind a retaining wall. In order to study this behavior they have carried out a set of 1 g shaking table tests on hunched back gravity type quay wall made of concrete blocks. The back-face slope of a hunchbacked wall was divided into two parts; negative back-slope, which is the elevation below the breaking point of hunch and positive back-slope, which is the elevation above the breaking point of hunch.

The outcomes of their study show that the negative back-slope reduces the lateral earth pressures and the positive back-slope increases them, which is consistent with Coulomb's lateral earth pressure theory [9].

This made a very different pressure distribution pattern behind the hunchbacked wall in comparison to the typical triangular pattern, which develops behind vertical back-face walls, causing the static thrust both before and after an earthquake to act at a higher level behind the hunchbacked wall.

In order to have a safer hunchback wall, the negative and positive back-slope parts should be chosen carefully. In addition to this advantage of a hunchback wall, is the stabilizing effect of the weight of the soil leaning on the positive back-slope of the wall, which reduces sliding and overturning of the wall. Also there is a region of safety just behind a hunchbacked wall that has a comparatively less settlement where industrial and coastal structures can be located [9].

CHAPTER 4

METHODOLOGY

Design Method Based on Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2008 (TSDC-CRA, 2008)

The conventional practice for evaluating seismic stability of gravity quay walls is based on pseudo-static approaches, despite the fact that the actual dynamic behavior is rather complex than it is treated in pseudo-static approach. In this procedure, a seismic coefficient, expressed in terms of the acceleration of gravity, is used to compute an equivalent pseudo-static inertia force for use in analysis and design [1]. In the pseudo-static approach, the stability of gravity quay walls is evaluated with respect to sliding, overturning and bearing capacity.

Horizontal and vertical inertial forces are assumed to act through the center of the failure mass [10]. The vertical pseudo-static force has less effect on the factor of safety; it can reduce or increase (depending on the direction) both the driving force and the resisting force. Thus, the influence of vertical acceleration is ignored in pseudo-static analysis [11].

4.1 Design Parameters and Basic Assumptions

The parameters contributing to the design of block type quay walls can be divided into three main groups such as geotechnical parameters, geometric parameters and load-related parameters [2].

Geotechnical parameters include; cohesion of soil, internal friction angle of soil, unit weight of soil and wall friction angle. Geometric parameters include; wall height, still water level, angle between the back of the wall and the vertical plane, inclined angle with the horizontal. Load-related parameters include; horizontal seismic coefficient, weight of the soil, surcharge on the ground surface behind the wall, bollard force and the active earth pressure.

Some assumptions need to be specified in order to define the design parameters [2]. The basic assumptions used in the design development stages are as follows;

- It is assumed that existing soil conditions are improved and are expected not to lead any unsatisfactory results. The most common soil improvement techniques can be classified as; densification techniques, drainage techniques, grouting techniques and reinforcement techniques. No liquefaction condition is satisfied.
- In order to develop the full active earth pressure wall movement must occur.
- Active earth pressure is a function of earth pressure coefficient, height of the wall and unit weight of the soil.
- The backfill material is assumed to be granular (cohesionless) soil.

▪ In this thesis, passive earth pressure is neglected and only active earth pressure is studied.

4.2 Types of Forces Acting on the Structure

Forces acting on the block type quay wall are divided in to two as horizontal and vertical forces;

4.2.1 Horizontal Forces

Horizontal forces acting on the block type quay wall are further divided into two as static and dynamic forces;

4.2.1.1 Static Forces

Static horizontal forces are further divided into three parts; active earth force (F_{ise}), surcharge (F_{iss}), hydrostatic force. The schematic presentation of these forces can be seen in Figure 4.1.

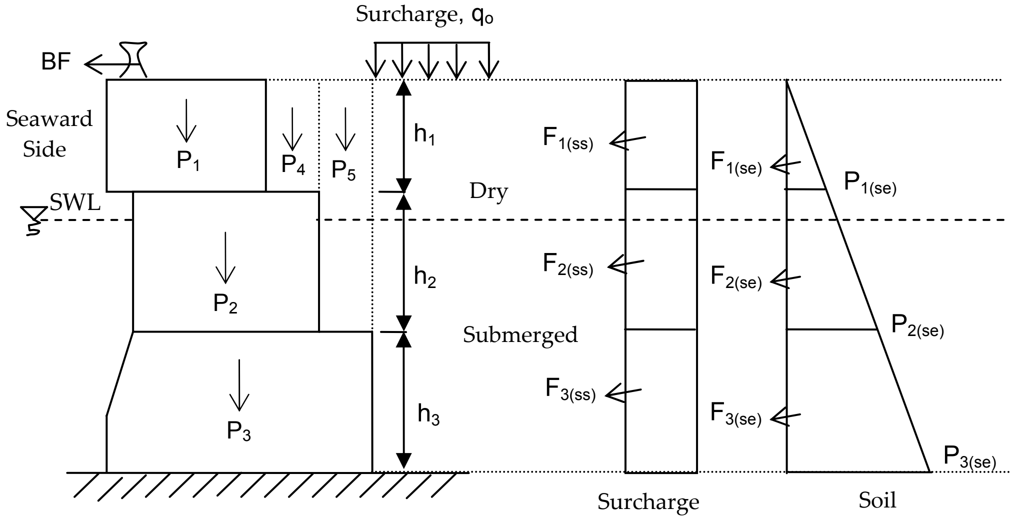


Figure 4.1: Static forces acting on block type quay wall [2]

where,

P_i Weight of each block

h_i Height of block i

q_0 Surcharge load

$F_{i(se)}$ Static earth force for block i

$F_{i(ss)}$ Static surcharge force for block i

$P_{i(se)}$ Static earth pressure for block i

BF Bollard force

4.2.1.1.1 Active Earth Force

Active earth pressure is calculated by using Coulomb approach, which is based on the limit-equilibrium method.

Using Coulomb expression, P_A and K_A are defined as follows;

$$P_A = \frac{1}{2} K_A \gamma H^2 \tag{4.1}$$

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

where,

P_A Active earth pressure

K_A Static active pressure coefficient

γ Unit weight of soil

H Wall height

ϕ Internal friction angle

α Angle between back of the retaining wall and the vertical plane

δ Friction angle between wall and soil

β Inclined angle with the horizontal

Static active pressure coefficient (K_A) includes unknown parameters related to property of soil and relation between the soil and blocks. 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 [2].

For each block the forces can be calculated by using the following formulas;

$$P_{i(se)}=K_A \gamma h_i \qquad (4.3)$$

$$F_{i\,(se)} = \left(\frac{P_{i(se)} + P_{i-1(se)}}{2} \right) h_i \qquad (4.4)$$

where,

$P_{i(se)}$ Static earth pressure for block i

h_i Height of block i

$F_{i(se)}$ Static earth force for block i

4.2.1.1.2 Surcharge

Assuming that the surcharge is applied to a certain distance from the top of the structure and is uniformly distributed, and adopting the dispersal influence of surcharge like a rectangular shape the forces for each block can be calculated by using the following formulas [2];

$$P_{i(ss)} = q_0 \, K_A \qquad (4.5)$$

$$F_{i(ss)} = P_{i(ss)} \, h_i \qquad (4.6)$$

where,

$P_{i(ss)}$	Static surcharge pressure for block i
q_0	Surcharge load
K_A	Static active pressure coefficient
$F_{i(ss)}$	Static surcharge force for block i
h_i	Height of block i

4.2.1.1.3 Hydrostatic Force

Hydrostatic forces acting on the blocks are not considered as it is assumed that the water levels on both sides of the wall are same. The hydrostatic pressure due to variation of water table is assumed negligible and thus not considered.

4.2.1.2 Dynamic Forces

Dynamic horizontal forces are divided into four parts; dynamic earth force, dynamic surcharge force, hydrodynamic forces and inertia forces.

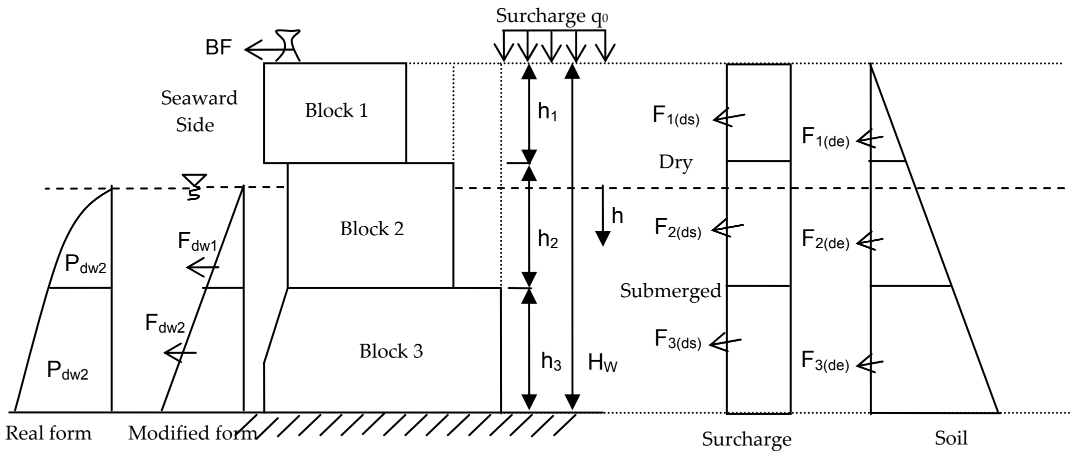


Figure 4.2: Dynamic forces acting on block type quay wall [2]

where,

P_{dwi}	Hydrodynamic pressure for block i
F_{dwi}	Hydrodynamic force
h_i	Height of block i
q_0	Surcharge load
$F_{i(ds)}$	Dynamic surcharge force for block i
$F_{i(de)}$	Dynamic earth force for block i
H_w	Total water depth
h	Water depth for each block

4.2.1.2.1 Dynamic Earth Force

During an earthquake; dynamic forces act on the structure as well as static forces. In order to define these dynamic forces total active pressure coefficient needs to be calculated by using Mononobe – Okabe method [1], which is a generally used form of pseudo-static analysis used to determine the earth pressures on gravity quay walls and is an extension of Coulomb theory for static stress conditions.

In Mononobe – Okabe Method, total active pressure coefficient and the total active thrust is calculated by the following formulas;

$$P_{AE} = \frac{1}{2} K_{AE} \gamma \, H^2 (1 - k_v) \qquad (4.7)$$

$$K_{AE} = \frac{(1 - k_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2(\alpha)\cos(\delta + \alpha + \lambda)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \lambda - \beta)}{\cos(\delta + \alpha + \lambda)\cos(\alpha - \beta)}}\right]^2} \qquad (4.8)$$

$$K_d = K_{AE} - K_A \qquad (4.9)$$

where,

P_{AE}	Total active thrust
K_{AE}	Total active pressure coefficient
K_A	Static active pressure coefficient
K_d	Dynamic active pressure coefficient
γ	Unit weight of soil
H	Wall height

k_v	Vertical seismic coefficient
ϕ	Internal friction angle
α	Angle between back of the retaining wall and the vertical plane
δ	Friction angle between wall and soil
β	Inclined angle with the horizontal
λ	Seismic inertia angle

One of the unknown parameters in total active earth pressure coefficient (K_{AE}) formulation is the seismic inertia angle (λ) of which definition depends on the peak ground surface acceleration (a_{max}) and horizontal seismic coefficient (k_h) [2]. The other unknown parameter is the vertical seismic coefficient (k_v) and the influence of vertical acceleration is ignored in pseudo-static analysis [11].

Spectral acceleration value with respect to soil classification (S_{MS}) is found by the following formula [12];

$$S_{MS} = F_a \times S_s \tag{4.10}$$

where,

S_{MS}	Spectral acceleration value with respect to soil classification
F_a	Short period soil coefficient
S_s	Short period spectral acceleration

Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2008 (TSDC-CRA, 2008) [12] approach assumes that spectral acceleration (S_S) value for T = 0.2 sec can be evaluated by using the coordinate of the region (TSDC-CRA, 2008 appendix A). Similarly, short period soil coefficient (F_a) is obtained from table 1.1 of TSDC-CRA (2008) [12].

Effective seismic coefficient (a_{max} / g) represented for Level-1 (L1) and Level-2 (L2) earthquakes as A_{10} and A_{20} respectively.

Level-1 (L1): L1 earthquake level refers to frequent but low intensity seismic shakes with probability of exceedence 50 % in 50 years. This event has 74 years return period and likely to occur one or more time during the lifetime of the structure.

Level-2 (L2): L2 earthquake level refers to less frequent but intensive major seismic shakes with probability of exceedence 10 % in 50 years. This event has 475 years return period.

Effective seismic coefficients are calculated by using following formulas [12];

$$A_{10} = 0.4 \times S_{MS} \quad \text{for} \quad L1 \tag{4.11}$$

$$A_{20} = 0.4 \times S_{MS} \quad \text{for} \quad L2 \tag{4.12}$$

where,

A_{10} Effective seismic coefficient (a_{max} / g for L1)

A_{20} Effective seismic coefficient (a_{max} / g for L2)

S_{MS} Spectral acceleration value with respect to soil classification g Gravity acceleration

Horizontal seismic coefficient (k_h) is calculated with the followings formulas by taking into consideration the earthquake levels [12];

$k_h = (2/3)\ A_{10}$ for L1 earthquake *(4.13)*

$k_h = A_{20}$ $(A_{20} \leq 0.20)$ for L2 earthquake *(4.14)*

$k_h = (1/3)\ A_{20}^{(1/3)}$ $(A_{20} > 0.20)$ for L2 earthquake *(4.15)*

where,

k_h Horizontal seismic coefficient

A_{10} Effective seismic coefficient (a_{max} / g) for L1

A_{20} Effective seismic coefficient (a_{max} / g) for L2

Then the seismic inertia angle (λ) can be found by using the following formula for dry soil condition [12];

$$\lambda = \arctan(k_h) \quad (4.16)$$

where,

λ Seismic inertia angle

k_h Horizontal seismic coefficient

The seismic inertia force for submerged soil is calculated by the following formula;

$$\lambda = \arctan(k_h^{'}) \qquad (4.17)$$

where,

k_h^{I} Modified horizontal seismic coefficient

Modified horizontal seismic coefficient (k_h^{I}) can be calculated by the following formula;

$$k_h^{I} = \frac{\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^{N}(\gamma_{sj} h_j) + q_o}{\sum_{j=1}^{ND}(\gamma_j h_j) + \sum_{j=ND+1}^{N}(\gamma_{bj} h_j) + q_o} k_h \qquad (4.18)$$

where,

ND Number of the dry soil class according to ground level

γ_j Dry unit weight of soil for soil class j

h_j Thickness of the soil class j

γ_{sj} Saturated unit weight of soil for soil class j

γ_{bj} Submerged unit weight of soil for soil class j

q_o Surcharge load

k_h Horizontal seismic coefficient

4.2.1.2.2 Surcharge

Assuming a uniformly distributed surcharge and adopting the dispersal influence of surcharge like a rectangular shape (see Figure 4.2) the dynamic surcharge forces for each block can be calculated by using the following formulas [2];

$$P_{i(ds)} = q_0 K_d \tag{4.19}$$

$$F_{i(ds)} = P_{i(ds)} h_i \tag{4.20}$$

where,

$P_{i(ds)}$	Dynamic surcharge pressure for block i
q_0	Surcharge load
K_d	Dynamic active pressure coefficient
$F_{i(ds)}$	Dynamic surcharge force for block i
h_i	Height of block i

4.2.1.2.3 Hydrodynamic Forces

The hydrodynamic pressure and hydrodynamic force are calculated by assuming that the pressure area acting on the first block has a triangular shape and the rest have a trapezoidal shape (see Figure 4.2) and by using the following formula [12];

$$p_{dwi} = \frac{7}{8} k_h \gamma_w \sqrt{H_w h_i} \tag{4.21}$$

$$F_{dwi} = \left(\frac{p_{dwi} + p_{dw(i-1)}}{2} \right) h_i \tag{4.22}$$

where,

P_{dwi} Hydrodynamic pressure for block i

k_h Horizontal seismic coefficient

γ_w Unit weight of seawater

H_w Total water depth

h_i Height of block i

F_{dwi} Hydrodynamic force

Considering the water in the backfill, it can be stated that the presence of water can be effective in three ways; changing the inertial forces within the backfill, developing the hydrodynamic pressures within the backfill and allowing excess pore water pressure generation because of the cyclic straining of the backfill soils [11]. Since the effects of excess pore water pressure are poorly defined, mainly it depends on engineering judgment [2].

4.2.1.2.4 Inertia Forces

The inertia forces are calculated by using the following formula;

$$F_h = \frac{a_h W}{g} = k_h W \tag{4.23}$$

where,

F_h Horizontal force

a_h Horizontal pseudo-static acceleration

W Weight of the failure mass

k_h Horizontal seismic coefficient

4.2.2 Vertical Forces

Vertical forces are divided into four parts as; weight of the blocks, weight of the soil, uplift forces and vertical component of total thrust.

4.2.2.1 Weight of the Blocks

Weight of each block (P_i) is calculated by taking into consideration the buoyancy force. Since it is assumed that there is no sea level difference, the uplift force resulting from water table difference is not considered.

4.2.2.2 Weight of the Soil

Weight of the soil mass above each block is taken into consideration as vertical force.

4.2.2.3 Vertical Component of Total Thrust

The vertical component of total static and dynamic earth and surcharge forces are considered as the vertical forces.

4.3 Stability Analysis

In the pseudo-static approach, the stability of gravity quay walls is evaluated with respect to sliding, overturning and bearing capacity. Sliding is often considered as the critical condition for a wall having a large width to height ratio. If width to height ratio is relatively smaller, then overturning or loss of bearing capacity becomes much more critical than sliding. Moreover, since excessive tilting of the wall will lead to collapse under strong shaking conditions, the instability with respect to overturning and bearing capacity is much more serious than for sliding. Therefore, it is a commonly used practice to assign a higher safety factor for overturning and bearing capacity than for sliding [1]. Factor of safety formulas for sliding and overturning are as follows;

$$FS_S = \frac{\mu \sum F_v}{\sum F_h} \tag{4.24}$$

$$FS_o = \frac{\sum M_r}{\sum M_o} \tag{4.25}$$

where,

FS_s Factor of safety against sliding

μ Coefficient of friction between the bottom of the wall body and the foundation

$\sum F_v$ Total vertical force

$\sum F_h$ Total horizontal force

FS_o Factor of safety against overturning

$\sum M_r$ Sum of the resisting moments around the toe of the wall

$\sum M_o$ Sum of the overturning moments around the toe of the wall

At minimum damage performance level, factor of safety values for sliding and overturning shall be grater than or equal to 1.2 and 1.3 respectively in accordance with TSDC-CRA (2008) [12].

Similarly, seismic and surcharge forces are taken into account by reducing its value by 50% ($q_0/2$) in the seismic horizontal load combinations. In addition to that, seismic earth and surcharge force values are increased by 50% in the overturning moment calculations [12].

4.4 Foundation Reactions

Although it is assumed in part 4.1 that the existing soil conditions are improved and are expected not to lead any unsatisfactory results, the bearing capacity of the foundation must be adequate to safely support the structure. The foundation reactions can be calculated by using the following formulas;

$$q_{max}=(\sum F_v/B)(1+6e/B) \quad (4.26)$$

$$q_{min}=(\sum F_v/B)(1-6e/B) \quad (4.27)$$

$$e = (B / 2) - (\sum M_r - \sum M_o) / \sum F_v \quad (4.28)$$

where,

q_{max} Maximum foundation reaction

q_{min} Minimum foundation reaction

$\sum F_v$ Total vertical force

e Eccentricity

B Base length of the wall

$\sum M_r$ Sum of the resisting moments around the toe of the wall

$\sum M_o$ Sum of the overturning moments around the toe of the wall

In the q_{min} formulation, it is obvious that if e = B / 6 then q_{min} will be equal to zero (Figure 4.3 (b)); if e is greater than B / 6 then q_{min} will be less than zero (Figure 4.3 (c)); if e is less than B / 6 then q_{min} will be greater than zero (Figure 4.3 (a)).

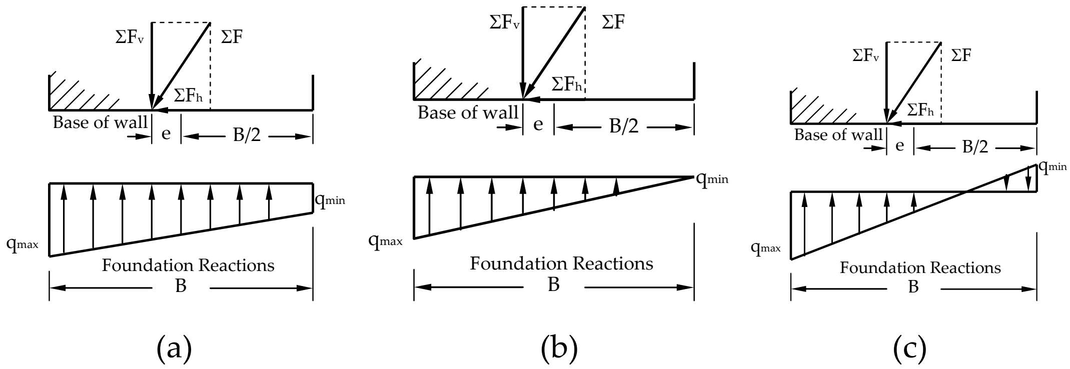


Figure 4.3 Foundation reactions

Since foundation can not carry negative reaction (tension) foundation reactions shall be modified (Figure 4.4) by using following formulas [13];

$$q_{max} = \frac{2}{3}\frac{\sum F_v}{c} \tag{4.29}$$

$$c = \frac{\sum M_r - \sum M_o}{\sum F_v} \tag{4.30}$$

where,

q_{max} Maximum foundation reaction

$\sum F_v$ Total vertical force

c Distance between total vertical force application point and edge of the structure

$\sum M_r$ Sum of the resisting moments around the toe of the wall

$\sum M_o$ Sum of the overturning moments around the toe of the wall

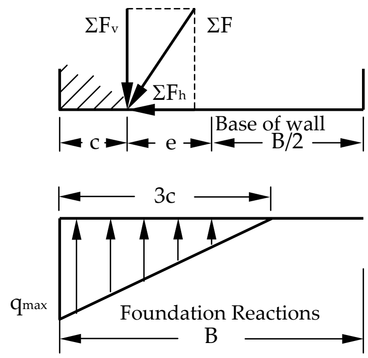


Figure 4.4 Modified foundation reactions

CHAPTER 5

BLOCK TYPE QUAY WALL STABILITY ANALYSIS PROGRAM (QSAP)

5.1 Introduction

The main objective of this study is developing a stability analysis program (QSAP) and secondly, to verify the computational results by using existing data of Derince Port block type quay wall, which was damaged during Marmara earthquake, occurred in 1999. Finally, to make a comparative study between conventional type of quay walls and knapsack type quay walls.

QSAP can solve up to 15 blocked quay wall using Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2008 (TSDC-CRA, 2008) [12].

5.2 Program Assumptions

There are some assumptions defined in QSAP and these are listed as follows;

- QSAP is developed in accordance with Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2008 (TSDC-CRA, 2008) [12].

- In the program the sea side is always located to the left and the land side located to the right (Figure 5.1).
- Only the block types shown in Figure 5.2 can be solved by QSAP.
- Origin of the global coordinate system is the bottom left corner of the block located on the foundation soil (Figure 5.1).
- Program can solve up to 15 blocked quay wall.
- Block-1 always refers to the block located on the top (Figure 5.1).
- There is no passive earth pressure in front of the wall.
- Water level is assumed to be between top and bottom level of Block-2 (Figure 5.1).
- Subsoil under the foundation rubble base and the other soil classes around the quay wall environment are assumed to be improved and the existing soil conditions are expected not to lead any large deformations due to poor soil strength and liquefaction.
- All the earth fills around the wall are highly permeable.
- Sea bed level is assumed to be below top level of the last block (Figure 5.1).
- Program assumes the block coordinates entered in counter-clockwise direction starting from bottom left corner.
- There is no individual load acting directly on the wall other than specified. Such individual loads can be snow loads, train load, vehicle load, cargo handling equipment load etc.
- It is assumed that there is no fluctuation between water levels at sea side and at land side due to wave or tidal action (Figure 5.1)

- All the input data is assumed to be entered in accordance with the units shown on the program input spaces.

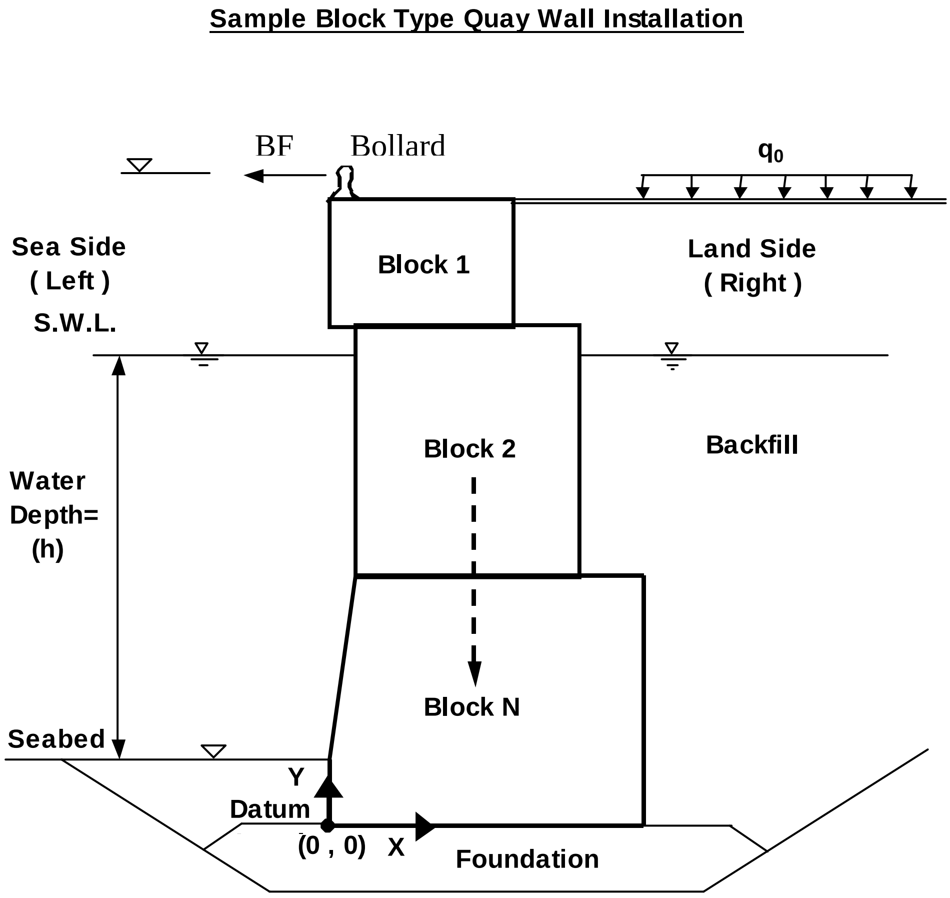


Figure 5.1: Sample block type quay wall installation

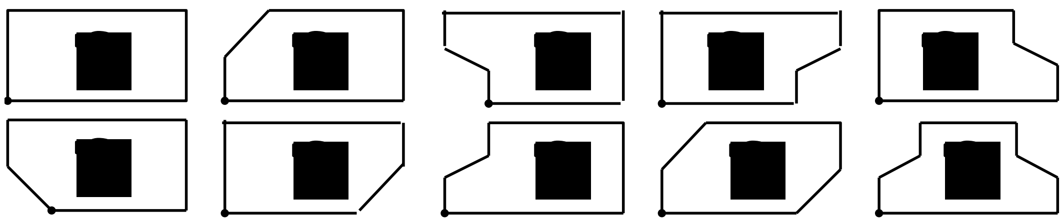


Figure 5.2: Block types that can be solved by QSAP

5.3 Operation of the Program

QSAP is a program developed in spreadsheet environment for the design of block type quay walls. By changing the input data and the number of blocks necessary, it is providing rapid and reliable results.

According to design process of a block type quay wall, first of all, water depth in front of the wall shall be known. Then block dimensions shall be specified by the designer according to probable crane capacities available at site. Depending on the maximum block dimensions and water depth, shape and number of the blocks shall be chosen. Bearing in mind that block type quay wall design is iterative and seeks to provide wall geometry that produces suitable factors of safety for sliding, overturning and bearing capacity.

It should be noted that, QSAP is a computer program and all well-known facts about commercial program are also valid for QSAP. Reliability of the results depends on the quality of the input data. So it is the user's responsibility to verify the reliability of the results produced by QSAP.

Flowchart of the program is presented in Figure 5.3 QSAP basically composed of three major parts that are common to all computer programs namely input, calculation and output phases.

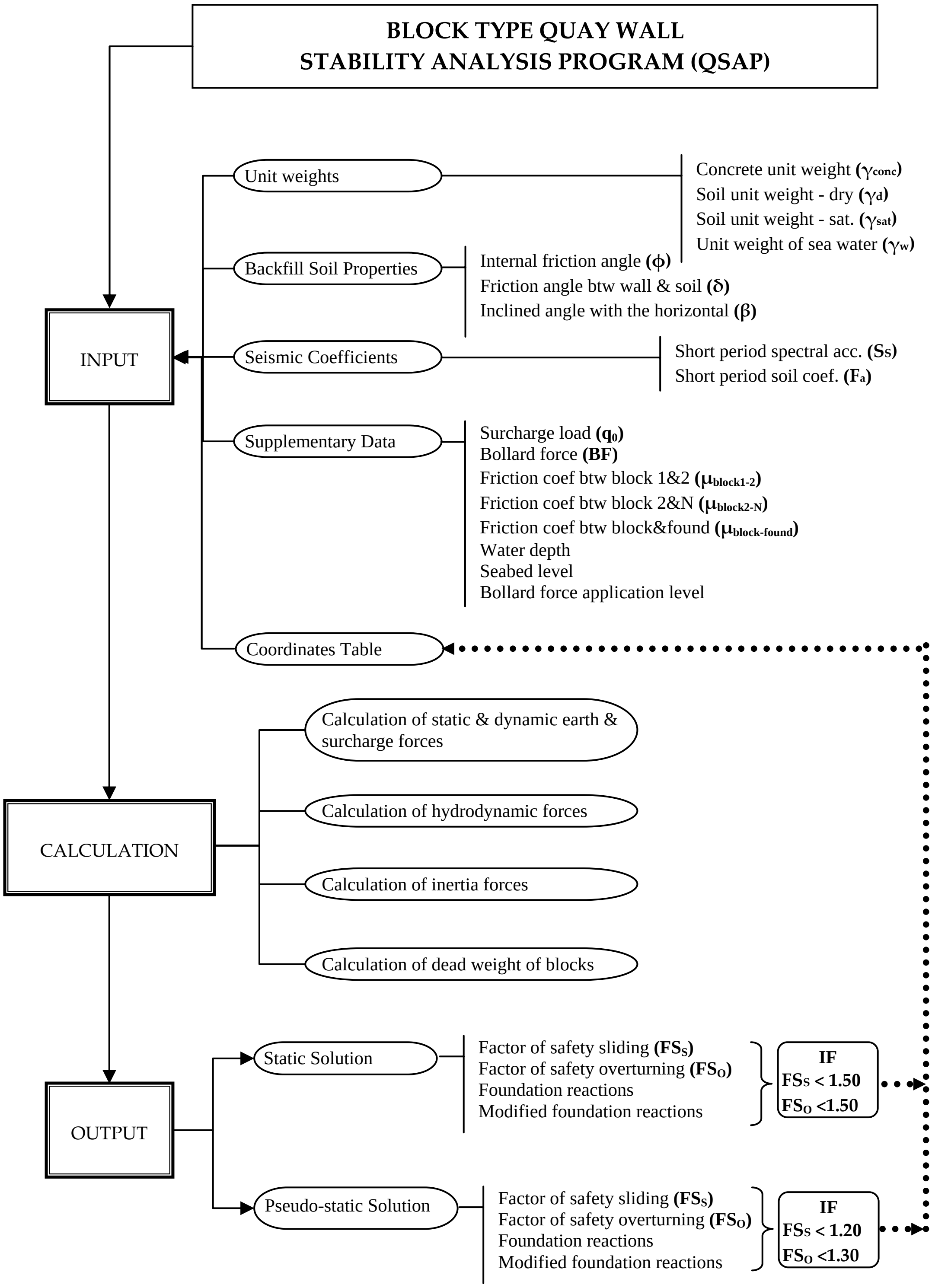


Figure 5.3: Flowchart for QSAP

5.4 Program Input Phase

QSAP needs five major input components in order to be able to start analysis. Four of them are designated with yellow boxes in the start page of the program. Fifth one is named as coordinates table, which is left empty to be filled by the user. Each of these components has vital importance in order to get reliable safety factor and foundation reaction results. In the following paragraphs, these components will be covered in detail to ensure proper input to QSAP.

5.4.1 Unit Weight Data

QSAP requires unit weight information, which is some of the fundamental inputs of the program. Necessary unit weight data is as follows;

- Unit weight of concrete (γ_{conc})
- Dry unit weight of backfill soil (γ_d)
- Saturated unit weight of backfill soil (γ_{sat})
- Unit weight of sea water (γ_w)

Other than these data submerged unit weight of concrete and submerged unit weight of backfill soil are calculated automatically by the program depending on the unit weight of the sea water. The following formulas are used in the calculations;

$$\gamma_{b-conc} = \gamma_{conc} - \gamma_w \quad (5.1)$$

$$\gamma_s = \gamma_{sat} - \gamma_w \quad (5.2)$$

where,

γ_{b-conc} Submerged unit weight of concrete

γ_{conc} Unit weight of concrete

γ_{s} Submerged unit weight of soil

γ_{w} Unit weight of seawater

γ_{sat} Saturated unit weight of soil

The units of the unit weight data shall be entered as kN / m^3.

5.4.2 Backfill Soil Properties Data

Another important data set is the backfill soil properties, which are composed of the parameters listed below;

- Internal friction angle (ϕ) is one of the basic characteristics of soil.
- Friction angle between wall and soil (δ); since lack of measurement is available about δ it can be taken in between $\phi/3 < \delta < 2\phi/3$. In saturated soil conditions δ value is automatically taken as $\delta/2$ by QSAP (TSDC-CRA, 2008) [12].
- Inclination of the ground surface behind the wall (β); in general the area behind the quay wall is utilized as flat storage area. Therefore most of the cases β value is taken as zero.

Since common block type quay wall shape does not have angle between the back of the wall and the vertical plane (α), α is set to default value zero in the program. Parameters in this data set are entered in degrees.

5.4.3 Seismic Coefficients Data

It is one of the major components of QSAP that is utilized in calculation of the pseudo-static forces. Only three coefficients are required to be entered by the user. These are specified as follows;

- Level of earthquake; earthquake level which is to be used in the design of the wall shall be selected with the help of drop-down menu provided in the program. Two different earthquake levels L1 and L2 can be solved by QSAP. L1 earthquake level refers to frequent but low intensity seismic shakes with probability of exceedance 50% in 50 years. Conversely, L2 earthquake level refers to less frequent but intensive seismic shakes with probability of exceedance 10% in 50 years [1].
- Short period (T=0.2 sec) spectral acceleration (S_S); it is related with the design earthquake level, the coordinates and the soil characteristics of the region. S_S values are listed in TSDC-CRA (2008) [12] Appendix A based on soil class B.
- Short period soil coefficient (F_a); if soil class is different than class B, F_a will be found out from table 1.1 of TSDC-CRA (2008) [12]. Further information about soil classifications can be found in Appendix B of TSDC-CRA (2008) [12].

Other seismic coefficients are derived automatically from two coefficients mentioned above. The detailed explanations about static and seismic coefficients are given in Chapter 4 Methodology.

5.4.4 Supplementary Data

Eight numbers of different data are required to be entered as supplementary data. Five of them are grouped and entered like tabular form. The others are entered on the "sample block type quay wall installation" sketch on the start page of the program.

Tabular form entries are listed as follows;

- Surcharge load (q_o); surcharge load depends on the amount of storage necessities just behind the wall, above backfill area. The surcharge load is entered in kN/m^2.
- Bollard force (BF); it is a mooring force generated by the ships. It depends on the tonnage of the ship and the length between successive contraction joints of Block-1. Bollard force can be found simply by dividing of mooring force to length of Block-1.
- Friction coefficient between block-1 and block-2 ($\mu_{block1\text{-}2}$); since block-1 is instu poured concrete block, friction coefficient can be taken a higher value as 0.75.
- Friction of coefficient between block-2 and block-N ($\mu_{block1\text{-}N}$); it is advised to take this value as 0.5 in table 2.4 of TSDC-CRA (2008) [12].
- Friction of coefficient between block and foundation ($\mu_{conc\text{-}found}$); it is advised to take this value as 0.6. For the other values table 2.4 of TSDC-CRA (2008) [12] can be used.

On-sketch entries are listed as follows;

- Sea bed level shall be entered in meters according to the global coordinate system shown on the sketch.
- Water depth is the distance between still water level (SWL) and sea bed level. It is entered in meters.
- Bollard force application level is generally taken as 0.5 m above the block on top. It is entered in meters.

5.4.5 Coordinates Table

It is the major input component of QSAP that requires to be filled with block coordinates. After the decision of maximum block dimensions and number of blocks used, the designer will enter raw block coordinates which are to be modified during design stage in order to have suitable factors of safety for sliding, overturning and bearing capacity.

Useful information about coordinates table entry are given below;

- Block coordinates are to be entered according to the global coordinate system shown on sample sketch at the start page of the program.
- Block coordinates are to be entered in counter-clockwise direction starting from bottom left corner of the block as demonstrated at the start page of the program.
- It is advisable to draw a small sketch to identify the global coordinates of the block corners before filling out the coordinates table.

- Model View, which is at the start page of the program, draws the structure with the given coordinates simultaneously. It can be utilized in order not to enter incorrect coordinates data.
- Only the necessary cells shall be filled. It must be assured that un-relevant cells shall be left empty.
- Data entry to coordinates table starts from top block (Block-1) to bottom block (Block-N).
- Only the block types demonstrated at the start page of the program can be solved by the program.
- It is not necessary to identify the shape of the block chosen anywhere in the program. QSAP detects the shape automatically as per the data entered in the coordinates table.
- Block-1 refers to the block located on top.
- Up to 15 block coordinates can be entered to the table.
- All coordinates are to be entered in meters.

5.5 Program Calculation Phase

5.5.1 Calculation of Static & Dynamic Earth and Surcharge Forces

Calculation methodology of static and dynamic earth and surcharge forces is explained in detail in Chapter 4 Methodology. In the program these forces are calculated individually for all blocks together with horizontal and vertical components. Calculations are based on triangular pressure distribution for earth forces and rectangular pressure distribution for surcharge loads.

5.5.2 Calculation of Hydrodynamic Forces

It is an important dynamic force, which exerts suction to seaward side of the wall. It is assumed that hydrodynamic suction pressure is only generated at the wall levels between still water level and sea bed level. In order to enable the calculations of hydrodynamic forces the pressure area acting on first block is assumed as triangular and the following areas are assumed as trapezoidal. Detailed explanations are given in Chapter 4 Methodology.

5.5.3 Calculation of Inertia Forces

According to TSDC-CRA (2008) [12], water buoyancy force is not taken into account in the inertia force calculation of blocks and soil body standing on the wall. In other words, inertia forces shall be found using dry unit weights of blocks and the soil. Inertia forces for concrete blocks are calculated by multiplying weight of the blocks and horizontal seismic coefficient (k_h). In similar inertia force for soil body standing on the wall is calculated by multiplying weight of the soil body with horizontal seismic coefficient (k_h) above still water level (SWL) and modified horizontal seismic coefficient (k_h') below SWL. QSAP calculates moment arms according to the block shape automatically.

5.5.4 Calculation of Dead Weight of the Blocks

Buoyancy is taken into account in calculation of the dead weights of the blocks. Dead weights are calculated by multiplying unit weight with area of the block. Area of each block (A) is calculated by using simple matrix.

QSAP again calculates moment arms according to the block shape automatically.

$$A = \begin{bmatrix} X_1 & Y_1 \\ X_2 & Y_2 \\ X_3 & Y_3 \\ X_4 & Y_4 \\ . & . \\ . & . \\ . & . \\ X_N & Y_N \\ X_1 & Y_1 \end{bmatrix} \quad (5.3)$$

Block having four corners;

$$A = [(X_1Y_2 + X_2Y_3 + X_3Y_4 + X_4Y_1) - (X_2Y_1 + X_3Y_2 + X_4Y_3 + X_1Y_4)] \times \frac{1}{2} \quad (5.4)$$

Block having five corners;

$$A = [(X_1Y_2 + X_2Y_3 + X_3Y_4 + X_4Y_5 + X_5Y_1) - (X_2Y_1 + X_3Y_2 + X_4Y_3 + X_5Y_4 + X_1Y_5)] \times \frac{1}{2} \quad (5.5)$$

Block having six corners;

$$A = \big[(X_1Y_2 + X_2Y_3 + X_3Y_4 + X_4Y_5 + X_5Y_6 + X_6Y_1) - (X_2Y_1 + X_3Y_2 + X_4Y_3 + X_5Y_4 + X_6Y_5 + X_1Y_6)\big] \times \frac{1}{2} \quad (5.6)$$

Block having eight corners;

$$A = \big[(X_1Y_2 + X_2Y_3 + X_3Y_4 + X_4Y_5 + X_5Y_6 + X_6Y_7 + X_7Y_8 + X_8Y_1) - (X_2Y_1 + X_3Y_2 + X_4Y_3 + X_5Y_4 + X_6Y_5 + X_7Y_6 + X_8Y_7 + X_1Y_8)\big] \times \frac{1}{2} \quad (5.7)$$

5.6 Program Output Phase

QSAP gives two major output components. These are static stability analyses and pseudo-static stability analyses. In the following paragraphs these components will be covered in details.

5.6.1 Static Solution

Four different analyses results are presented in the program and these are discussed below;

- Factor of safety against sliding (FS_s); it is found by multiplying friction coefficient with division of total vertical force to total horizontal force. QSAP automatically chooses the related friction coefficient depending on the block location. Minimum safety factor is taken as 1.5 in static condition.
- Factor of safety against overturning (FS_o); it is found simply by dividing the total resisting moment to total overturning moment. Minimum safety factor is taken as 1.5 in static condition.
- Foundation reactions; QSAP also gives information about foundation reactions in order to compare with bearing capacity of foundation. To give extra information about reactions between the blocks, foundation reactions are calculated for all blocks regardless of location in the wall geometry. It should be noted that exact block base lengths are used in the calculation of reactions. Therefore, the reactions between the blocks, whose bases are not fully contacting to the top surface of the next block, should be considered carefully. Necessary equations and explanations are provided in Chapter 4 Methodology.
- Modified foundation reactions; since foundation can not carry negative reaction (tension), foundation reaction shall be modified by adding negative reaction amount to compression side according to the rules explained in Chapter 4 Methodology.

5.6.2 Pseudo-static Solution

Four different analyses results are also presented for the pseudo-static solution in the program. These are listed below;

- Factor of safety against sliding (FS_s); static and seismic surcharge forces are taken into account by reducing its value by 50% ($q_0/2$) in the seismic horizontal load combinations. The static bollard force is taken into account by reducing its value by 50% for seismic design [1]. Except the minimum safety factor value, which is taken as 1.2 for minimum damage performance level, explanations in static solution part are valid for the pseudo-static solution part.
- Factor of safety against overturning (FS_o); except minimum safety factor, which is taken as 1.3 for minimum damage performance level, explanations in static solution part are valid for the pseudo-static solution part. Moreover, static and seismic surcharge forces are taken into account by reducing its value by 50% ($q_0/2$) in the seismic horizontal load combinations. The static bollard force is taken into account by reducing its value by 50% for seismic design [1]. In addition to that seismic earth and surcharge force values are increased by 50% in the overturning moment calculation.
- Foundation reaction; explanations in the static solution part are valid for the pseudo-static solution.
- Modified foundation reactions; explanations in the static solution part are valid for the pseudo-static solution part.

CHAPTER 6

PROGRAM VERIFICATION: CASE STUDY ON DERINCE PORT BLOCK TYPE QUAY WALL

6.1 Introduction

Derince port block type quay wall is one the structures that seriously damaged during the Marmara earthquake, 1999, which caused over 15,000 fatalities and 20 billion USD losses. Observations revealed that the block type quay wall moved seaward without any vertical displacement, where 0.5 m to 0.7 m lateral displacement occurred in the structure. At some quays mid-span deflections and relative corner movements were observed. The peak ground accelerations were obtained approximately 0.25g to 0.30 g [1].

The stability analysis of Derince Port block type quay wall (see Figure 6.1) damaged during Marmara earthquake (1999) is carried out by using TSDC-CRA, 2008 [12] with developed block type quay wall stability analysis program QSAP.

6.2 Stability Analysis Performed with QSAP

Since some of the block types of Derince Port can not be solved by QSAP, some assumptions shall be made to overcome this incompatibility. The assumptions are as follows;

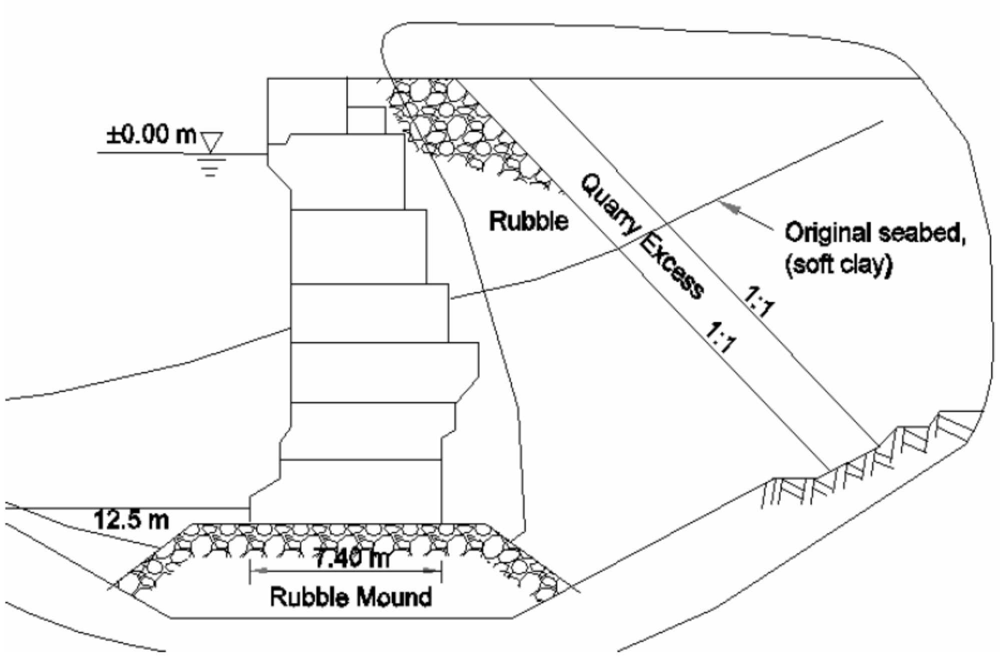


Figure 6.1: Block type quay wall in Derince Port

- The blocks are named different than manual solution of Derince Port quay wall presented in Appendix A. Block names shown in manual solution crown, block-1, block-2, block-3, block-4, block-5 and block-6 are represented with block-1, block-2, block-3, block-4, block-5, block-6 and block-7 respectively in QSAP solution.
- Actual block-1 shape is represented with rectangular shaped block.

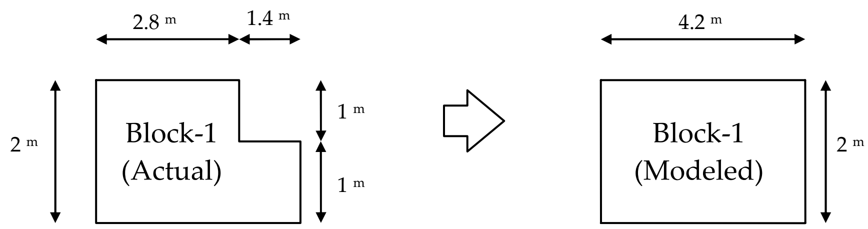


Figure 6.2: Assumed shape of block-1

▪ Actual block-6 shape is represented with a similar block shape, which can be solved by QSAP.

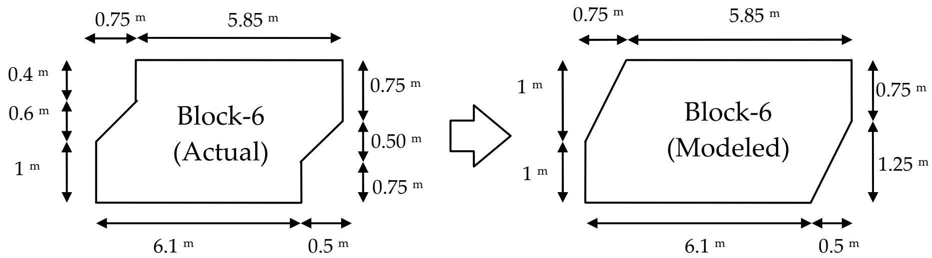


Figure 6.3: Assumed shape of block-6

QSAP pseudo-static stability analyses are presented in the following pages (Figures 6.4 to 6.11). All the input data and its derivations can be found in the start page of the program.

BLOCK TYPE QUAY WALL STABILITY ANALYSIS PROGRAM (QSAP)

Unit Weight Properties:

γ_{conc} =	23.00	kN / m³
$\gamma_{b\text{-}conc}$ =	13.00	kN / m³
γ_d =	18.00	kN / m³
γ_{sat} =	21.00	kN / m³
γ_s =	11.00	kN / m³
γ_w =	10.00	kN / m³

Backfill Properties:

φ =	40.00	°
δ =	13.33	°
β =	0	°
α =	0	°

Static & Seismic Coefficients:

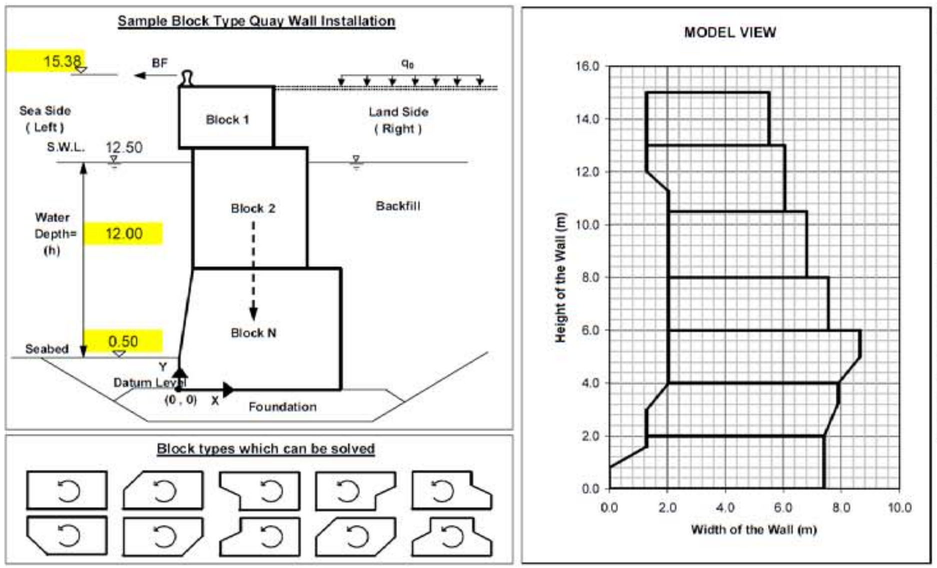
Earthquake Level =	L1	
S_s=	0.76	g
F_a=	0.80	
S_{MS}=	0.61	g
A_{10} or A_{20}=	0.243	g
k_h=	0.16	g
λ_{dry}=	9.21	°
k_h=	0.26	g
λ_{sat}=	14.57	°
Dry:		
K_{as}=	0.202	
K_{at}=	0.291	
K_{ad}=	0.089	
Saturated:		
K_{as}=	0.202	
K_{at}=	0.363	
K_{ad}=	0.161	

Supplementary Data:

q_0 =	30.00	kN / m²
BF =	49.50	kN / m
$\mu_{block1\text{-}2}$	0.50	
$\mu_{block2\text{-}N}$	0.50	
$\mu_{block\text{-}found}$	0.60	

Coordinates of Blocks

	X_1	Y_1	X_2	Y_2	X_3	Y_3	X_4	Y_4	X_5	Y_5	X_6	Y_6	X_7	Y_7	X_8	Y_8
Block 1	1.30	13.00	5.50	13.00	5.50	15.00	1.30	15.00								
Block 2	2.05	10.50	6.05	10.50	6.05	13.00	1.30	13.00	1.30	12.00	2.05	11.25				
Block 3	2.05	8.00	6.80	8.00	6.80	10.50	2.05	10.50								
Block 4	2.05	6.00	7.55	6.00	7.55	8.00	2.05	8.00								
Block 5	2.05	4.00	7.90	4.00	8.65	5.00	8.65	6.00	2.05	6.00						
Block 6	1.30	2.00	7.40	2.00	7.90	3.25	7.90	4.00	2.05	4.00	1.30	3.00				
Block 7	0.00	0.00	7.40	0.00	7.40	2.00	1.30	2.00	1.30	1.60	0.00	0.80				
Block 8																
Block 9																
Block 10																
Block 11																
Block 12																
Block 13																
Block 14																
Block 15																



Calculation Summary

Block No.	Pseudostatic Solution FS_s	FS_o
1	1.07	3.28
2	0.89	1.29
3	0.79	1.01
4	0.77	0.95
5	0.80	1.00
6	0.73	0.97
7	0.82	1.04
8		
9		
10		
11		
12		
13		
14		
15		

Figure 6.4 Start page of QSAP

Block 1 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
P_1	193.20	2.10	405.72
F_{s1v}	2.79	4.20	11.74
F_{e1v}	1.68	4.20	7.04

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
BF	49.50	2.38	117.56
F_{s1h}	11.79	1.00	11.79
F_{e1h}	7.08	0.67	4.72

Pseudostatic Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
P_{1d}	31.32	1.00	31.32
F_{ds1h}	5.20	1.00	5.20
F_{de1h}	3.12	0.67	2.08
T_{filld}	18.39	1.00	18.39
F_{ds1v}	1.23	4.20	5.17
F_{de1v}	0.74	4.20	3.10

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
199.64	93.15	432.77	132.02

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 1.07 **<1.20 NOT OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 3.28 **>1.30 OK**

Figure 6.5: QSAP stability calculation for block-1

Block 2 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1	197.67	-0.75	-148.25
P_2	166.22	1.78	296.06
F_{s2v}	2.11	4.00	8.42
F_{e2v}	0.84	4.00	3.35
F_{e3v}	0.10	4.00	0.42
F_{e4v}	2.11	4.00	8.44
F_{e5v}	0.52	4.00	2.06
T_{fill}	19.80	3.73	73.76

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1	68.37	2.50	170.92
F_{s2h}	14.99	1.25	18.73
F_{e2h}	3.54	2.25	7.96
F_{e3h}	0.44	2.17	0.96
F_{e4h}	18.06	1.00	18.06
F_{e5h}	4.41	0.67	2.94

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1	93.15	2.50	232.87
P_{2d}	41.14	1.30	53.52
F_{ds2h}	10.89	1.15	12.52
F_{de2h}	1.56	2.25	3.51
F_{de3h}	0.19	2.17	0.42
F_{de4h}	7.96	1.00	7.96
F_{de5h}	3.52	0.67	2.35
T_{filld}	28.13	1.17	32.87
F_{dw}	6.95	0.67	4.63
F_{ds2v}	1.43	4.00	5.72
F_{de2v}	0.37	4.00	1.48
F_{de3v}	0.05	4.00	0.18
F_{de4v}	0.93	4.00	3.72
F_{de5v}	0.41	4.00	1.64

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
394.52	221.98	689.77	535.30

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 0.89 **<1.20 NOT OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.29 **<1.30 NOT OK**

Figure 6.6: QSAP stability calculation for block-2

Block 3 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	389.36	0.00	0.00
P_3	154.38	2.38	366.64
F_{s3v}	1.76	4.75	8.35
F_{e6v}	3.93	4.75	18.65
F_{e7v}	0.81	4.75	3.83
T_{fill}	50.25	4.38	219.84

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	109.81	2.50	274.52
F_{s3h}	15.05	1.25	18.81
F_{e6h}	33.61	1.25	42.01
F_{e7h}	6.90	0.83	5.75

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	221.98	2.50	554.96
P_{3d}	44.28	1.25	55.35
F_{ds3h}	11.99	1.25	14.99
F_{de6h}	18.87	1.25	23.58
F_{de7h}	5.53	0.83	4.61
T_{filld}	21.65	1.25	27.06
F_{dw}	21.72	1.17	25.34
F_{ds3v}	1.40	4.75	6.66
F_{de6v}	3.13	4.75	14.87
F_{de7v}	0.64	4.75	3.05

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
610.81	388.06	1331.67	1318.11

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 0.79 **<1.20 NOT OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.01 **<1.30 NOT OK**

Figure 6.7: QSAP stability calculation for block-3

Block 4 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	600.48	0.00	0.00
P_4	143.00	2.75	393.25
F_{s4v}	1.41	5.50	7.74
F_{e8v}	4.43	5.50	24.37
F_{e9v}	0.52	5.50	2.84
T_{fill}	70.88	5.13	363.23

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	165.36	2.00	330.72
F_{s4h}	12.04	1.00	12.04
F_{e8h}	37.92	1.00	37.92
F_{e9h}	4.41	0.67	2.94

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	388.06	2.00	776.11
P_{4d}	41.02	1.00	41.02
F_{ds4h}	9.59	1.00	9.59
F_{de8h}	23.95	1.00	23.95
F_{de9h}	3.54	0.67	2.36
T_{filld}	10.30	1.00	10.30
F_{dw}	22.95	0.97	22.25
F_{ds4v}	1.12	5.50	6.17
F_{de8v}	3.53	5.50	19.42
F_{de9v}	0.41	5.50	2.26

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
836.10	542.97	2150.95	2267.35

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 0.77 <1.20 NOT OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 0.95 <1.30 NOT OK**

Figure 6.8: QSAP stability calculation for block-4

Block 5 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	820.71	0.00	0.00
P_5	166.73	3.21	535.32
F_{s5v}	1.41	6.60	9.28
F_{e10v}	5.46	6.60	36.05
F_{e11v}	0.52	6.60	3.40
T_{fill}	128.15	6.05	775.31

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	219.73	2.00	439.46
F_{s5h}	12.04	1.00	12.04
F_{e10h}	46.75	1.00	46.75
F_{e11h}	4.41	0.67	2.94

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	542.97	2.00	1085.93
P_{5d}	47.83	1.02	48.76
F_{ds5h}	9.59	1.00	9.59
F_{de10h}	31.03	1.00	31.03
F_{de11h}	3.54	0.67	2.36
T_{filld}	0.00	0.00	0.00
F_{dw}	26.86	0.98	26.26
F_{ds5v}	1.12	6.60	7.40
F_{de10v}	4.35	6.60	28.74
F_{de11v}	0.41	6.60	2.71

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1144.25	714.20	3549.17	3547.32

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 0.80 <1.20 NOT OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.00 <1.30 NOT OK**

Figure 6.9: QSAP stability calculation for block-5

Block 6 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	1122.97	0.75	842.23
P_6	162.66	3.31	538.93
F_{s6v}	1.41	6.60	9.28
F_{e12v}	6.49	6.60	42.86
F_{e13v}	0.52	6.60	3.40
T_{fill}	0.00	6.60	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	282.93	2.00	565.86
F_{s6h}	12.04	1.00	12.04
F_{e12h}	55.58	1.00	55.58
F_{e13h}	4.41	0.67	2.94

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	714.20	2.00	1428.41
P_{6d}	46.66	0.99	46.41
F_{ds6h}	9.59	1.00	9.59
F_{de12h}	38.12	1.00	38.12
F_{de13h}	3.54	0.67	2.36
T_{filld}	0.00	0.00	0.00
F_{dw}	30.25	0.98	29.72
F_{ds6v}	1.12	6.60	7.40
F_{de12v}	5.18	6.60	34.16
F_{de13v}	0.41	6.60	2.71

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1322.04	903.58	5030.15	5190.32

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 0.73 <1.20 NOT OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 0.97 <1.30 NOT OK**

Figure 6.10: QSAP stability calculation for block-6

Block 7 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	1294.05	1.30	1682.26
P_7	178.88	3.94	704.56
F_{s7v}	1.41	7.40	10.41
F_{e14v}	7.53	7.40	55.69
F_{e15v}	0.52	7.40	3.82
T_{fill}	0.00	7.40	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	354.96	2.00	709.92
F_{s7h}	12.04	1.00	12.04
F_{e14h}	64.40	1.00	64.40
F_{e15h}	4.41	0.67	2.94

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	903.58	2.00	1807.16
P_{7d}	51.31	0.96	49.11
F_{ds7h}	9.59	1.00	9.59
F_{de14h}	45.20	1.00	45.20
F_{de15h}	3.54	0.67	2.36
T_{filld}	0.00	0.00	0.00
F_{dw}	24.71	1.24	30.68
F_{ds7v}	1.12	7.40	8.30
F_{de14v}	6.00	7.40	44.39
F_{de15v}	0.41	7.40	3.04

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1517.89	1107.98	7542.61	7235.21

$FS_{sliding}$ ($\mu * \Sigma F_v / \Sigma F_h$)= **0.82** **<1.20 NOT OK** **$FS_{overturning}$ ($\Sigma M_r / \Sigma M_o$)=** **1.04** **<1.30 NOT OK**

Figure 6.11: QSAP stability calculation for block-7

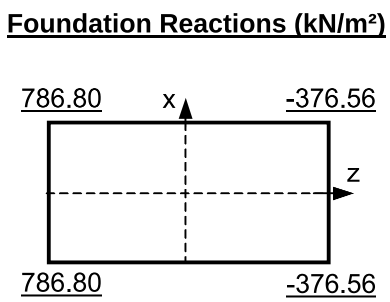


Figure 6.12: Foundation reactions

6.3 Program Verification

QSAP has been verified by using it for the stability calculation of Derince Port quay wall, which is manually solved by Hülya Karakuş, 2007 [2]. Factor of safety values are tabulated below for both QSAP solution and manual solution, which is presented in Appendix A.

As it seen from Table 6.1 almost all factor of safety values are in the same order, except block-1 values. Considering the assumption made for Block-1, representing the soil part of the actual shape with concrete unavoidably increased the factor of safety values. On the other hand the safety factor differences are diminished at the next block since the over-predicted force, resulted from the assumption, is very small comparing to the forces acting on the rest of the wall. In addition to that slight difference is observed at block-7 factor of safety values due to the assumption made for block-6. Lastly, the orders of magnitudes of the forces acting on the wall are the same in both solutions.

Table 6.1: Factor of safety values both manual solution and QSAP solution for comparison

Block Number	Manual Solution		QSAP Solution	
	FS_S	FS_o	FS_S	FS_o
Block-1	1.05	3.15	1.07	3.28
Block-2	0.88	1.26	0.89	1.29
Block-3	0.79	1.00	0.79	1.01
Block-4	0.77	0.94	0.77	0.95
Block-5	0.80	1.00	0.80	1.00
Block-6	0.73	0.98	0.73	0.97
Block-7	0.81	1.05	0.82	1.04

It is obvious from the results obtained from both solutions that the factor of safety values for sliding are less than the required limits indicating a failure of structure, which is compatible with the actual field measurement where block type quay wall moved seaward without any vertical displacement and overturning failure. This result brings out the importance of factor of safety criteria, which is important for an economical solution. Since factor of safety overturning and sliding values are obtained in the range of 3.28 - 0.94 and in the range of 0.73 – 1.07 respectively for both manual and QSAP solutions, it can be deduced that in the block type quay wall design factor of safety values both overturning and sliding can be taken in the range of 1.00 – 1.05. However, this discussion must be verified by further studies based on real cases.

As a conclusion, QSAP solution is found to be consistent with the manual solution based on TSDC-CRA, 2008 [12]

CHAPTER 7

COMPARISON STUDY BETWEEN CONVENTIONAL PLACING AND KNAPSACK PLACING OF QUAY WALL BLOCKS

7.1 Introduction

Gravity type walls are good choice for quay wall construction in regions with adequate foundation bearing capacity. Concrete block quay wall, which is the most common and simplest gravity type quay wall, is composed of concrete blocks with various shapes sitting on each other to form the wall section. Dimensions and weight of the blocks depend on geotechnical conditions, design calculations and the contractor's equipment. Mainly, gravity type quay wall resistance to various driving forces is provided by wall weight [5].

Concrete block types of gravity quay walls have been used world wide with various wall heights. Derince Port (Turkey), San Antonio Port (Chile) Kalamata Port (Greece), Porto Arabia (Qatar), Sohar Port (Oman), Pars Petrochemical Port (Iran) are some of the examples of this kind [5].

Placing of blocks can be done in two ways named conventional (vertical front face) and knapsack formation. In conventional formation individual blocks are placed on top of each other to form a vertical front face and stepped-like shape at the back face. Knapsack formation especially in seismic regions, where the water depth is grater than 7 m, is another alternative in order to push the center of gravity of the wall to landside and increase the stability of the wall [7].

The researchers mentioned in the literature survey chapter, has been investigated the various advantages and disadvantages of the knapsack shaped block type quay walls. In this study, factor of safety values of conventional placed and knapsack placed block type quay walls, which are solved in accordance with TSDC-CRA, 2008 [12] codes, will be compared by using QSAP.

7.2 Assumptions and Methodology

In order to compare the differences between two block placing methods in the stability analysis calculations, slightly bigger wall with 12 blocks are used.

Assumptions are listed as follows;

- Antalya Harbor is chosen as the project site for the comparison study. Short period spectral acceleration (Ss) belonging to Antalya Harbor site coordinates is determined in the condition of L1 earthquake level and "A class" soil characteristics.
- In the stability analyses limit values of factor of safety sliding and overturning are taken as 1.20 and 1.30 respectively in accordance with TSDC-CRA,2008 [12] in the condition of minimum damage performance level.

- It is assumed that soil in the foundation and the soil around the wall are improved in order not to give unsatisfactory bearing and liquefaction problems.
- Starting dimensions and shape of the knapsack formed block type quay wall is taken from study of Jalili et al. [7].
- Block heights are taken constant as 1.50 m.
- All the input data is fixed for either types of quay wall formation and can be reached from the start page of the program.

Methodology is described below;

- Analysis is initiated with conventional placed block type quay wall with temporary dimensions.
- Conventional formed wall is designed using limit factor of safety values after iteration process performed by QSAP.
- Then the section area of conventional formed wall is measured.
- A knapsack formed block type quay wall having the same section area with conventional one is designed regarding limit factor of safety values after iteration process performed by again QSAP.

For the sections of conventional and kapsack formed block type quay walls designed for comparison are demonstrated in Figure 7.1 and Figure 7.2 respectively.

QSAP input, calculation and output data for the stabilitiy calculation of the walls are given in Appendix B.

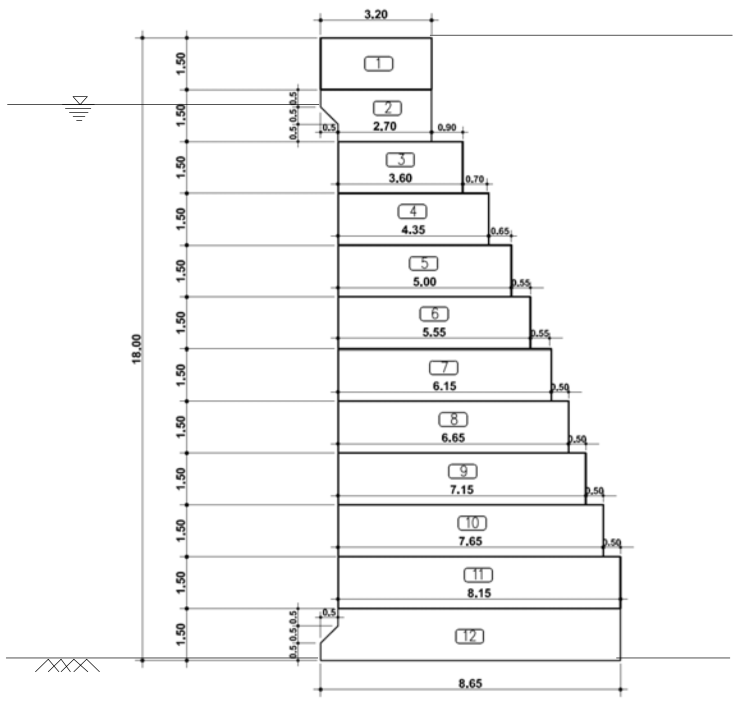


Figure 7.1: Conventional (vertical front face) placed block type quay wall

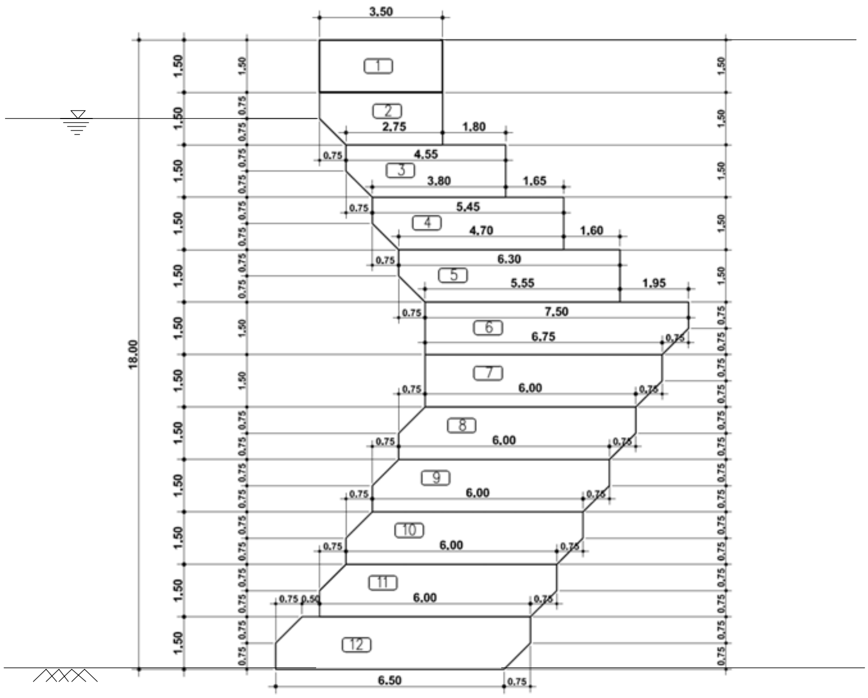


Figure 7.2: Knapsack placed block type quay wall

7.3 Discussions on Stability Analyses Results

Factor of safety values for both placing methods are summarized in Table 7.1.

Table 7.1: Summary of factor of safety values

Block Number	Conventional Placing Method		Knapsack Placing Method	
	FS_s	FS_o	FS_s	FS_o
Block-1	1.73	2.69	1.76	3.09
Block-2	1.21	1.45	1.23	1.37
Block-3	1.21	1.57	1.40	1.36
Block-4	1.21	1.60	1.53	1.35
Block-5	1.21	1.58	1.65	1.36
Block-6	1.20	1.52	1.82	2.21
Block-7	1.21	1.51	1.66	1.82
Block-8	1.20	1.46	1.52	1.91
Block-9	1.20	1.42	1.40	1.92
Block-10	1.20	1.40	1.30	1.89
Block-11	1.20	1.38	1.21	1.84
Block-12	1.38	1.38	1.38	1.93

For the conventional placed block type quay wall, factor of safety for sliding, which is the governing case has to be satisfied. In the design, block dimensions are chosen in order to have 1.20 or greater factor of safety sliding values. According to the geometric necessities only block-12 is having slightly higher safety values for sliding. Factor safety overturning values are higher than 1.30 for all blocks and decreasing gradually towards the bottom block (see Table 7.1). Block-1 has slightly higher factor of safety sliding and overturning values due to higher friction coefficient (0.75) and the absence of water buoyancy effect respectively.

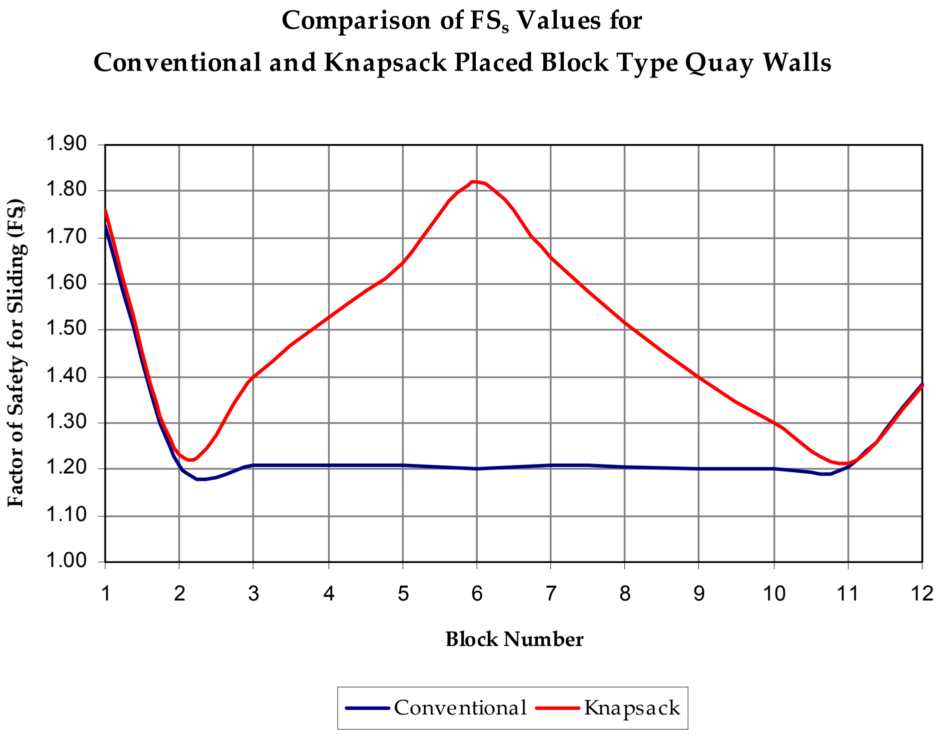


Figure 7.3: Comparison of FS_s values

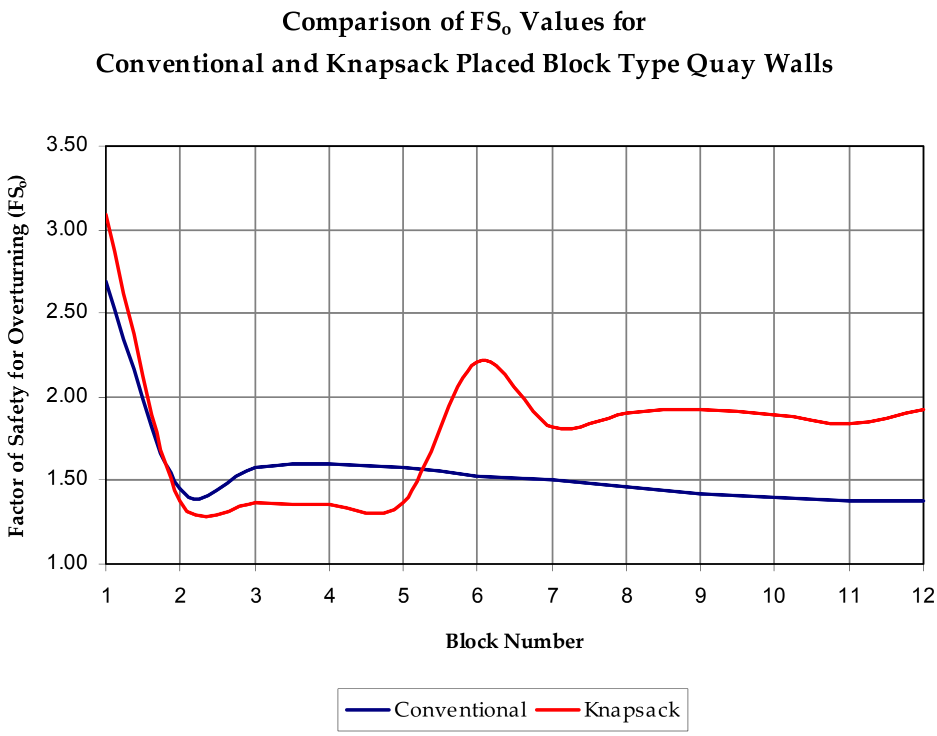


Figure 7.4: Comparison of FS_o values

For the knapsack placed block type quay wall, both sliding and overturning factor of safety values are governing in different parts of the wall. Mainly factor of safety overturning values are more critical on the upper blocks, which are block-3, 4 and block-5 (see Table 7.1). Factor of safety sliding values are more critical on the lower blocks, which are block-6, 7, 8, 9, 10, 11 and block-12 (see Table 7.1). Block-1 has slightly higher factor of safety sliding and overturning values due to higher friction coefficient (0.75) and the absence of water buoyancy effect respectively. Both safety values in sliding and overturning are higher than 1.20 and 1.30 respectively for knapsack formed wall.

With the same cross-sectional area properties, in general knapsack formed wall has greater factor of safety sliding values for all blocks comparing to conventional formed one (see Figure 7.3). On the other hand except block-1, conventional formed wall has greater overturning safety values up to block-6, which is at the turning point for the knapsack back slope, comparing to knapsack formed wall. For the rest of the blocks below block-6, knapsack formed wall has greater overturning safety values comparing to the conventional formed wall due to its increasing nature of moment arms (see Figure 7.4). Factor of safety sliding values of last block (block-12) are slightly higher for both formations since the friction coefficient is taken as 0.60 between concrete and foundation.

To sum up, stability analysis results are evaluated for two types of placing methods solved in accordance with TSDC-CRA, 2008 [12]. As it is seen from the results of example problem, comparing two placement methodologies (conventional and knapsack) from the stability point of view, knapsack placement methodology might be found more economic. However, considering the difficulties in construction of knapsack section, it is necessary to check the concrete volumes of the alternatives for economic and feasibility evaluation.

CHAPTER 8

CONCLUSION AND RECOMMENDATIONS

In this study, a computer program named QSAP (block type Quay wall Stability Analysis Program) has been developed using pseudo-static methodolgy, verified with Derince Port manual solution and implemented for the comparison of knapsack and conventional placed block type quay walls. QSAP is developed as a reliable and user-friendly stability analysis program for block type quay walls, which shortens the design period of block type quay walls.

In the development of QSAP, following assumptions are used for the stability analysis of block type quay wall;

- Existing soil conditions are improved in order not to have unsatisfactory performance, such as liquefaction.
- Peak ground acceleration (a_{max}) and seismic coefficient (k_h) are calculated in accordance with Turkish Seismic Design Codes for Coastal Structures, Railways and Airport Structures, 2008 (TSDC-CRA, 2008) [12].
- Seismic coefficient (k_h) is modified ($k_h^{\prime}$) for the backfill material below still water level (SWL).
- Seismic and static surcharge forces are taken into account by reducing its value by 50% ($q_0/2$) in the seismic horizontal load

combinations. Surcharge forces are applied in rectangular distribution.

- Seismic earth and surcharge force values are increased by 50% in the overturning moment calculations.
- Mononobe-Okabe Method is used to introduce the dynamic behavior of active earth thrust. Dynamic earth pressure is applied in triangular distribution.
- Hydrodynamic forces, which lead suction pressure, are taken into consideration according to Wetergaard, 1933.
- Inertia forces are applied to the blocks by multiplying seismic coefficient value (k_h) with dry weight of the blocks. For the backfill soil, dry weight of the soil is multiplied by seismic coefficient value (k_h) above SWL and multiplied by modified seismic coefficient value ($k_h^{'}$) below SWL.

In the program after defining static and dynamic forces for the stability analyses, following steps are performed for each block:

- For each block, sliding and overturning factor of safety values are computed. Due to the friction angle (δ) between the wall and the backfill soil, both static and dynamic forces due to soil and surcharge are calculated with their horizontal and vertical components.
- In common, sliding of the blocks is more critical than overturning by the blocks. For the major seismic events overturning is found to be more destructive than sliding. Because of that a higher safety factor for overturning (FS_o) than sliding (FS_s) is recommended.

- Friction coefficients are taken as 0.75, 0.50 and 0.60 for block 1-2 ,block 2-N and block-foundation respectively.

QSAP is verified with Derince Port manual solution [2] and showed good agreement with the results of manual solution. The recent developments in the design of block type quay walls are also reviewed and reflected in this study. With regard to new design concepts, knapsack is taken into consideration, which aims at a more economical solution. A comparative study is carried out between conventional (vertical front face) and knapsack placing methods.

Comparative study between knapsack placed and conventional placed block type quay walls shows that;

- Knapsack formed wall is more stable in sliding for pseudo-static conditions.
- Knapsack formed wall is more stable in overturning in the lower part of the wall. In contrary, it is less stable in the upper part of the wall comparing to conventional formed wall in pseudo-static conditions.
- Knapsack formed wall is increasing the stability of the wall by decreasing the lateral forces 25% and shifting the center of gravity to landside.
- As it is mentioned in the study of Jalili et al., 2008 [7], due to the construction and production difficulties of complex block shapes, knapsack should be chosen for more than 7 m water depth in the whole level of seismic regions.
- Knapsack shape can be a choice in the design of block type quay wall in Turkey, if it is proven economically feasible. In

knapsack design formwork of the blocks are almost the same. Therefore using less number of formwork might result in more economical solution. However, construction methodology has to be considered as an important decision parameter.

For the future studies, the following can be suggested;

- QSAP can be adapted to higher programming tool environment like commercial programs.
- Block shape should be identified without any limitations.
- Variety of loads different than specified and variety of backfill soil can be modeled using the program.
- In ports, if the quay is used as loading-unloading purposes, crane load should be included in the program considering its position and distribution.
- Effects of water level fluctuation on the land side can be added to the program.
- A simplified dynamic analysis model should be developed for the stability of block type quay walls, for performance based design where displacement and rotation limitations are defined as design criteria.

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APPENDIX A:

STABILITY ANALYSIS OF DERINCE PORT BLOCK TYPE QUAY WALL

Stability analysis of Derince Port block type quay wall structure performed in accordance with TSDC-CRA (2008) is provided hereunder [2];

Table A.1 shows the necessary parameters to calculate the forces acting on block type quay walls.

Table A.1: Necessary parameters to calculate the forces acting on block type quay walls

Φ (deg)	**δ(deg)**	**β(deg)**	**α (deg)**	**H (m)**	**H_{sub} (m)**	**q_o (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_d **(kN/m³)**	γ_b **(kN/m³)**	γ_w **(kN/m³)**
800	16.16	23	13.00	18	11	10

1. Coordinates of derince port is 29.80 longitude and 40.8 lattitude for %50 exceedance probability $S_s = 0.761$

2. The soil clasification is chosen as "A" and the short period soil coefficient (F_a) is found by using the tablo 4.1 of TSDC-CRA (2008) [12].

$S_s = 0.76 \Rightarrow F_a = 0.8$

3. $S_{MS} = 0.8 \times 0.76 = 0.61$ (Eq. 4.10)

4. $A_{10} = 0.4 \times 0.61 = 0.243$ (Eq. 4.11)

5. $k_h = (2/3)(0.243) = 0.16$ (Eq. 4.13)

6. Static case; (Eq. 4.2)

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

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 $\left(\delta = \frac{\phi}{3} = 13.33\right)$.

6.1. Active earth pressure;

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

where, p_{ai} is the active pressure for soil class i, K_{ai} is active pressure coefficient for soil class i, γ_j is dry unit weight of soil for soil class j, γ_{bj} is submerged unit weight of soil for soil class j, h_j is thickness of the soil class j, ND is number of the dry soil class according to ground level, q_o is surcharge load, α is angle between the back of the retaining wall and the vertical plane, β is inclined angle with the horizontal. (α and β are zero)

This formulation is divided into two parts; a. active earth presure (p_{ai}), b. surcharge ($p_{ai,s}$)

a. Active Earth Pressure; (Figure A.1)

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

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

$P_{a1} = 0.202(18 \times 2.5 + 11 \times 2) = 13.534$ kN/m^2

$P_{a2} = 0.202(18 \times 2.5 + 11 \times 2 + 11 \times 2.5) = 19.089$ kN/m^2

$P_{a3} = 0.202(18 \times 2.5 + 11 \times 2 + 11 \times 2.5 + 11 \times 2) = 23.533$ kN/m^2

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

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

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

b. Surcharge; (Figure A.2)

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

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

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

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

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

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

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

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

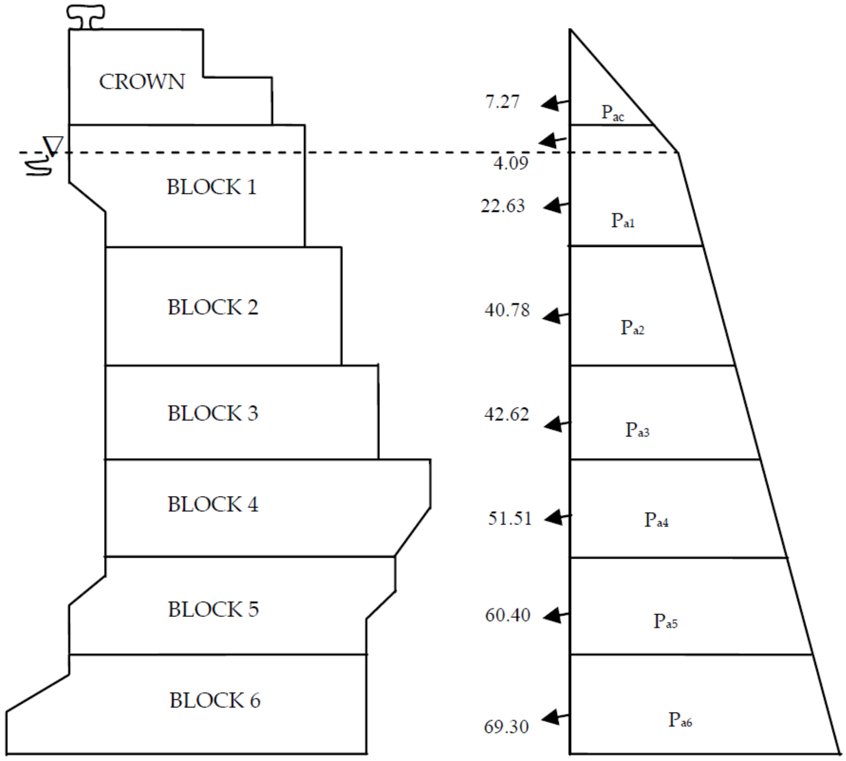


Figure A.1: The soil effect on the block type quay walls for static case

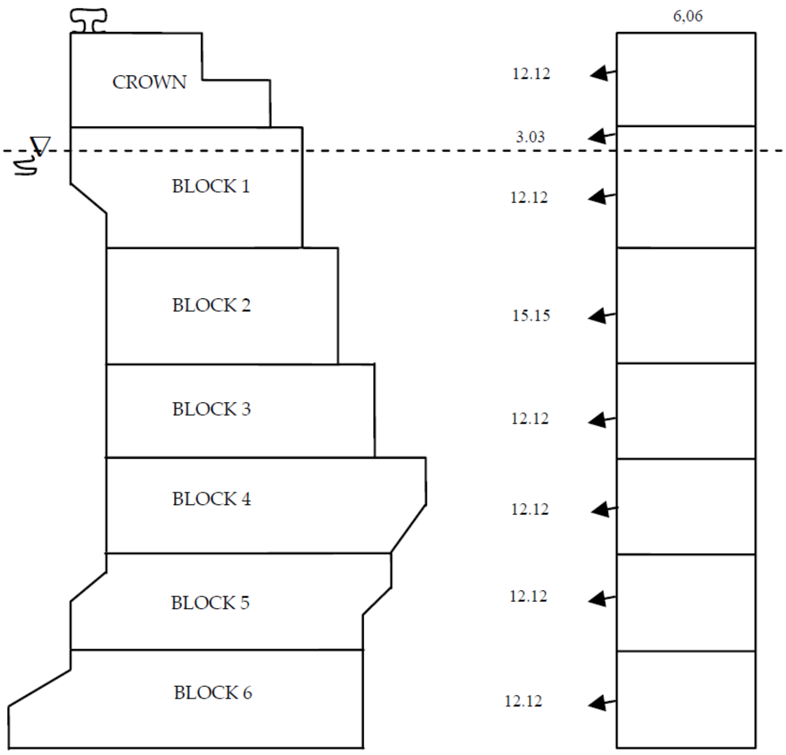


Figure A.2: The surcharge effect on the block type quay walls for static case

7. Dynamic Case;

7.1.For dry part seismic inertia angle;

$$\lambda = \arctan(k_h)$$

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

For dry part dynamic active earth pressure coefficient; (Eq. 4.8)

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

$$K_{AE} = 0.290$$

For dry part dynamic coefficient;

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

7.2.For saturated part:

For saturated part modified seismic coefficient;

$$k_h^I = \frac{18\times2.5+21\times2+21\times2.5+21\times2+21\times2+21\times2+21\times2+15}{18\times2.5+11\times2+11\times2.5+11\times2+11\times2+11\times2+11\times2+15}\times0.16$$

k_h^I=0.26

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

$$\lambda = \arctan(k_h^I)$$

$$\lambda = \arctan(0.26) \quad \Rightarrow \quad \lambda = 14.57^o$$

For saturated part dynamic active earth pressure coefficient;

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

$$K_{AE} = 0.363$$

For dry part dynamic coefficient;

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

7.3 Dynamic Earth Pressure; (Figure A.3)

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

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

$P_{a,d} = 0.088(18\times 2) = 3.17$ 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

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

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

7.4. Dynamic Surcharge Force; (Figure A.4)

$P_{a1,ds} = 0.088\times 30 = 2.64$ kN/m^2

$P_{a,ds} = 0.088\times 30 = 2.64$ kN/m^2

$P_{a2,ds} = 0.161\times 30 = 4.83$ kN/m^2

$P_{a3,ds} = 0.161\times 30 = 4.83$ kN/m^2

$P_{a4,ds} = 0.161\times 30 = 4.83$ kN/m^2

$P_{a5,ds} = 0.161\times 30 = 4.83$ kN/m^2

$P_{a6,ds} = 0.161 \times 30 = 4.83$ kN/m^2

$P_{a7,ds} = 0.161 \times 30 = 4.83$ kN/m^2

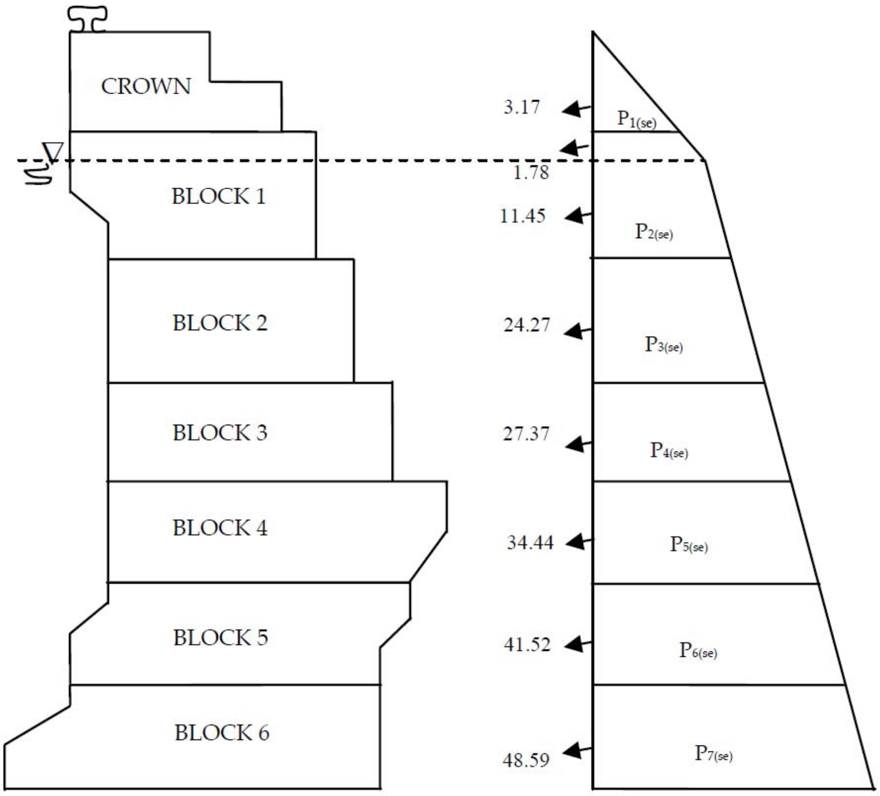


Figure A.3: The soil effect on the block type quay walls for dynamic case

The total thrust (P_{AE}) acting on block type quay wall and the horizontal and vertical components of total thrust are shown in Figure A.5.

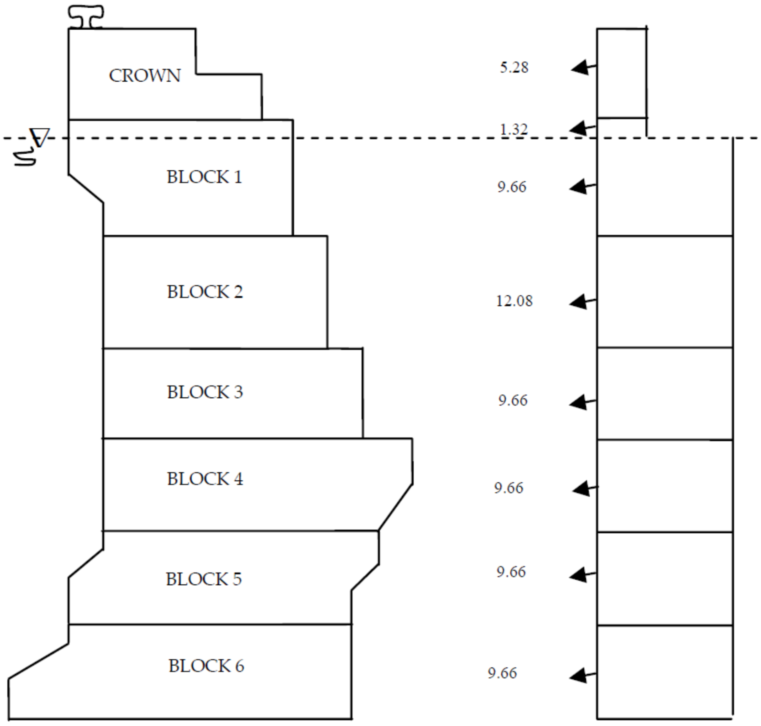


Figure A.4: The surcharge effect on the block type quay wall for dynamic case

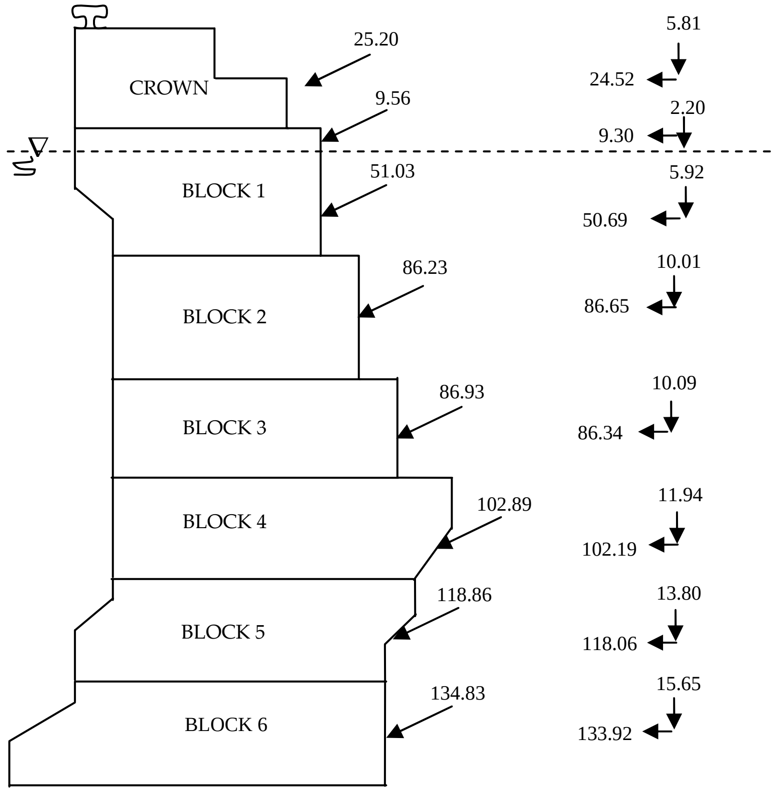


Figure A.5: The total force on the block type quay wall

7.5. Hydrodynamic Force

Using Eq.4.21 and Eq.4.22; 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 **triangular**;

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

The other areas are assumed **trapezoidal**;

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

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

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

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

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

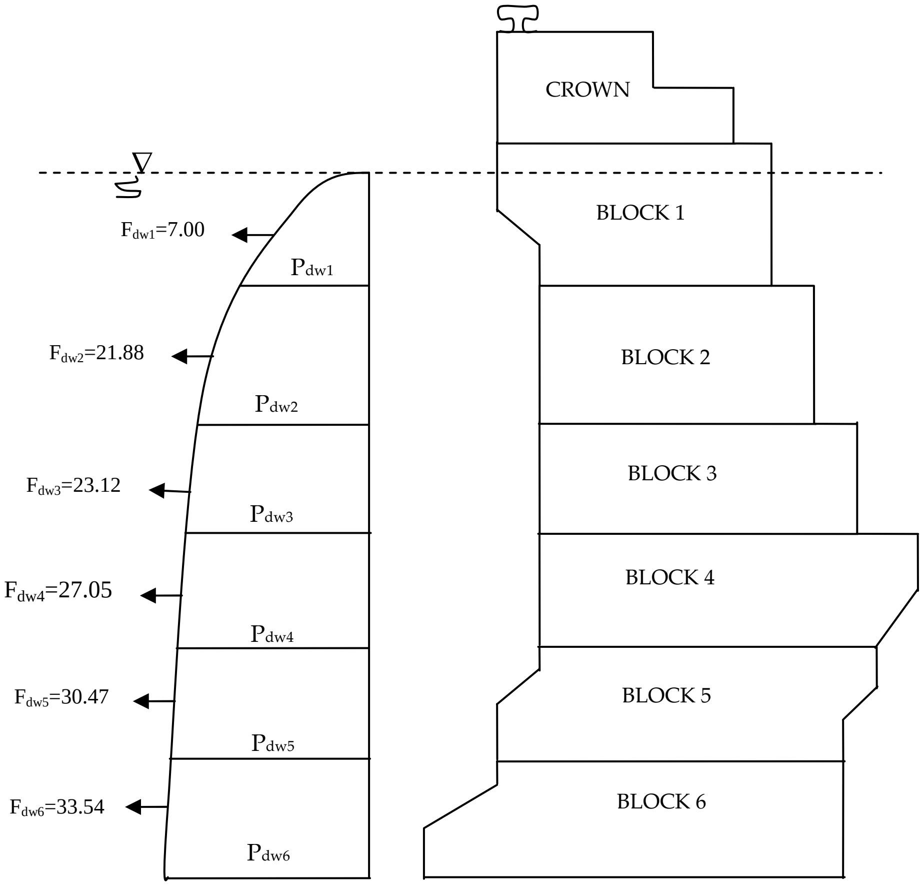


Figure A.6: The hydrodynamic forces on the block type quay wall

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;

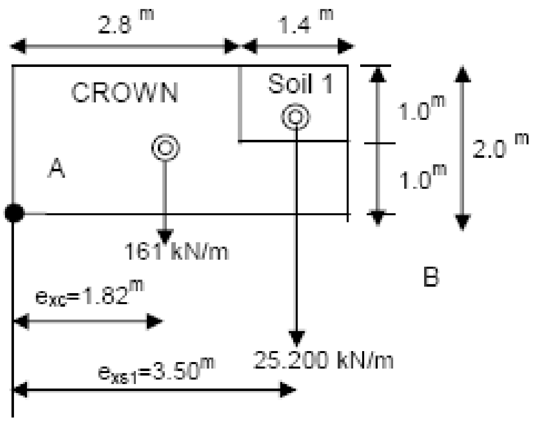
$$e_{xc} = \frac{128.8 \times 1.4 + 32.2 \times (2.8 + 1.4 / 2)}{128.8 + 32.2} = 1.82^m$$

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_{b-conc} 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)

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

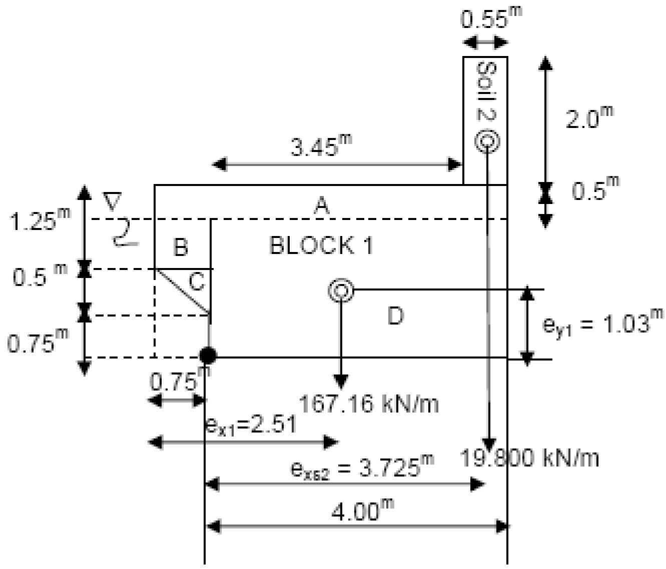
$$\frac{-(0.75)(0.75)(0.75/2)(13)}{167.16}=2.51$$

Soil 2;

W_{S2} = (0.55) (2)(18.00)= 19.8 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_{b-conc} is taken)

W_2= (4.75)(2.5)(13)=154.375 kN/m

The center of gravity;

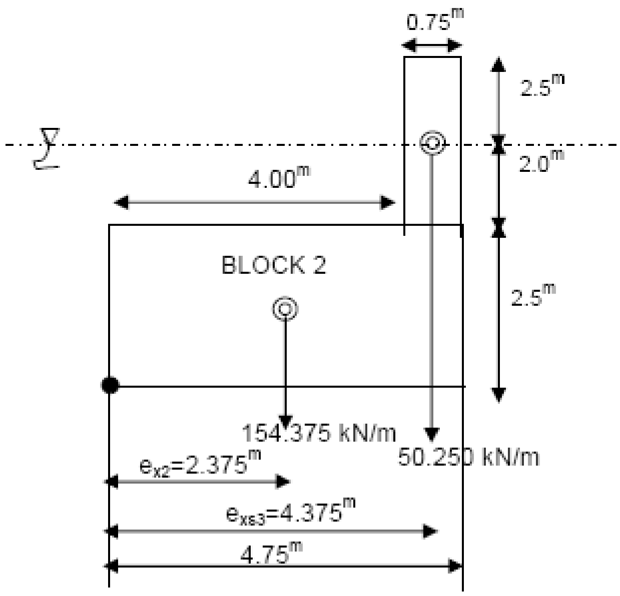
e_{x2}= 4.75/2 +4=2.375^{m}

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_{b-conc} is taken)

W_3= (5.5)(2.0)(13)=143.00 kN/m

The center of gravity;

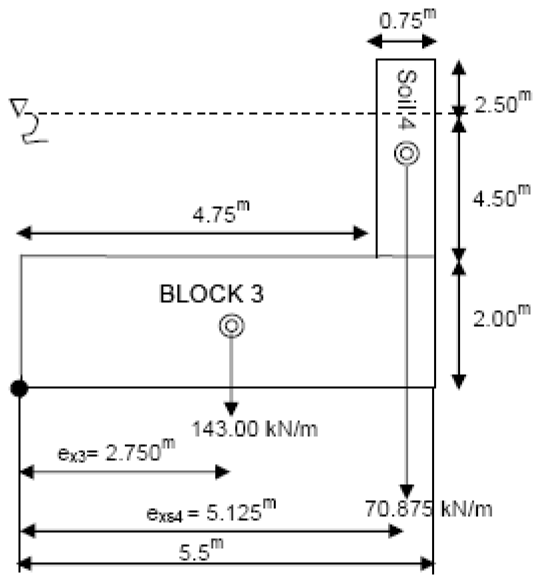
e_{x3}= 5.5/2 =2.75^m

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_{b-conc} is taken)

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

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

W_{4T} = 171.6-4.875 =166.725kN/m

The center of gravity;

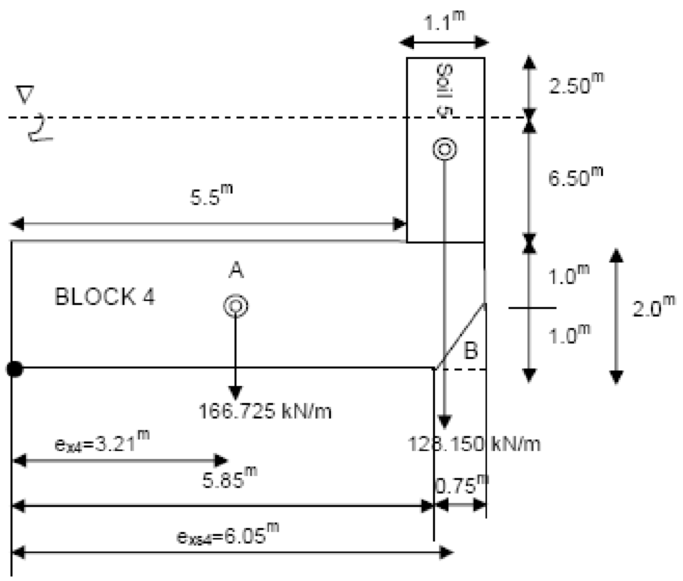
$e_{x4}=\frac{(171.6)(6.6/2)-(4.875)(5.85+0.5)}{166.725}=3.21^m$

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_{b-conc} 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

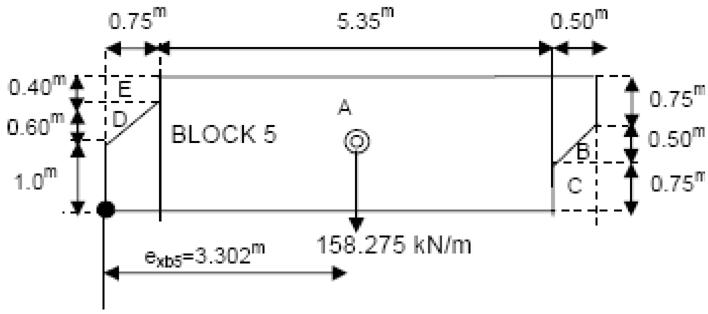
W_{5E}= (0.4)(0.75)(13.0)= 3.9 kN/m

W_{b5}= 171.6-4.875-1.625-2.925 -3.9 =158.275 kN/m

The center of gravity;

$$e_{x5}=\frac{(139.10)(5.35/2+0.75)+(4.875)(0.5/2+6.10)+(1.625)(0.5/3+6.10)}{158.275}$$

$$\frac{+(2.925)(0.75\times 2/3)+(9.75)(0.75/2)}{158.275}=3.302$$



- **Block 6**

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

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

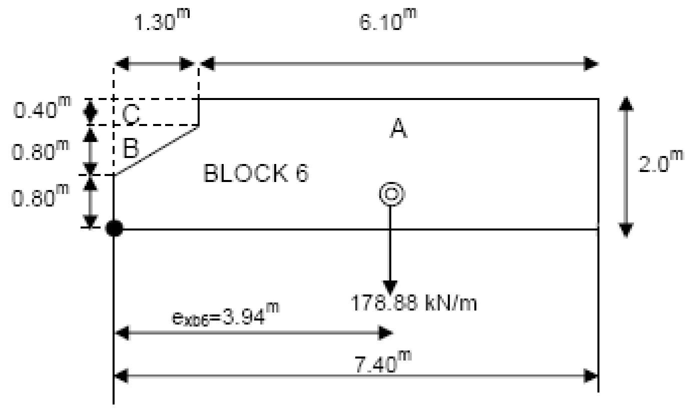
$W_{6B}= \frac{(1.30)(0.8)}{2}(13.0) = 6.76$ kN/m

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

W_{b6}= 192.40-6.76-6.76 = 178.88 kN/m

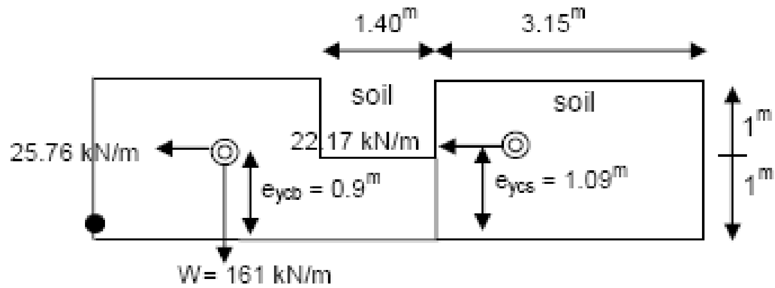
The center of gravity;

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



9. Inertia Forces;

- **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

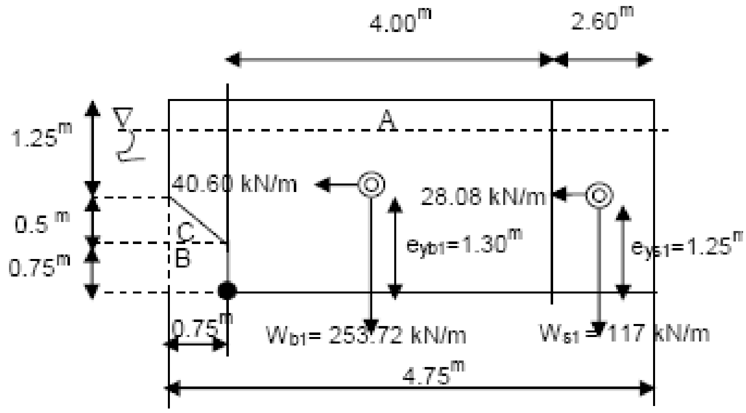
** Total Blocks Force = 128.8+32.2 = 161 kN/m

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

** 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**



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

$e_{yb1} = 1.30^m$

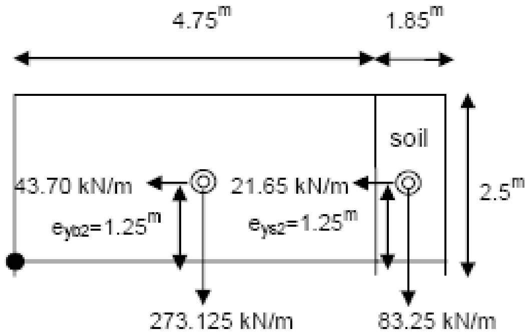
From soil ;

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)

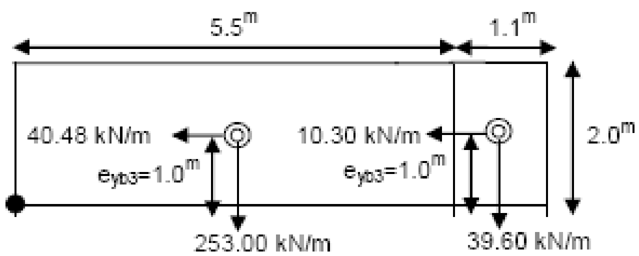
W_{s2}=(1.85)(2.5)(18.0) = 83.25 kN/m

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_{b3}= (5.5)(2.0)(23)=253 kN/m

From Block 3 ;

F_{hb3}= (253.00)(0.16)=40.48 kN/m

$e_{yb3} = 2/2 = 1^m$

Total weight of Soil: (γ =18 kN/m^3)

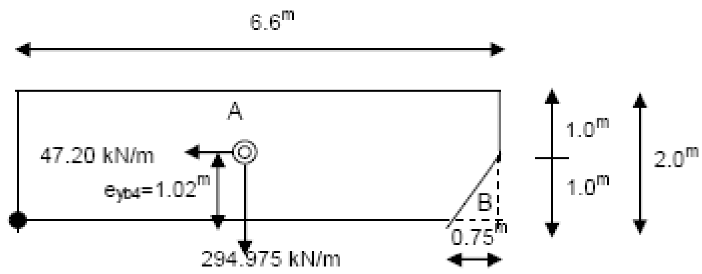
W_{s3}=(1.10)(2.0)(18.0) = 39.6 kN/m

From soil ;

(39.6)(0.26)= 10.30 kN/m

$e_{ys3} = 2/2 = 1^m$

- **Block 4**



Total weight of Block 4:

W_{4A}= (6.6)(2.0)(23)=303.6 kN/m

W_{4B}= $\frac{(0.75)(1.0)}{2}$(23)=8.625 kN/m

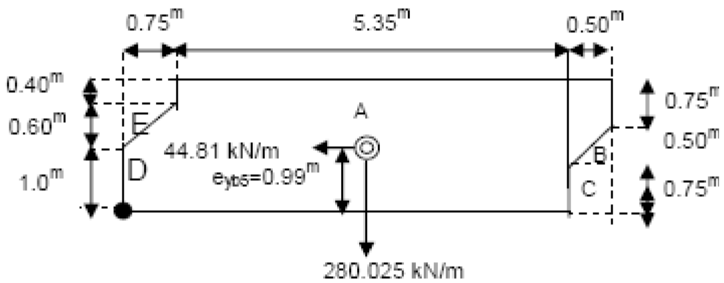
W_{b4}= 303.6-8.625 =294.975 kN/m

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_{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

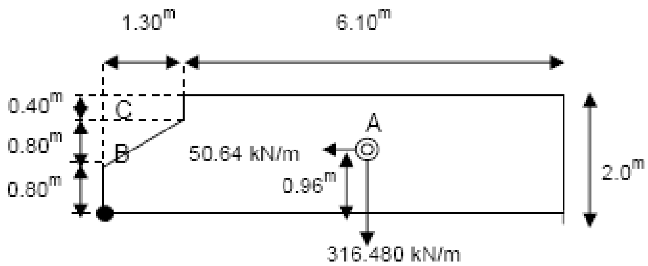
W_{b5}= 303.6-8.625-2.875-5.175-6.9 =280.025 kN/m

From block 5 ;

F_{hb5}= (280.025)(0.16)=44.81 kN/m

$$e_{yb5} = \frac{(303.6)(1.0)-(8.625)(0.75/2)-(2.875)(0.917)}{280.025}$$
$$\frac{-(5.175)(1.40)-(6.9)(1.8)}{280.025} = 0.99$$

- **Block 6**



Total weight of Block 6:

W_{6A}= (7.4)(2.0)(23.0)= 340.40 kN

$W_{6B}= \frac{(1.30)(0.8)}{2}(23.0) = 11.96$ kN

W_{6C}= (1.30)(0.4)(23.0)= 11.96 kN

W_{b6}= 340.40-11.96-11.96 = 316.48 kN

From block 6;

F_{h6}= (316.480)(0.16)=50.64 kN/m

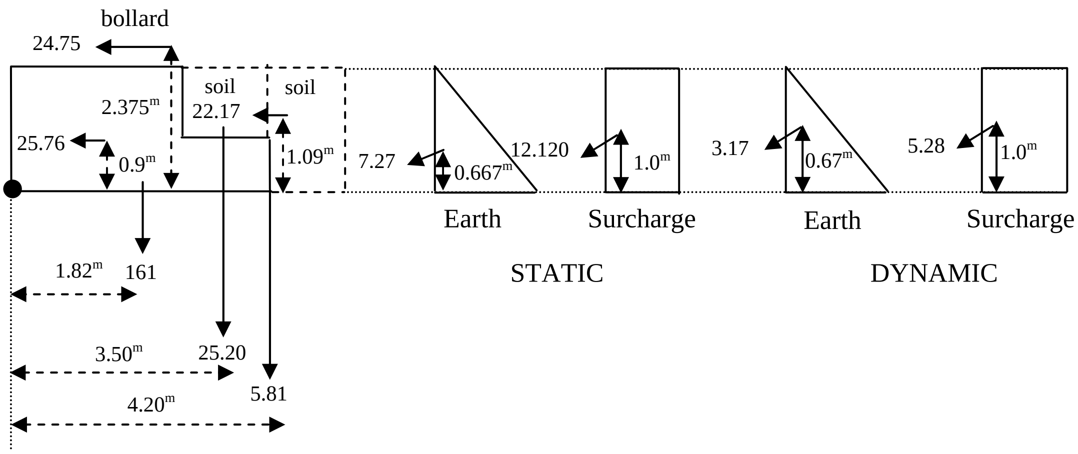
The center of gravity;

e_{yb6}=

$$\frac{(340.40)(2.0/2)-(11.96)(0.8\times 2/3+0.8)-(11.96)(0.4/2+1.6)}{316.48}=0.96$$

10. Stability Analysis;

- **Crown**



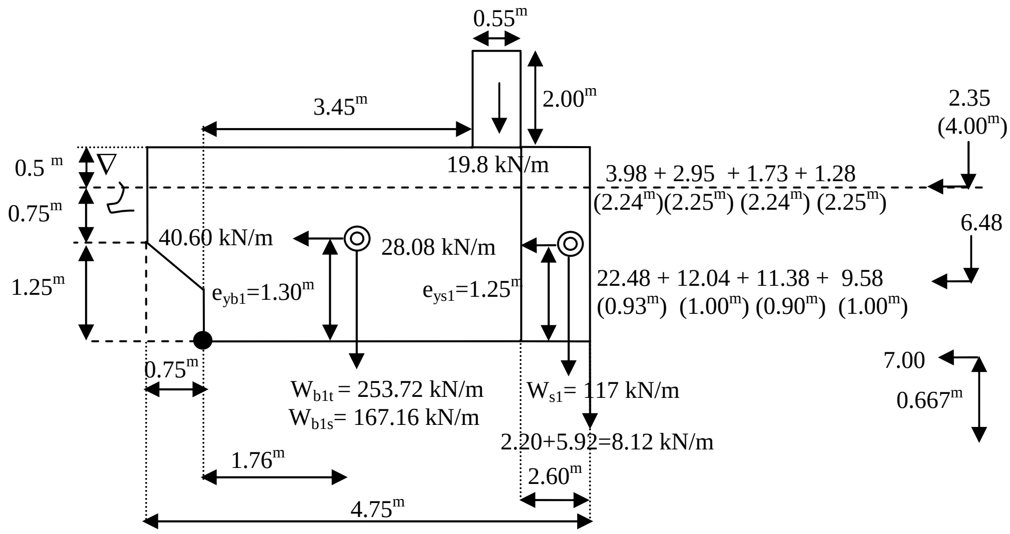
∑ Horizontal Force= 25.76 + 22.17+(7.27)cos13.33+(0.5)(12.12)cos13.33 (3.17)cos13.33+(0.5)(5.28)cos13.33 +24.75 =91.30 kN/m

∑ Vertical Force = 161+25.20+6.42 =192.62 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) =129.57 kNm/m
Resisting Moments = (161)(1.82) +(25.20)(3.50)+(6.42)(4.20) =408.18 kNm/m
Factor of safety for sliding and overturning;

$$FS_s = \frac{(0.50)(192.62)}{91.30} = 1.05 < 1.1 (\ x\ ) \qquad FS_o = \frac{408.18}{129.57} = 3.15 \ > 1.2 \ (OK)$$

- **Block 1**



∑Horizontal Force= 91.3 +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(1.32)cos13.33+0.5(9.66) cos(13.33/2)) +7.00=219.48 kN/m

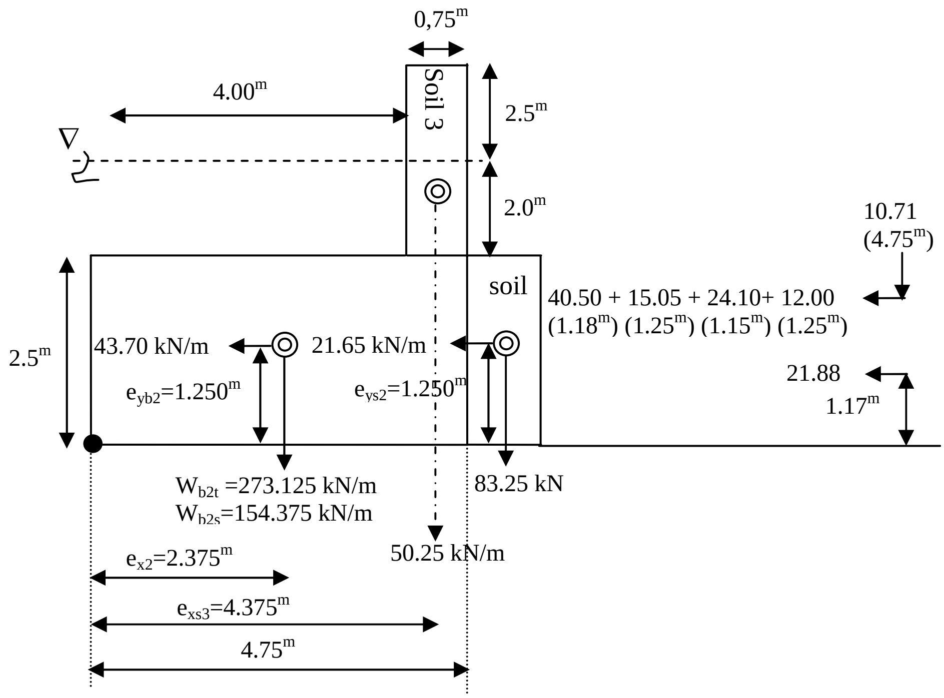
∑ Vertical Force = 192.62 + 167.16 + 19.8+ 8.83= 388.41 kN/m

Overturning Moments= 129.57 +(91.30)(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)+((0.5)(1.28)(2.25)+(0.5)(9.58)(1.00))(1.50)+ (7.0)(2/3) = 529.39 kNm/m

Resisting Moments = (128.8)(1.40-0.75)+(32.2)(2.8-0.75+0.7)+(25.20)(2.75) +(6.42)(3.45)+(167.16)(1.76)+(19.8)(3.725)+(8.83)(4.00)= 666.99 kNm/m

$$FS_s = \frac{(0.50)(388.41)}{219.48} = 0.88 < 1.1 \ \ (\ x\ ) \qquad FS_o = \frac{666.99}{529.39} = 1{,}26 \ \ > 1.2 \ \ \ (\ ok\ )$$

- **Block 2**



∑ Horizontal Force= 219.48+ 43.70 + 21.65+ 40.50 + 0.5 (15.05) +24.10+ 0.5 (12.00)+21.88 = 384.84 kN/m

∑ Vertical Force = 388.41 + 154.375 + 50.25 +10.71= 603.75 kN/m

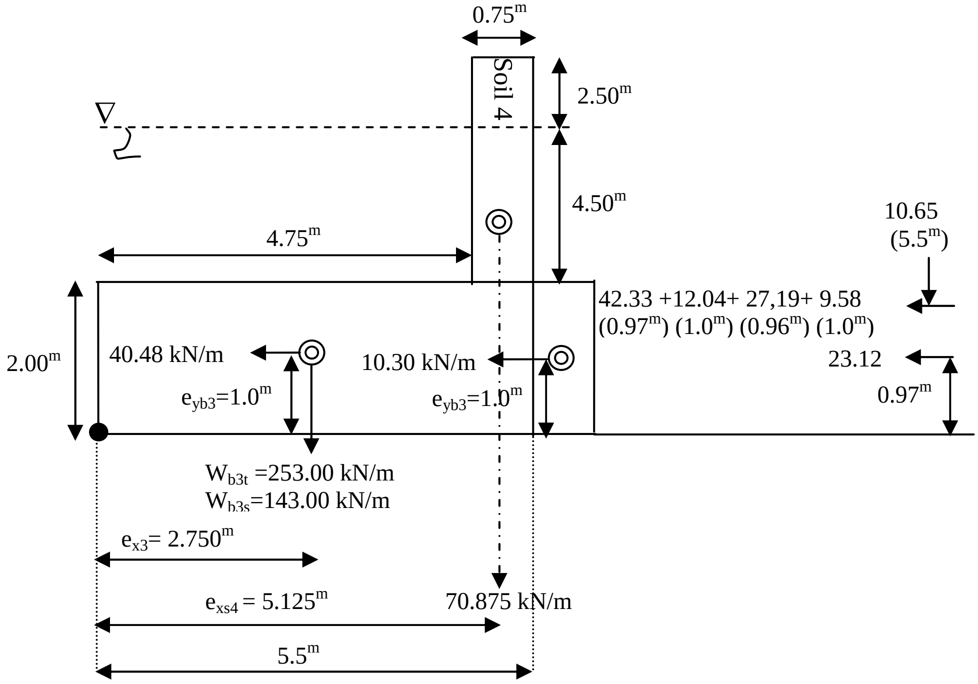
Overturning Moments= 529.39 + (219.48)(2.50)+ (43.70)(1.25) +(21.65)(1.25) +(40.50)(1.18) +(15.05)(1.25)+ (24.10)(1.15)(1.50) +(0.5)(12.00)(1.25)(1.50)+ (21.88)(1.17) = 1304.80 kNm/m

Resisting Moments = 662.05 + (154.375)(2.375) +(50.25)(4.375) +(10.71)(4.75) = 1304.35 kNm/m

$$FS_s = \frac{(0.50)(603.75)}{384.84} = 0.79 < 1.1(\ x\ )$$

$$FS_o = \frac{1304.35}{1304.80} = 1.00 \ \ < 1.2\ (x)$$

- **Block 3**



∑ Horizontal Force= 384.84 + 40.48 + 10.30 + 42.33 + 0.5(12.04)+27.19 + 0.5(9.58)
+ 23.12 =539.07 kN/m

∑ Vertical Force = 603.75 + 143.000 + 70.875 + 10.65 = 828.28 kN/m

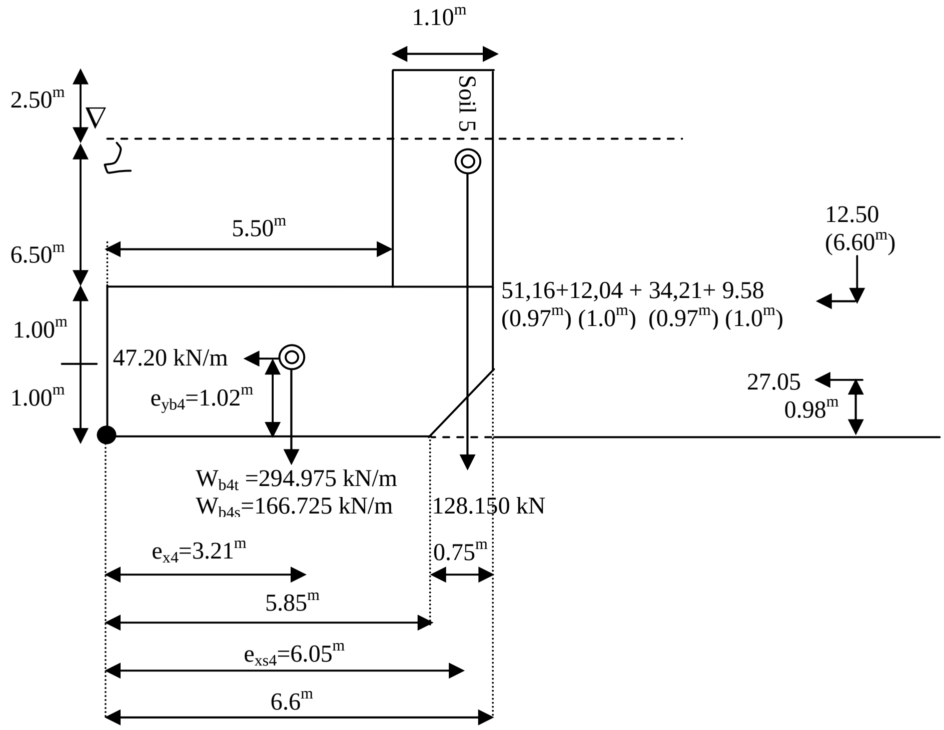
Overturning Moments= 1304.80 + (384.84)(2.00) +(40.48)(1.00)+(10.30)(1.00) +(42.33)(0.97) + (12.04)(1.00) +(27.19)(0.96)(1.50) +(0.5)(9.58)(1.00)(1.50) +(23.12)(0.97) = 2247.13 kNm/m

Resisting Moments = 1304.35 + (143.00)(2.75) +(70.875)(5.125) +(10.65)(5.50) = 2119.41 kNm/m

$$FS_s = \frac{(0.50)(828.28)}{539.07} = 0{,}77 < 1.1(\text{ x })$$

$$FS_o = \frac{2119.41}{2247.13} = 0.94 \quad < 1.2\ (\text{x})$$

- **Block 4**



∑ Horizontal Force= 539.07 + 47.20 + 51.16 + 0.5(12.04) + 34.21 +0.5(9.58) + 27.05
= 709.50 kN/m

∑ Vertical Force = 828.28 +166.725+128.15+12.50 = 1135.66 kN/m

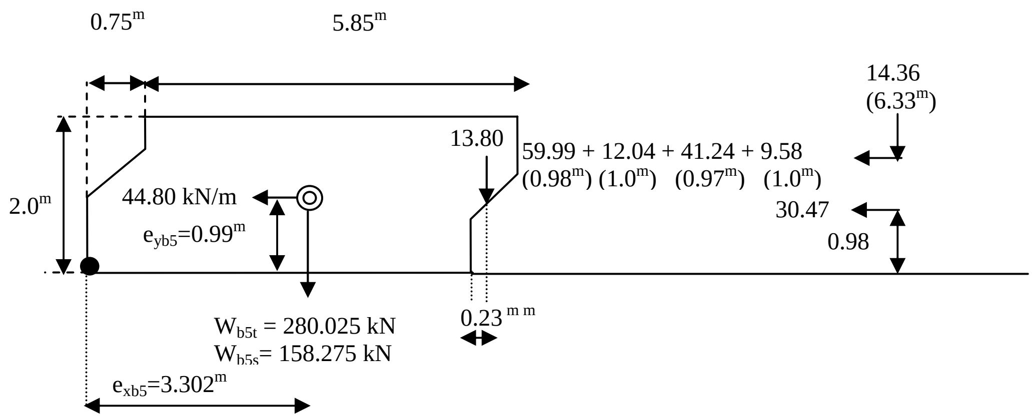
Overturning Moments= 2247.13 + (539.07)(2.00) + (47.20)(1.02) + (51.16)(0.97) +(12.04)(1.00)+(34.21)(0.97)(1.50)+(0.5)(9.58)(1.00)(1.50)+(27.05)(0.98) = 3518.55 kNm/m

Resisting Moments = 2119.41+(166.725)(3.21)+(128.15)(6.05) +(12.50)(6.60) =3512.40 kNm/m

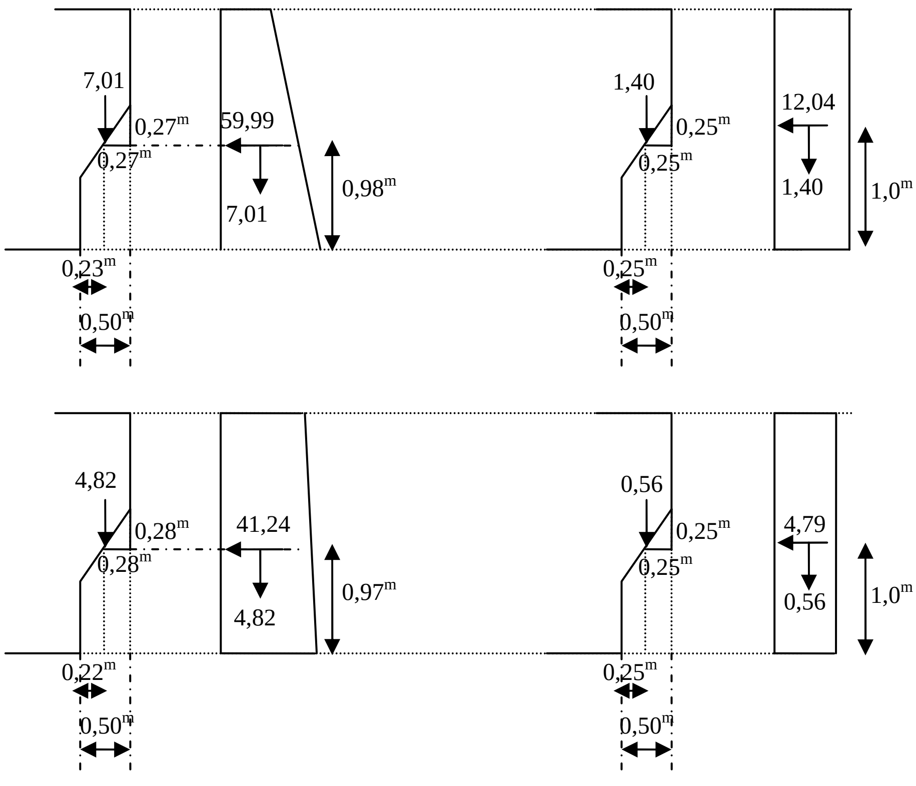
$$FS_s = \frac{(0.50)(1135.66)}{709.50} = 0.80 < 1.1(\ x\ )$$

$$FS_o = \frac{3512.40}{3518.55} = 1.00 \quad < 1.2\ (x)$$

- **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=** 709.50+ 44.80+ 59.99 + 0.5(12.04)+41.24+0.5(9.58)+ 30.47

= 896.81 kN/m

∑ **Vertical Force =** 1135.66 + 158.275 + 14.36 = 1308.30 kN/m

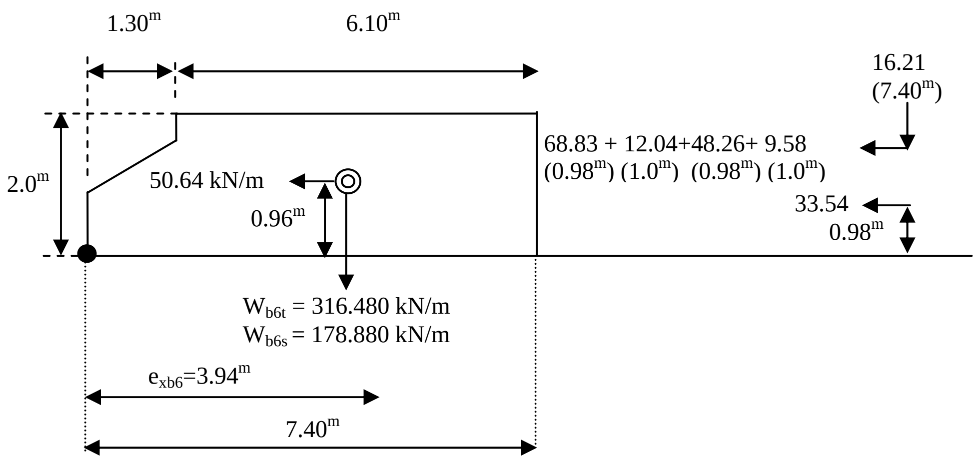
Overturning Moments= 3518.55 + (709.50)(2.00) + (44.80)(0.99)+(59.99)(0.98) + (12.04)(1.00) + (41.24)(0.97)(1.50) +(0.5)(9.58)(1.00)(1.50) +(30.47)(0.98) = 5090.99 kNm/m

Resisting Moments = 3512.40 + (1135.66) (0.75) + (158.275)(3.302) + (14.36)(6.33) = 4977.67 kNm/m

$$FS_s = \frac{(0.50)(1308.30)}{896.81} = 0.73 < 1.1(\ x\ )$$

$$FS_o = \frac{4977.67}{5090.99} = 0.98 \ < 1.2\ (x)$$

- **Block 6**



∑ **Horizontal Force =** 896.81 + 50.64 + 68.83 + 0.5(12.04) + 48.26 + 0.5(9.58) + 33.54 = 1108.89 kN/m

∑ **Vertical Force =** 1308.30 + 178.88 + 16.21 = 1503.39 kN/m

Overturning Moments= 5090.99 + (896.81)(2.00) + (50.64)(0.96) + (68.83)(0.98) + (12.04)(1.00) + (48.26)(0.98)(1.50) + (0.5)(9.58)(1.00)(1.50) + (33.54)(0.98) = 7123.71 kNm/m

Resisting Moments = 4977.67 + (1308.30)(1.30) + (178.880)(3.94) + (16.21)(7.40) = 7503.20 kNm/m

$$FS_s = \frac{(0.60)(1503.39)}{1108.89} = 0.81 < 1.1(\text{ x }) \quad FS_o = \frac{7503.20}{7123.71} = 1.05 \; < 1.2\ (\text{x})$$

Table A.2: Factor of safety values against sliding and overturning for (TSDC-CRA, 2008)

TSDC-CRA 2008		
Block	**FS_s**	**Fs_o**
Crown	1.05	3.15
Block 1	0.88	1.26
Block 2	0.79	1.00
Block 3	0.77	0.94
Block 4	0.80	1.00
Block 5	0.73	0.98
Block 6	0.81	1.05

APPENDIX B

STABILITY ANALYSIS OF CONVENTIONAL AND KNAPSACK PLACED WALLS

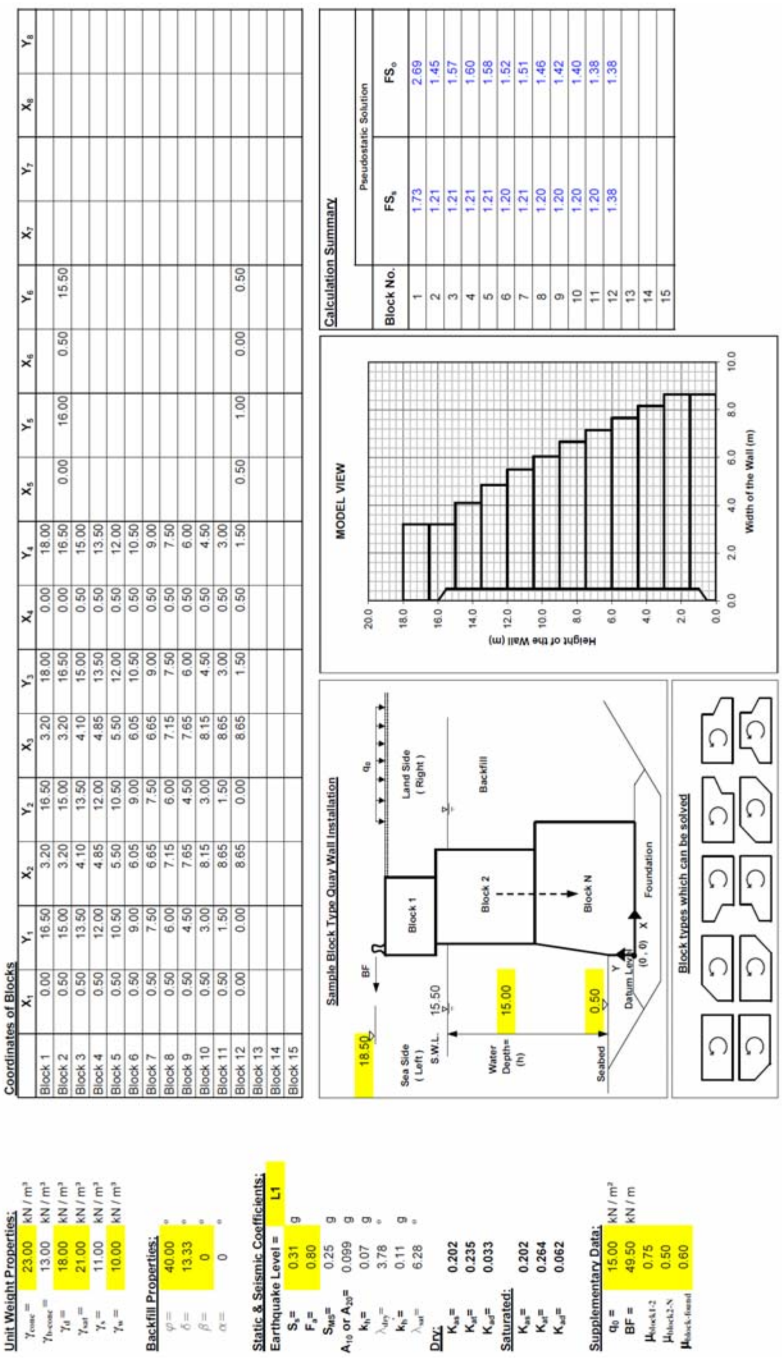


Figure B.1 Start page of QSAP (Conventional Placed)

Block 1 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
P_1	110.40	1.60	176.64
F_{s1v}	1.05	3.20	3.35
F_{e1v}	0.94	3.20	3.02

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
BF	49.50	2.00	99.00
F_{s1h}	4.42	0.75	3.32
F_{e1h}	3.98	0.50	1.99

Pseudostatic Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
P_{1d}	7.30	0.75	5.48
F_{ds1h}	0.72	0.75	0.54
F_{de1h}	0.65	0.50	0.33
T_{filld}	9.73	0.75	7.30
F_{ds1v}	0.17	3.20	0.55
F_{de1v}	0.15	3.20	0.49

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
112.72	48.99	184.05	68.48

$FS_{sliding}$ (μ*$\Sigma F_v/\Sigma F_h$)= 1.73 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 2.69 >1.30 OK**

Figure B.2: QSAP stability calculation for block-1 (Conventional Placed)

Block 2 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1	112.39	-0.50	-56.20
P_2	88.28	1.22	107.41
F_{s2v}	0.87	2.70	2.36
F_{e2v}	1.26	2.70	3.40
F_{e3v}	0.42	2.70	1.13
F_{e4v}	0.53	2.70	1.42
F_{e5v}	0.03	2.70	0.09
T_{fill}	0.00	2.70	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1	57.90	1.50	86.85
F_{s2h}	4.45	0.75	3.34
F_{e2h}	5.31	1.00	5.31
F_{e3h}	1.77	0.83	1.47
F_{e4h}	4.51	0.25	1.13
F_{e5h}	0.28	0.17	0.05

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1	48.99	1.50	73.48
P_{2d}	6.73	0.78	5.25
F_{ds2h}	0.94	0.63	0.60
F_{de2h}	0.87	1.00	0.87
F_{de3h}	0.29	0.83	0.24
F_{de4h}	0.74	0.25	0.18
F_{de5h}	0.08	0.17	0.01
T_{filld}	11.88	0.66	7.84
F_{dw}	0.40	0.17	0.07
F_{ds2v}	0.17	2.70	0.45
F_{de2v}	0.21	2.70	0.55
F_{de3v}	0.07	2.70	0.18
F_{de4v}	0.09	2.70	0.23
F_{de5v}	0.01	2.70	0.03

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
204.64	84.54	245.12	168.81

$FS_{sliding}$ (μ*$\Sigma F_v/\Sigma F_h$)= 1.21 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.45 >1.30 OK**

Figure B.3: QSAP stability calculation for block-2 (Conventional Placed)

Block 3 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	203.78	0.00	0.00
P_3	70.20	1.80	126.36
F_{s3v}	0.53	3.60	1.90
F_{e6v}	1.78	3.60	6.39
F_{e7v}	0.29	3.60	1.04
T_{fill}	45.45	3.15	143.17

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	74.22	1.50	111.33
F_{s3h}	4.51	0.75	3.39
F_{e6h}	15.20	0.75	11.40
F_{e7h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	84.54	1.50	126.81
P_{3d}	8.21	0.75	6.16
F_{ds3h}	1.39	0.75	1.04
F_{de6h}	2.74	0.75	2.05
F_{de7h}	0.77	0.50	0.38
T_{filld}	13.51	0.75	10.14
F_{dw}	3.57	0.67	2.38
F_{ds3v}	0.16	3.60	0.58
F_{de6v}	0.55	3.60	1.96
F_{de7v}	0.09	3.60	0.32

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
323.68	133.97	526.85	334.76

$FS_{sliding}$ (μ*$\Sigma F_v/\Sigma F_h$)= 1.21 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.57 >1.30 OK**

Figure B.4: QSAP stability calculation for block-3 (Conventional Placed)

Block 4 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	322.02	0.00	0.00
P_4	84.83	2.18	184.49
F_{s4v}	0.53	4.35	2.29
F_{e8v}	2.36	4.35	10.25
F_{e9v}	0.29	4.35	1.26
T_{fill}	50.25	3.98	199.74

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	96.42	1.50	144.63
F_{s4h}	4.51	0.75	3.39
F_{e8h}	20.16	0.75	15.12
F_{e9h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	133.97	1.50	200.95
P_{4d}	9.92	0.75	7.44
F_{ds4h}	1.39	0.75	1.04
F_{de8h}	4.27	0.75	3.21
F_{de9h}	0.77	0.50	0.38
T_{filld}	11.29	0.75	8.46
F_{dw}	5.52	0.72	3.95
F_{ds4v}	0.16	4.35	0.70
F_{de8v}	0.72	4.35	3.15
F_{de9v}	0.09	4.35	0.39

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
462.90	191.34	929.13	581.48

$FS_{sliding}$ (μ*$\Sigma F_v/\Sigma F_h$)= 1.21 **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.60 **>1.30 OK**

Figure B.5: QSAP stability calculation for block-4 (Conventional Placed)

Block 5 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	460.27	0.00	0.00
P_5	97.50	2.50	243.75
F_{s5v}	0.53	5.00	2.64
F_{e10v}	2.94	5.00	14.68
F_{e11v}	0.29	5.00	1.45
T_{fill}	54.28	4.68	253.74

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	123.58	1.50	185.37
F_{s5h}	4.51	0.75	3.39
F_{e10h}	25.13	0.75	18.85
F_{e11h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	191.34	1.50	287.01
P_{5d}	11.41	0.75	8.56
F_{ds5h}	1.39	0.75	1.04
F_{de10h}	5.81	0.75	4.36
F_{de11h}	0.77	0.50	0.38
T_{filld}	9.36	0.75	7.02
F_{dw}	6.90	0.73	5.02
F_{ds5v}	0.16	5.00	0.81
F_{de10v}	0.90	5.00	4.51
F_{de11v}	0.09	5.00	0.45

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
619.58	256.14	1451.15	920.45

$FS_{sliding}$ (μ*$\Sigma F_v/\Sigma F_h$)= **1.21** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.58 **>1.30 OK**

Figure B.6: QSAP stability calculation for block-5 (Conventional Placed)

Block 6 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	615.80	0.00	0.00
P_6	108.23	2.78	300.32
F_{s6v}	0.53	5.55	2.93
F_{e12v}	3.52	5.55	19.52
F_{e13v}	0.29	5.55	1.61
T_{fill}	55.00	5.28	290.13

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	155.71	1.50	233.56
F_{s6h}	4.51	0.75	3.39
F_{e12h}	30.10	0.75	22.57
F_{e13h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	256.14	1.50	384.21
P_{6d}	12.66	0.75	9.50
F_{ds6h}	1.39	0.75	1.04
F_{de12h}	7.34	0.75	5.51
F_{de13h}	0.77	0.50	0.38
T_{filld}	7.72	0.75	5.79
F_{dw}	8.04	0.73	5.90
F_{ds6v}	0.16	5.55	0.90
F_{de12v}	1.08	5.55	5.99
F_{de13v}	0.09	5.55	0.49

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
788.47	328.21	2073.04	1362.66

$FS_{sliding}$ (μ*$\Sigma F_v/\Sigma F_h$)= 1.20 **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.52 **>1.30 OK**

Figure B.7: QSAP stability calculation for block-6 (Conventional Placed)

Block 7 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	783.36	0.00	0.00
P_7	119.93	3.08	368.77
F_{s7v}	0.53	6.15	3.24
F_{e14v}	4.10	6.15	25.20
F_{e15v}	0.29	6.15	1.78
T_{fill}	69.90	5.85	408.92

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	192.80	1.50	289.20
F_{s7h}	4.51	0.75	3.39
F_{e14h}	35.06	0.75	26.30
F_{e15h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	328.21	1.50	492.31
P_{7d}	14.03	0.75	10.52
F_{ds7h}	1.39	0.75	1.04
F_{de14h}	8.88	0.75	6.66
F_{de15h}	0.77	0.50	0.38
T_{filld}	5.94	0.75	4.46
F_{dw}	9.04	0.74	6.66
F_{ds7v}	0.16	6.15	1.00
F_{de14v}	1.26	6.15	7.73
F_{de15v}	0.09	6.15	0.55

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
984.72	407.36	2890.22	1918.88

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.21 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.51 >1.30 OK**

Figure B.8: QSAP stability calculation for block-7 (Conventional Placed)

Block 8 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-7	978.10	0.00	0.00
P_8	129.68	3.33	431.17
F_{s8v}	0.53	6.65	3.51
F_{e16v}	4.68	6.65	31.10
F_{e17v}	0.29	6.65	1.93
T_{fill}	66.50	6.40	425.60

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-7	234.86	1.50	352.28
F_{s8h}	4.51	0.75	3.39
F_{e16h}	40.03	0.75	30.02
F_{e17h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-7	407.36	1.50	611.04
P_{8d}	15.17	0.75	11.38
F_{ds8h}	1.39	0.75	1.04
F_{de16h}	10.41	0.75	7.81
F_{de17h}	0.77	0.50	0.38
T_{filld}	4.46	0.75	3.34
F_{dw}	9.94	0.74	7.34
F_{ds8v}	0.16	6.65	1.08
F_{de16v}	1.44	6.65	9.55
F_{de17v}	0.09	6.65	0.59

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1188.08	493.56	3794.75	2599.70

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.20 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.46 >1.30 OK**

Figure B.9: QSAP stability calculation for block-8 (Conventional Placed)

Block 9 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-8	1179.77	0.00	0.00
P_9	139.43	3.58	498.44
F_{s9v}	0.53	7.15	3.77
F_{e18v}	5.26	7.15	37.59
F_{e19v}	0.29	7.15	2.07
T_{fill}	74.75	6.90	515.78

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-8	281.88	1.50	422.82
F_{s9h}	4.51	0.75	3.39
F_{e18h}	44.99	0.75	33.74
F_{e19h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-8	493.56	1.50	740.34
P_{9d}	16.31	0.75	12.24
F_{ds9h}	1.39	0.75	1.04
F_{de18h}	11.95	0.75	8.96
F_{de19h}	0.77	0.50	0.38
T_{filld}	2.97	0.75	2.23
F_{dw}	10.76	0.74	7.97
F_{ds9v}	0.16	7.15	1.16
F_{de18v}	1.61	7.15	11.54
F_{de19v}	0.09	7.15	0.64

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1410.19	586.74	4865.74	3415.64

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.20 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.42 >1.30 OK**

Figure B.10: QSAP stability calculation for block-9 (Conventional Placed)

Block 10 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-9	1400.02	0.00	0.00
P_{10}	149.18	3.83	570.59
F_{s10v}	0.53	7.65	4.04
F_{e20v}	5.84	7.65	44.66
F_{e21v}	0.29	7.65	2.22
T_{fill}	83.00	7.40	614.20

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-9	333.87	1.50	500.80
F_{s10h}	4.51	0.75	3.39
F_{e20h}	49.96	0.75	37.47
F_{e21h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-9	586.74	1.50	880.11
P_{10d}	17.45	0.75	13.09
F_{ds10h}	1.39	0.75	1.04
F_{de20h}	13.48	0.75	10.11
F_{de21h}	0.77	0.50	0.38
T_{filld}	1.49	0.75	1.11
F_{dw}	11.52	0.74	8.55
F_{ds10v}	0.16	7.65	1.24
F_{de20v}	1.79	7.65	13.71
F_{de21v}	0.09	7.65	0.68

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1651.07	686.83	6117.07	4377.11

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= **1.20** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** **1.40** **>1.30 OK**

Figure B.11: QSAP stability calculation for block-10 (Conventional Placed)

Block 11 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-10	1638.85	0.00	0.00
P_{11}	158.93	4.08	647.62
F_{s11v}	0.53	8.15	4.30
F_{e22v}	6.42	8.15	52.31
F_{e23v}	0.29	8.15	2.36
T_{fill}	91.25	7.90	720.88

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-10	390.82	1.50	586.24
F_{s11h}	4.51	0.75	3.39
F_{e22h}	54.92	0.75	41.19
F_{e23h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block1-10	686.83	1.50	1030.25
P_{11d}	18.60	0.75	13.95
F_{ds11h}	1.39	0.75	1.04
F_{de22h}	15.02	0.75	11.26
F_{de23h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	12.23	0.74	9.09
F_{ds11v}	0.16	8.15	1.32
F_{de22v}	1.97	8.15	16.05
F_{de23v}	0.09	8.15	0.73

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1910.70	793.80	7562.64	5494.46

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= **1.20** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** **1.38** **>1.30 OK**

Figure B.12: QSAP stability calculation for block-11 (Conventional Placed)

Block 12 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-11	1896.26	0.50	948.13
P_{12}	163.80	4.45	728.44
F_{s12v}	0.53	8.65	4.56
F_{e24v}	7.00	8.65	60.54
F_{e25v}	0.29	8.65	2.51
T_{fill}	0.00	8.65	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-11	452.74	1.50	679.12
F_{s12h}	4.51	0.75	3.39
F_{e24h}	59.89	0.75	44.92
F_{e25h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block1-11	793.80	1.50	1190.70
P_{12d}	19.17	0.74	14.17
F_{ds12h}	1.39	0.75	1.04
F_{de24h}	16.55	0.75	12.41
F_{de25h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	8.53	1.00	8.51
F_{ds12v}	0.16	8.65	1.40
F_{de24v}	2.15	8.65	18.58
F_{de25v}	0.09	8.65	0.77

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
2084.71	904.14	9327.56	6777.35

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= **1.38** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** **1.38** **>1.30 OK**

Figure B.13: QSAP stability calculation for block-12 (Conventional Placed)

Unit Weight Properties:

γ_{conc} =	23.00	kN / m³
$\gamma_{b\text{-}conc}$ =	13.00	kN / m³
γ_d =	18.00	kN / m³
γ_{sat} =	21.00	kN / m³
γ_s =	11.00	kN / m³
γ_w =	10.00	kN / m³

Backfill Properties:

φ =	40.00	°
δ =	13.33	°
β =	0	°
α =	0	°

Static & Seismic Coefficients:

Earthquake Level =	L1	
S_s=	0.31	g
F_a=	0.80	
S_{MS}=	0.25	g
A_{10} or A_{20}=	0.099	g
k_h=	0.07	g
λ_{dry}=	3.78	°
k_h'=	0.11	g
λ_{sat}=	6.28	°
Dry:		
K_{as}=	0.202	
K_{at}=	0.235	
K_{ad}=	0.033	
Saturated:		
K_{as}=	0.202	
K_{at}=	0.264	
K_{ad}=	0.062	

Supplementary Data:

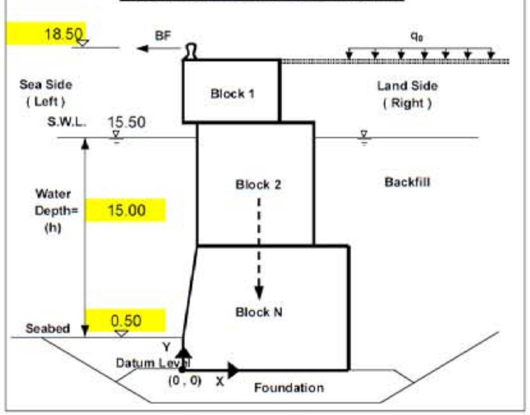
q_0 =	15.00	kN / m²
BF =	49.50	kN / m
$\mu_{block1\text{-}2}$	0.75	
$\mu_{block2\text{-}N}$	0.50	
$\mu_{block\text{-}found}$	0.60	

BLOCK TYPE QUAY WALL STABILITY ANALYSIS PROGRAM (QSAP)

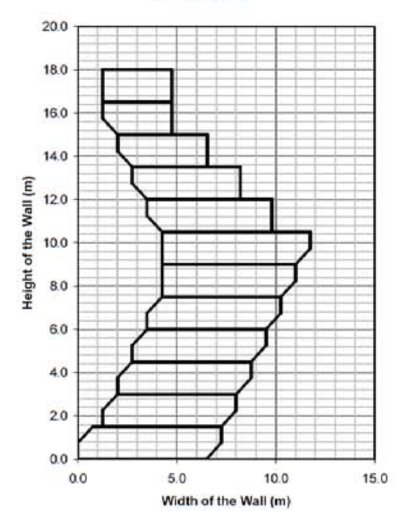
Coordinates of Blocks

	X_1	Y_1	X_2	Y_2	X_3	Y_3	X_4	Y_4	X_5	Y_5	X_6	Y_6	X_7	Y_7	X_8	Y_8
Block 1	1.25	16.50	4.75	16.50	4.75	18.00	1.25	18.00								
Block 2	2.00	15.00	4.75	15.00	4.75	16.50	1.25	16.50	1.25	15.75						
Block 3	2.75	13.50	6.55	13.50	6.55	15.00	2.00	15.00	2.00	14.25						
Block 4	3.50	12.00	8.20	12.00	8.20	13.50	2.75	13.50	2.75	12.75						
Block 5	4.25	10.50	9.80	10.50	9.80	12.00	3.50	12.00	3.50	11.25						
Block 6	4.25	9.00	11.00	9.00	11.75	9.75	11.75	10.50	4.25	10.50						
Block 7	4.25	7.50	10.25	7.50	11.00	8.25	11.00	9.00	4.25	9.00						
Block 8	3.50	6.00	9.50	6.00	10.25	6.75	10.25	7.50	4.25	7.50	3.50	6.75				
Block 9	2.75	4.50	8.75	4.50	9.50	5.25	9.50	6.00	3.50	6.00	2.75	5.25				
Block 10	2.00	3.00	8.00	3.00	8.75	3.75	8.75	4.50	2.75	4.50	2.00	3.75				
Block 11	1.25	1.50	7.25	1.50	8.00	2.25	8.00	3.00	2.00	3.00	1.25	2.25				
Block 12	0.00	0.00	6.50	0.00	7.25	0.75	7.25	1.50	0.75	1.50	0.00	0.75				
Block 13																
Block 14																
Block 15																

Sample Block Type Quay Wall Installation



Block types which can be solved



Calculation Summary

Block No.	Pseudostatic Solution FS_s	FS_o
1	1.76	3.09
2	1.23	1.37
3	1.40	1.36
4	1.53	1.35
5	1.65	1.36
6	1.82	2.21
7	1.66	1.82
8	1.52	1.91
9	1.40	1.92
10	1.30	1.89
11	1.21	1.84
12	1.38	1.93
13		
14		
15		

Figure B.14 Start page of QSAP (Knapsack Placed)

Block 1 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
P_1	120.75	1.75	211.31
F_{s1v}	1.05	3.50	3.67
F_{e1v}	0.94	3.50	3.30

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
BF	49.50	2.00	99.00
F_{s1h}	4.42	0.75	3.32
F_{e1h}	3.98	0.50	1.99

Pseudostatic Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
P_{1d}	7.99	0.75	5.99
F_{ds1h}	0.72	0.75	0.54
F_{de1h}	0.65	0.50	0.33
T_{filld}	12.50	0.75	9.37
F_{ds1v}	0.17	3.50	0.60
F_{de1v}	0.15	3.50	0.54

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v, kN)	Total Horizontal Force (ΣF_h, kN)	Total Resisting Moment (ΣM_r, kN-m)	Total Overturning Moment (ΣM_o, kN-m)
123.07	52.44	219.42	71.06

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.76 >1.20 OK

$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 3.09 >1.30 OK

Figure B.15: QSAP stability calculation for block-1 (Knapsack Placed)

Block 2 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1	122.74	-0.75	-92.06
P_2	100.53	1.08	109.07
F_{s2v}	0.87	2.75	2.40
F_{e2v}	1.26	2.75	3.46
F_{e3v}	0.42	2.75	1.15
F_{e4v}	0.53	2.75	1.45
F_{e5v}	0.03	2.75	0.09
T_{fill}	0.00	2.75	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1	57.90	1.50	86.85
F_{s2h}	4.45	0.75	3.34
F_{e2h}	5.31	1.00	5.31
F_{e3h}	1.77	0.83	1.47
F_{e4h}	4.51	0.25	1.13
F_{e5h}	0.28	0.17	0.05

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1	52.44	1.50	78.66
P_{2d}	7.56	0.78	5.88
F_{ds2h}	0.94	0.63	0.60
F_{de2h}	0.87	1.00	0.87
F_{de3h}	0.29	0.83	0.24
F_{de4h}	0.74	0.25	0.18
F_{de5h}	0.08	0.17	0.01
T_{filld}	15.26	0.66	10.07
F_{dw}	0.40	0.17	0.07
F_{ds2v}	0.17	2.75	0.46
F_{de2v}	0.21	2.75	0.56
F_{de3v}	0.07	2.75	0.19
F_{de4v}	0.09	2.75	0.24
F_{de5v}	0.01	2.75	0.03

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v, kN)	Total Horizontal Force (ΣF_h, kN)	Total Resisting Moment (ΣM_r, kN-m)	Total Overturning Moment (ΣM_o, kN-m)
227.25	92.20	246.46	179.44

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.23 >1.20 OK

$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.37 >1.30 OK

Figure B.16: QSAP stability calculation for block-2 (Knapsack Placed)

Block 3 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	226.38	-0.75	-169.79
P_3	85.07	1.61	137.13
F_{s3v}	0.53	3.80	2.00
F_{e6v}	1.78	3.80	6.75
F_{e7v}	0.29	3.80	1.10
T_{fill}	90.90	2.90	263.61

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	74.22	1.50	111.33
F_{s3h}	4.51	0.75	3.39
F_{e6h}	15.20	0.75	11.40
F_{e7h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-2	92.20	1.50	138.30
P_{3d}	9.95	0.77	7.68
F_{ds3h}	1.39	0.75	1.04
F_{de6h}	2.74	0.75	2.05
F_{de7h}	0.77	0.50	0.38
T_{filld}	15.44	0.75	11.58
F_{dw}	3.57	0.67	2.38
F_{ds3v}	0.16	3.80	0.62
F_{de6v}	0.55	3.80	2.07
F_{de7v}	0.09	3.80	0.34

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v, kN)	Total Horizontal Force (ΣF_h, kN)	Total Resisting Moment (ΣM_r, kN-m)	Total Overturning Moment (ΣM_o, kN-m)
406.60	145.30	490.30	359.84

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.40 >1.20 OK

$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.36 >1.30 OK

Figure B.17: QSAP stability calculation for block-3 (Knapsack Placed)

Block 4 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	404.95	-0.75	-303.71
P_4	102.62	2.06	211.72
F_{s4v}	0.53	4.70	2.48
F_{e8v}	2.36	4.70	11.07
F_{e9v}	0.29	4.70	1.36
T_{fill}	110.55	3.88	428.38

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	96.42	1.50	144.63
F_{s4h}	4.51	0.75	3.39
F_{e8h}	20.16	0.75	15.12
F_{e9h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-3	145.30	1.50	217.95
P_{4d}	12.01	0.77	9.22
F_{ds4h}	1.39	0.75	1.04
F_{de8h}	4.27	0.75	3.21
F_{de9h}	0.77	0.50	0.38
T_{filld}	10.54	0.75	7.91
F_{dw}	5.52	0.72	3.95
F_{ds4v}	0.16	4.70	0.76
F_{de8v}	0.72	4.70	3.40
F_{de9v}	0.09	4.70	0.42

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
623.92	204.01	846.19	624.77

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.53 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.35 >1.30 OK**

Figure B.18: QSAP stability calculation for block-4 (Knapsack Placed)

Block 5 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	621.29	-0.75	-465.97
P_5	119.19	2.49	296.67
F_{s5v}	0.53	5.55	2.93
F_{e10v}	2.94	5.55	16.30
F_{e11v}	0.29	5.55	1.61
T_{fill}	133.60	4.75	634.60

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	123.58	1.50	185.37
F_{s5h}	4.51	0.75	3.39
F_{e10h}	25.13	0.75	18.85
F_{e11h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-4	204.01	1.50	306.01
P_{5d}	13.95	0.77	10.67
F_{ds5h}	1.39	0.75	1.04
F_{de10h}	5.81	0.75	4.36
F_{de11h}	0.77	0.50	0.38
T_{filld}	5.79	0.75	4.34
F_{dw}	6.90	0.73	5.02
F_{ds5v}	0.16	5.55	0.90
F_{de10v}	0.90	5.55	5.00
F_{de11v}	0.09	5.55	0.49

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
881.62	267.79	1338.72	982.19

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.65 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 1.36 >1.30 OK**

Figure B.19: QSAP stability calculation for block-5 (Knapsack Placed)

Block 6 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	877.84	0.00	0.00
P_6	142.59	3.66	521.93
F_{s6v}	0.53	7.50	3.96
F_{e12v}	3.52	7.50	26.38
F_{e13v}	0.29	7.50	2.18
T_{fill}	195.00	6.53	1272.38

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	155.71	1.50	233.56
F_{s6h}	4.51	0.75	3.39
F_{e12h}	30.10	0.75	22.57
F_{e13h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-5	267.79	1.50	401.68
P_{6d}	16.68	0.76	12.73
F_{ds6h}	1.39	0.75	1.04
F_{de12h}	7.34	0.75	5.51
F_{de13h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	8.04	0.73	5.90
F_{ds6v}	0.16	7.50	1.21
F_{de12v}	1.08	7.50	8.10
F_{de13v}	0.09	7.50	0.67

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1224.88	336.15	3175.51	1439.31

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= 1.82 >1.20 OK **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)= 2.21 >1.30 OK**

Figure B.20: QSAP stability calculation for block-6 (Knapsack Placed)

Block 7 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	1219.76	0.00	0.00
P_7	127.97	3.29	420.47
F_{s7v}	0.53	6.75	3.56
F_{e14v}	4.10	6.75	27.65
F_{e15v}	0.29	6.75	1.96
T_{fill}	0.00	6.75	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	192.80	1.50	289.20
F_{s7h}	4.51	0.75	3.39
F_{e14h}	35.06	0.75	26.30
F_{e15h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-6	336.15	1.50	504.23
P_{7d}	14.97	0.76	11.44
F_{ds7h}	1.39	0.75	1.04
F_{de14h}	8.88	0.75	6.66
F_{de15h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	9.04	0.74	6.66
F_{ds7v}	0.16	6.75	1.09
F_{de14v}	1.26	6.75	8.49
F_{de15v}	0.09	6.75	0.60

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1359.27	410.30	3639.34	2003.90

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 1.66 **>1.20 OK**   **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.82 **>1.30 OK**

Figure B.21: QSAP stability calculation for block-7 (Knapsack Placed)

Block 8 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-7	1352.65	0.75	1014.49
P_8	124.31	3.38	419.55
F_{s8v}	0.53	6.75	3.56
F_{e16v}	4.68	6.75	31.57
F_{e17v}	0.29	6.75	1.96
T_{fill}	0.00	6.75	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-7	234.86	1.50	352.28
F_{s8h}	4.51	0.75	3.39
F_{e16h}	40.03	0.75	30.02
F_{e17h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-7	410.30	1.50	615.45
P_{8d}	14.55	0.75	10.91
F_{ds8h}	1.39	0.75	1.04
F_{de16h}	10.41	0.75	7.81
F_{de17h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	9.94	0.74	7.34
F_{ds8v}	0.16	6.75	1.09
F_{de16v}	1.44	6.75	9.69
F_{de17v}	0.09	6.75	0.60

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1490.77	491.42	5121.85	2685.33

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 1.52 **>1.20 OK**   **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.91 **>1.30 OK**

Figure B.22: QSAP stability calculation for block-8 (Knapsack Placed)

Block 9 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-8	1482.45	0.75	1111.84
P_9	124.31	3.38	419.55
F_{s9v}	0.53	6.75	3.56
F_{e18v}	5.26	6.75	35.49
F_{e19v}	0.29	6.75	1.96
T_{fill}	0.00	6.75	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-8	281.88	1.50	422.82
F_{s9h}	4.51	0.75	3.39
F_{e18h}	44.99	0.75	33.74
F_{e19h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-8	491.42	1.50	737.13
P_{9d}	14.55	0.75	10.91
F_{ds9h}	1.39	0.75	1.04
F_{de18h}	11.95	0.75	8.96
F_{de19h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	10.76	0.74	7.97
F_{ds9v}	0.16	6.75	1.09
F_{de18v}	1.61	6.75	10.89
F_{de19v}	0.09	6.75	0.60

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1623.02	579.86	6706.84	3494.50

$FS_{sliding}$ ($\mu*\Sigma F_v/\Sigma F_h$)= 1.40 **>1.20 OK**   **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** 1.92 **>1.30 OK**

Figure B.23: QSAP stability calculation for block-9 (Knapsack Placed)

Block 10 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-9	1612.84	0.75	1209.63
P_{10}	124.31	3.38	419.55
F_{s10v}	0.53	6.75	3.56
F_{e20v}	5.84	6.75	39.41
F_{e21v}	0.29	6.75	1.96
T_{fill}	0.00	6.75	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block 1-9	333.87	1.50	500.80
F_{s10h}	4.51	0.75	3.39
F_{e20h}	49.96	0.75	37.47
F_{e21h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block 1-9	579.86	1.50	869.79
P_{10d}	14.55	0.75	10.91
F_{ds10h}	1.39	0.75	1.04
F_{de20h}	13.48	0.75	10.11
F_{de21h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	11.52	0.74	8.55
F_{ds10v}	0.16	6.75	1.09
F_{de20v}	1.79	6.75	12.09
F_{de21v}	0.09	6.75	0.60

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1756.03	675.56	8394.74	4442.36

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= **1.30** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** **1.89** **>1.30 OK**

Figure B.24: QSAP stability calculation for block-10 (Knapsack Placed)

Block 11 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-10	1743.81	0.75	1307.86
P_{11}	124.31	3.38	419.55
F_{s11v}	0.53	6.75	3.56
F_{e22v}	6.42	6.75	43.32
F_{e23v}	0.29	6.75	1.96
T_{fill}	0.00	6.75	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-10	390.82	1.50	586.24
F_{s11h}	4.51	0.75	3.39
F_{e22h}	54.92	0.75	41.19
F_{e23h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block1-10	675.56	1.50	1013.34
P_{11d}	14.55	0.75	10.91
F_{ds11h}	1.39	0.75	1.04
F_{de22h}	15.02	0.75	11.26
F_{de23h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	12.23	0.74	9.09
F_{ds11v}	0.16	6.75	1.09
F_{de22v}	1.97	6.75	13.30
F_{de23v}	0.09	6.75	0.60

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
1889.80	778.48	10185.99	5539.76

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= **1.21** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** **1.84** **>1.30 OK**

Figure B.25: QSAP stability calculation for block-11 (Knapsack Placed)

Block 12 Stability Calculation:

Vertical Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-11	1875.36	1.25	2344.20
P_{12}	134.06	3.63	485.98
F_{s12v}	0.53	7.25	3.82
F_{e24v}	7.00	7.25	50.74
F_{e25v}	0.29	7.25	2.10
T_{fill}	0.00	7.25	0.00

Horizontal Forces (kN/m)		M. Arm (m)	Moment (kNm/m)
Block1-11	452.74	1.50	679.12
F_{s12h}	4.51	0.75	3.39
F_{e24h}	59.89	0.75	44.92
F_{e25h}	2.48	0.50	1.24

Pseudostatic Forces (ton/m)		M. Arm (m)	Moment (kNm/m)
Block1-11	778.48	1.50	1167.72
P_{12d}	15.69	0.75	11.76
F_{ds12h}	1.39	0.75	1.04
F_{de24h}	16.55	0.75	12.41
F_{de25h}	0.77	0.50	0.38
T_{filld}	0.00	0.00	0.00
F_{dw}	8.53	1.00	8.51
F_{ds12v}	0.16	7.25	1.17
F_{de24v}	2.15	7.25	15.57
F_{de25v}	0.09	7.25	0.65

ii) Pseudostatic Solution:

Total Vertical Force (ΣF_v , kN)	Total Horizontal Force (ΣF_h , kN)	Total Resisting Moment (ΣM_r , kN-m)	Total Overturning Moment (ΣM_o , kN-m)
2034.08	885.34	13090.22	6797.27

$FS_{sliding}$ ($\mu^*\Sigma F_v/\Sigma F_h$)= **1.38** **>1.20 OK** **$FS_{overturning}$ ($\Sigma M_r/\Sigma M_o$)=** **1.93** **>1.30 OK**

Figure B.26: QSAP stability calculation for block-12 (Knapsack Placed)