HONG KONG SEAWALL DESIGN STUDY

GEO REPORT No. 30

P.M. Aas & A. Engen

GEOTECHNICAL ENGINEERING OFFICE

CIVIL ENGINEERING DEPARTMENT

THE GOVERNMENT OF THE HONG KONG SPECIAL ADMINISTRATIVE REGION

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This report was originally produced in March 1993
under Consultancy Agreement CE 46/91

First published, September 1993
Reprinted, April 1995
Reprinted, August 2000

Prepared by:

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This publication is available from:

Government Publications Centre,
Ground Floor, Low Block,
Queensway Government Offices,
66 Queensway,
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Overseas orders should be placed with:

Publications Sales Section,
Information Services Department,
Room 402, 4th Floor, Murray Building,
Garden Road, Central,
Hong Kong.

Price in Hong Kong: HK$68
Price overseas: US$11 (including surface postage)

An additional bank charge of **HK$50** or **US$6.50** is required per cheque made in currencies other than Hong Kong dollars.

Cheques, bank drafts or money orders must be made payable to
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PREFACE

In keeping with our policy of releasing information of general technical interest, we make available some of our internal reports in a series of publications termed the GEO Report series. The reports in this series, of which this is one, are selected from a wide range of reports produced by the staff of the Office and our consultants.

Copies of GEO Reports have previously been made available free of charge in limited numbers. The demand for the reports in this series has increased greatly, necessitating new arrangements for supply. In future a charge will be made to cover the cost of printing.

The Geotechnical Engineering Office also publishes guidance documents and presents the results of research work of general interest in GEO Publications. These publications and the GEO Reports are disseminated through the Government's Information Services Department. Information on how to purchase them is given on the last page of this report.

A. W. Malone
Principal Government Geotechnical Engineer
April 1995

FOREWORD

In view of limited capacity for disposal of dredged spoil in Hong Kong waters, and in particular the difficulty and cost of safe disposal of contaminated spoil, the Geotechnical Engineering Office (GEO) of the Civil Engineering Department has been examining construction techniques that require less dredging. In 1992, the GEO commissioned a study to identify, evaluate and recommend design concepts for seawall construction without dredging soft mud, and to compare costs and practical aspects of such designs against the traditional dredged designs in common use in Hong Kong.

The Study was carried out by Messrs P.M. Aas and A. Engen under the supervision of Mr K. Karlsrud, of the Norwegian Geotechnical Institute, as Consultants to the GEO. It forms part of the GEO research programme on marine geotechnology.

JB Massey

(J.B. Massey)
Government Geotechnical Engineer/Development
September 1993

CONTENTS

SUMMARY

In the Hong Kong harbour areas there are soft marine mud and clay in the top soil strata. The current practice in Hong Kong is to install both vertical seawalls and sloping seawalls on a prepared seabed by dredging the marine mud and backfilling with proper foundation material. The aim of the work herein has been to identify, evaluate and recommend alternative designs for vertical and sloping seawalls, which do not require mud dredging, and to compare the costs of the alternatives to current practice.

The work has consisted of the following main tasks :

- Relatively rough design calculations to establish an acceptable geometry of the alternative structures.
- Assessing the necessity or benefit of ground strength improvement for all alternatives.
- Outlining practical construction methods, construction scheduling and other technical aspects.
- Estimating construction costs.

The report is organized into three parts. The main conclusions are presented on the next pages with accompanying figures. These conclusions all refer to what has been defined as a "base case", that is a water depth of 10 m, and 10 m of soft mud above 10 m of alluvial clay and underlying sand. Thereafter background material is presented in some more detail. Appendices with calculation details are attached at the end. On the figures, all levels are given in metres, while all construction measures are given in millimetres.

For cost comparison of the various alternatives it is necessary to define a geometrical cost limit. A vertical limit line at 55 m behind the front is chosen as this covers almost all cases. The 55 m is to be measured from the top of front of quay, for vertical seawall, and tip of slope for sloping seawall. Expected settlement of the existing ground has been included in the quantity assessment.

The main purpose of the cost estimates is to give comparative costs of the various solutions and not necessarily the final cost. This, in any case, has to be based upon a tender. The cost estimates have been checked against typical local unit costs received from Gammon H.K.

The cost items, as presented in the summary sheets, cover the following:

General cost	:	Mob/demob + site cost
Concrete work	:	All concrete related works
Foundation work	:	Piling, beds for caissons, geotextiles, dredging/backfilling
Filling work	:	Sand in and behind caisson, rock berm above sea floor, bearing course and paving
Drains	:	Band drains
Soil improvement	:	Cement stabilization. As described in Appendix C, the Deep Cement Mixing Method (DCM-method) is recommended for soil improvement.

As an overall conclusion, it is found that a method where about 25% of the soft marine mud and underlying alluvium clay are improved by cement stabilization gave the same cost as the current practice where the mud is dredged and replaced by frictional material. This conclusion is valid for both vertical and sloping seawalls. Further, a piled quay solution is found to be about 40% more expensive than a caisson on improved soil.

The most cost effective solution for a sloping seawall is placing a counterfill in front of the wall. This solution may however have some negative implications, as outlined in the report.

The different alternatives for vertical and sloping seawalls are discussed further on the following pages.

VERTICAL SEAWALL, BASE CASE

Current Practice, Caisson or Block Wall on a Dredged Foundation

Figures 1 and 2 show sketches of the geometry and key elements of the solutions, a brief description and cost figures.

For the base case standard vertical seawall in a dredged trench, a concrete caisson type as well as a block-wall type are investigated.

The soft marine mud will be removed by dredging (approx. 600 m^3/m of seawall has been assumed) and band drains are then installed in the underlying alluvium clay.

A non-woven geotextile is assumed placed in the dredged trench before backfilling with proper frictional material. On top of this a foundation bed of quarry-run rock is placed and levelled, forming a foundation for caissons or concrete blocks. The filter fabric is regarded as preferable both from a construction point of view and for better cost comparison.

With the specified mud strength properties this alternative may be susceptible to instability of the soft mud during dredging/backfilling operation.

The indicated prices for the individual blocks in the blockwall given by GEO (1992c), have been increased with 17% to compensate for price increase from 1990 to 1991.

A general evaluation of the price level of the concrete blocks indicates in our view that these are on the low side, lower than the unit prices for the other items in report. We do think that it might be difficult to obtain such prices in the market today.

The general evaluation indicates that there will not be any difference in construction time between concrete blockwall on rockfill foundation and concrete caisson on replaced fill. Therefore the same construction time was assumed for the two alternatives.

As seen from Figures 1 and 2 both alternatives are equally attractive with respect to economy. It should be noted, however, that the caisson solution can be built to cover a range of water depths at the quay front, whereas the present block wall type has a fixed water depth of about 5 m at the front. The caisson solution is therefore, more flexible to required water or sailing depths.

Effects of parameter variations on the cost figures are given in Chapter 4.

Caisson on the Sea Floor with Ground Strength Improvement Underneath and Vertical Drains Under Fill Behind

Figure 3 shows a sketch of the geometry and key elements of the solution, a brief description and cost figures.

The soft soils underneath the caisson are assumed to be improved by the deep cement mixing method. It is foreseen to establish cement stabilized wall panels (DCM panels) perpendicular to the seawall down to the sand layer. Approximately 25 % of the soft marine mud and alluvium clay is assumed to be improved.

On top of the walls, a geotextile will be placed before forming the quarry-run rock foundation for the caisson.

Behind the DCM improved zone, vertical band drains in a grid 1.5 m by 1.5 m are assumed to achieve consolidation and sufficient strength under the fill, and to increase the rate of consolidation settlements.

As seen on Figure 3, the alternative is economically comparable to the current practice (Figures 1 and 2).

Effects of parameter variations on the cost figures are given in Chapter 4.

Piled Quay in front of Primary Fill on Strength Improved Subsoil, and Vertical Drains Under Fill Behind

Figure 4 shows a sketch of the geometry and key elements of the solution, a brief description and cost figures.

A stable sloping wall of rockfill on a central cement stabilized foundation soil is formed. The rockfill berm has to be free from excessive settlement as the quay structure is susceptible to such settlements.

Band drains of c/c 1.5 m are assumed under the fill placed behind the central part to achieve sufficient strength and stability.

The piled quay is a standard structure founded on steel tube piles driven down to bed rock or hard layers.

In Figure 4 a transitional structure behind the piled platform will allow for differential settlements. This structure consists of an inclined hinged slab and horizontal slab buried in the ground.

This alternative is considerably more expensive than the current practice, but it has one big advantage: a sloping seawall may be transferred to a proper vertical seawall at a later stage by installing the piled quay.

Effects of parameter variations on the cost figures are given in Chapter 4.

SLOPING SEAWALL, BASE CASE

Current Practice, Fill on a Dredged Foundation

Figure 5 shows the geometry of the standard solution with descriptions and cost figures.

To form a stable foundation for the sloping wall, the soft mud will be removed by dredging down to the alluvium clay and replaced by good quality friction material i.e. sand. A non-woven geotextile will be placed in the trench before backfilling starts.

The sloping seawall has a standard quarry-run rockfill core with blocks in front as erosion protection.

Effects of parameter variations on the cost figures are given in Chapter 4.

Fill on Strength Improved Subsoil and Consolidation by Means of Drains

Figure 6 shows the geometry of the alternative with descriptions and cost figures.

To form a stable foundation for the sloping wall, 25% of the soft mud and alluvium clay will have DCM stabilized wall panels down to the sand layer.

A woven synthetic geotextile has to be placed on top of the cement stabilized walls to form a continuous foundation for the sloping wall.

Band drains at c/c 1.5 m are required under the fill behind the central core.

The cost of this alternative is practically the same as for the "Current practice" case, ref. Figures 5 and 6.

Effects of parameter variations on the cost figures are given in Chapter 4.

Fill with a Gently Sloping Counterfill in Front

Figures 7 shows the geometry of the alternative with description and cost figures.

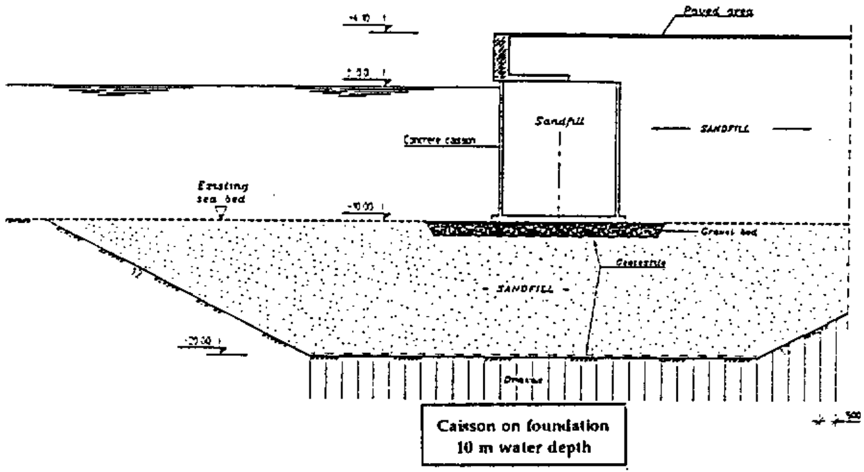
The strength improved zone at the toe of the counterfill should be established before any filling works. Band drains are then installed underneath the "primary fill" and extending 10 m out to each side from the toe. A geotextile is installed underneath the entire counterfill.

The "counterfill" and the "primary fill" up to the same height as the "counterfill" is then installed and left to settle for about one year. Then the "primary fill" is installed to its full height. (It should be noted that the terms "primary fill" and "counterfill" are only used to separate the two parts of the total structure in the text).

As seen from Figure 7, the cost of this alternative is about 16% less than the two other sloping seawall alternatives considered above. The "counterfill" implies, however, very small water depths which may be a hazard to ship traffic.

Effects of parameter variations on the cost figures are given in Chapter 4.

LIST OF FIGURES



Comments:			
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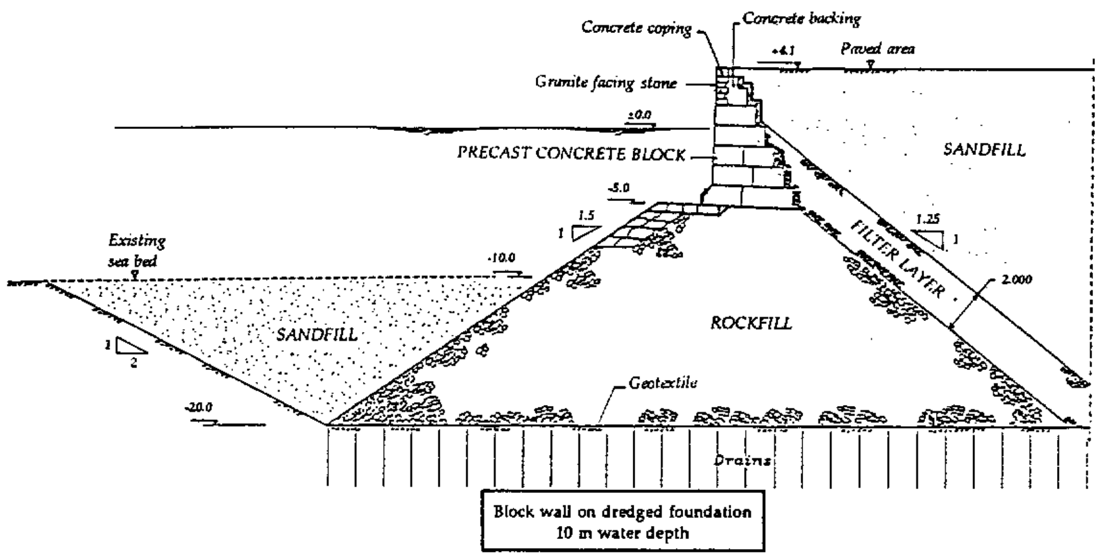
	Concrete works:	"	52,000,000
	Foundations works:	"	67,000,000
	Filling works:	"	56,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	230,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 1 - Cost Figure for Vertical Seawall : Standard Alternative,
Caisson on Dredged Foundation, Base Case



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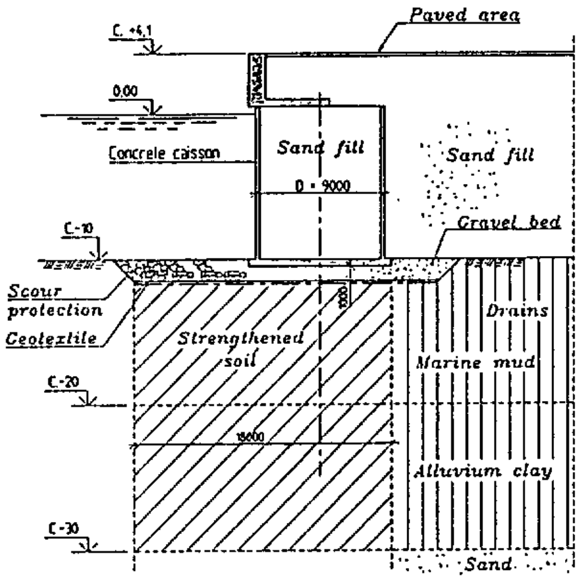
	Filling works:	"	74,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	230,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 2 - Cost Figure for Vertical Seawall : Blockwall on Dredged Foundation, 10 m Water Depth

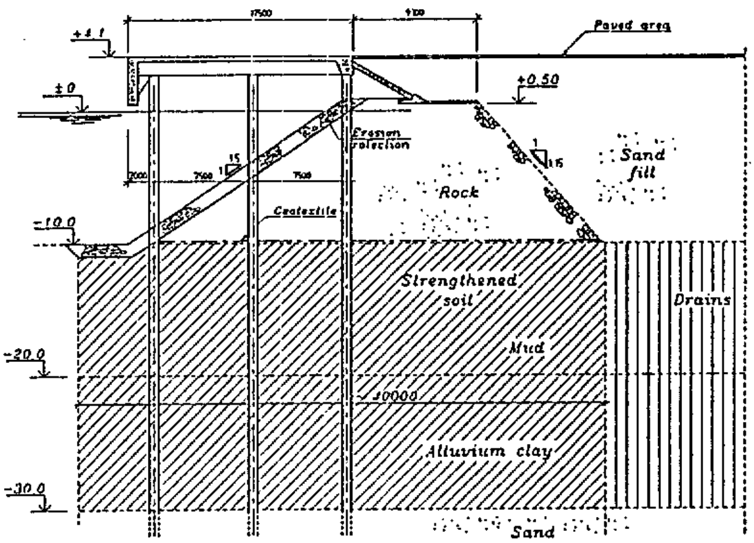


Comments:	General cost:	HK$	41,000,000
Cost effective, technically flexible, well known technology, not sensitive to variation in soil data and layering	Concrete works:	"	52,000,000
	Foundations works:	"	12,000,000
	Filling works:	"	56,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	54,000,000
	SUM: 1000 m	HK$	229,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 3 - Cost Figure for Vertical Seawall : Caisson on Sea Floor, Ground Strength Improvement and Consolidation by Means of Drains, Base Case

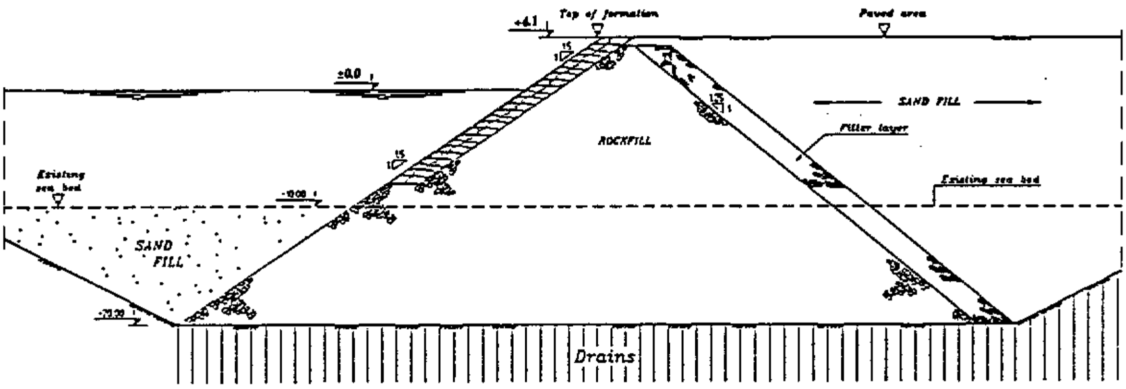


Comments:		
Technically flexible, well known technology, quay front free of settlements, Costs sensitive to water depth and depth to bed rock	General cost: HK$	36,000,000
	Concrete works: "	35,000,000
	Foundations works: "	48,000,000
	Filling works: "	61,000,000
	Drains: "	5,000,000
	Ground improvements: "	120,000,000
	SUM: 1000 m HK$	305,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4 - Cost Figure for Vertical Seawall : Fill on Strength Improved Subsoil and Consolidation by Means of Drains, Subsequently Installed Piled Quay, Base Case

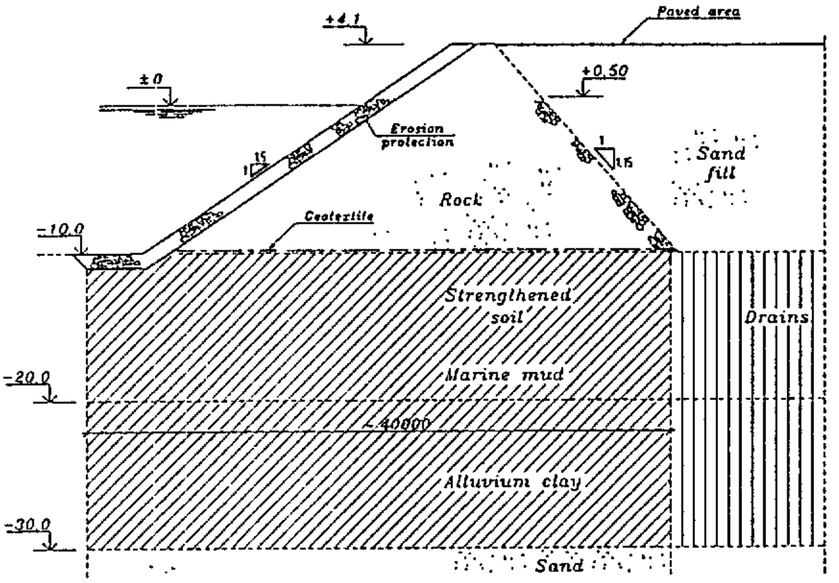


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	18,000,000
	Filling works:	"	154,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	222,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1		2		3		4	
Ground improvements								
Foundation works								
Concrete works								
Filling works								

Figure 5 - Cost Figure for Sloping Seawall : Standard Alternative, Fill on Dredged Foundation, Base Case

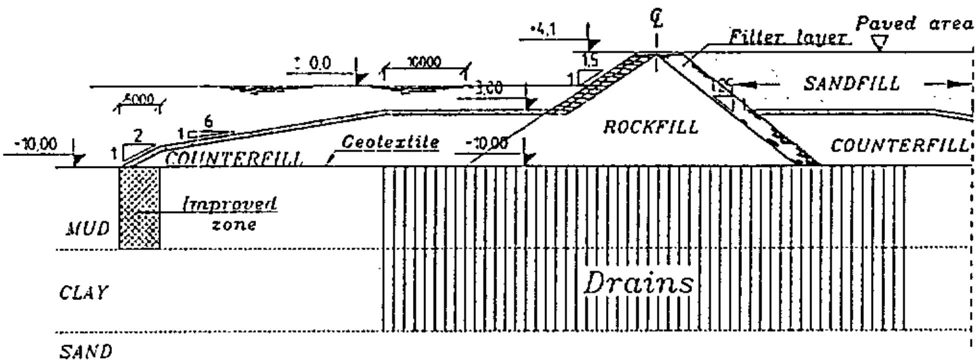


Comments:			
Cost effective, technically flexible, well known technology, sea wall free of settlements	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	-
	Filling works:	"	61,000,000
	Drains:	"	5,000,000
	Ground improvements:	"	118,000,000
	SUM: 1000 m	HK$	220,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 6 - Cost Figure for Sloping Seawall : Fill on Strength Improved Subsoil and Consolidation by Means of Drains, Base Case



COUNTER FILL CONCEPT
WATER DEPTH = 10 m

Comments:			
Technically flexible, cost effective, relatively long construction phase	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	5,000,000
	Filling works:	"	103,000,000
	Drains:	"	26,000,000
	Ground improvements:	"	15,000,000
	SUM: 1000 m	HK$	185,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 7 - Cost Figure for Sloping Seawall : Fill with a Counterfill of Very Gentle Slope in Front, Base Case

1. INTRODUCTION

The Norwegian Geotechnical Institute, NGI, has been contracted by the Geotechnical Engineering Office, GEO, of Hong Kong's Civil Engineering Department in order to undertake a study of alternative designs of vertical and sloping seawalls. The background for this study is that the proposed infrastructure developments in Hong Kong will involve construction of considerable lengths of seawalls and breakwaters. In the past, the practice in Hong Kong has been to found such structures on firm strata by removing the soft, marine mud and replacing it by fill material. Efforts are now being made to restrict the future dredging of these muds in order to reduce the amount of spoil that will have to be disposed of. The Government of Hong Kong is examining the possibilities for alternative seawall designs that do not involve mud removal and the economy of such construction alternatives.

The study has concentrated on one set of standard ground conditions, which has been taken as representative for typical ground conditions at potential seawall sites in Hong Kong. Parametric studies have been carried out to assess the sensitivity to cost and construction aspects of variations in water depth, thickness of the marine mud and the strength of the alluvium clay below the mud, varying only one of the parameters at a time.

The study has identified, evaluated and recommended design concepts for seawall structures without dredging soft mud. In the course of the work some designs have been ruled out, either due to technical reasons, cost aspects or both. These concepts have not been included in the main summary above, but they are included in the list of concepts evaluated together with the reasons why they are eliminated.

Cost evaluation and construction phasing have mainly been subcontracted to the largest civil contracting company in Norway, Selmer A/S. Gammon Ltd. of Hong Kong has been subcontracted for the purpose of reviewing the cost figures for a calibration to the local cost level in Hong Kong.

2. DESIGN CRITERIA

2.1 WATER DEPTH

The base case water depth is 10 m. In addition, a variation from 5 to 15 m has been included in the study.

2.2 WAVE LOADS

Wave load impact on a vertical wall is calculated on the basis of criteria by British Standard Code of Practice for Marine Structures, Part 1, General criteria, BSI (1984). Details of the calculations are given in Appendix B.

Wave pressure amplitudes on a sloping wall are roughly taken as 70% of the impact on a vertical wall, NHL (1992).

2.3 SOIL PROFILE

Soil design parameters are given in Appendix A, with a summary in Table 2.1.

The thickness of the marine mud, and the strength of the mud and the alluvium clay below, may have major impact on the design. As base case assumption the mud thickness is 10 m. Effects of variations in mud thickness from 5 to 15 m and undrained shear strength of the alluvium clay from 20 to 60 kPa, have been evaluated.

For several alternatives, band drains are assumed installed through the mud and alluvium clay in order to achieve consolidation and some gain in strength underneath the fill during construction. On the basis of test fills in connection with the Chek Lap Kok airport in Hong Kong, referred to in Appendix C, it has been assumed that 100% consolidation is achieved within 2 years from the commencement of filling for band drains at c/c 1.5 m. After 100% consolidation is reached, the undrained shear strength of the marine mud and alluvium clay can be taken as
$s_u = 0.28\sigma_v'$, where σ_v' is the effective vertical stress at the given depth.

As concluded in Section 3.2, the DCM improved panels will have an undrained shear strength of at least 800 kPa. With 25% of the soil improved, the average strength in the improved zones is conservatively taken as 200 kPa.

The friction angle of the sand fill and gravel bed is taken as 37 deg., and the total unit weight as 20 kN/m^3.

2.4 SHIP IMPACT LOAD

With the given ship sizes the displacement is less than 200 t. Therefore the impact loads are negligible, ref. NGI (1992).

2.5 CURRENTS

Protection against potential erosion due to currents from various sources, such as waves, tide and ships have been accounted for in the proposed designs.

2.6 LIVE LOADS

With the non-specified use and function of the reclaimed area, live loads of 10 kPa has been implemented in the design calculations on the basis of suggestion by GEO (1992a).

This assumption may be good enough for comparison of the different concepts. However, it is emphasized that the design concepts presented are not directly applicable in case later functional requirements specify higher live loads.

2.7 SAFETY FACTOR

For foundation stability a lump sum safety factor of 1.2 has been suggested by GEO (1992a).

A lump sum safety factor is to be understood as a total safety factor including both the uncertainty in the soil parameters and in the loads, in contradiction to a safety margin expressed in terms of partial coefficients.

2.8 LENGTH OF WALL

The length of wall to be constructed may be important for the cost aspects. A wall length 0.5-1 km has been assumed based on suggestion by GEO (1992b).

3. MAIN DESIGN ASPECTS

3.1 GENERAL

The major design challenge has been to design structures that can withstand the fill weight, earth pressures from the fill and wave load impact (and a small live load), on top of a soft, marine mud.

The main principles of the stability assessments are summarized in this section.

Measures to improve the strength of the top soft strata have been found necessary for most of the concepts, and this pretreatment is an important element of the concepts. Different alternatives for improvement have been considered, as described below.

3.2 GROUND IMPROVEMENT MEASURES

3.2.1 DEEP CEMENT MIXING METHODS

The following alternatives have been considered:

1) Method based on in situ mixing of clay with *cement slurry*. To our knowledge this method has only been developed and used in Japan. (The method is called DCM-method, Deep Cement Mixing method, or Deep Chemical Mixing method).
2) Method based on in situ mixing of clay with dry cement, or lime powder. The method was first developed and used in Sweden, then Finland and Norway. (It was originally called "Lime Column Method", but lime has to large extent been replaced with cement or cement/lime mix). The method and equipment are also developed in Japan, and more recently in Italy.
3) Jet-grouting method. The method is based on using water jets to partly remove and partly mix in situ soil with cement-based slurry. The method was originated in Japan, but has later been adopted in many countries.

As to choice and applicability of these methods, it is a question of what strength can be obtained of treated material, costs and rate of production.

Method 3) is ruled out because it appears to be much more costly than the others. The production rate also seems (much) lower. Furthermore, there are no experience with applications at sea. Problems are also anticipated with treating the upper few meters of very soft marine mud.

Only Japanese companies have to our knowledge the equipment for carrying out treatment according to Methods 1) and 2) at sea to the depths required. At least 50 different companies are members of the Cement Deep Mixing Method Association, and about 20 mill. m^3 of soil has been improved up until 1992.

There are some conflicting data regarding which method gives the highest potential improved shear strength in the type of soils in question. Method 1) seems best documented with respect to similar applications at sea. At the present stage we have therefore based our evaluations on Method 1). Appendix C summarizes a brief description of the method and equipment, strengths obtained and cost figures. Based on these results we have assumed somewhat conservatively that an undrained shear strength of 800 kPa can be obtained.

Two Japanese companies (Takenaka Civil Eng. & Constr. Co. and Shimizu Corporation) have been contacted for cost of improvement. They suggest about HK$ 590 per m^3 of treated soil.

The production rate is suggested to be 1230 m^3/day assuming 16 hour working days and fairly good operational weather conditions. Wave- and wind restrictions are not completely clarified, but a characteristic wave height of 0.6 m has been indicated as a limit by Japanese contractors.

3.2.2 CONSOLIDATION BY USE OF VERTICAL DRAINS

This method to accelerate the consolidation rate of clay strata has successfully been used since 1937. The principles, materials used and applications are frequently described in literature. The most adequate references for the present study are Siu and Ko (1990) and Malone and Premchitt (1990). These papers describe the observations made in connection with a test fill near to the planned Chek Lap Kok airport in Hong Kong. The test fill was placed on a marine mud, with properties that form partly the data basis for the present study. Key excerpts are given in Appendix C.

The observations revealed that vertical band drains are more efficient than vertical sand drains.

With vertical drains of c/c distance 1.5 m 100% consolidation of the marine mud can be obtained within 2 years from the commencement of the fill. The alluvium clay below will consolidate faster, due to higher c_V values. 100% dissipation may be achieved within about 1 year in this layer for 1.5 m c/c drain spacing.

According to our experience, a cost of HK$ 35/m drain installed is realistic, and has been applied.

3.2.3 GEOTEXTILES

Geotextiles have been considered in combination with DCM strength improvement in the mud and clay layers. Two factors are governing: Firstly, geotextiles will prevent a coarse fill from sinking into the soft mud in between the strength improved vertical panels. Secondly, a geotextile can provide satisfactory transfer of loads from a fill to the DCM improved panels.

It should, however, be noted that in cases where geotextiles with high strengths are required, polyester is needed. The strength of the polyester may be affected by direct contact to cement stabilized columns, due to chemical interaction. This interaction can be avoided by a two-layer system, by use of polypropylene (lower strength, but not so sensitive to cement contact) closest to the stabilized columns and polyester above a bottom layer of fill material. This is not included in the cost figures, but such details will not affect the cost comparisons as the geotextile costs have marginal impact on the total costs.

Appendix C gives a short description of the method, applications and relationships between cost and strengths of the geotextiles.

For the strength required herein, a cost of HK$ 47-134 per m^2 installed is realistic.

3.2.4 LIGHT WEIGHT FILL

Use of light weight aggregates in the fill will reduce the load on the soft layers underneath, and can in principle contribute to an improved foundation stability. However, use of light weight fill will reduce the gain in strength achieved by consolidation and drains.

The effects on global stability from introducing a light weight fill have been investigated by a simplified approach assuming an average unit weight of the fill behind the seawall of 11 kN/m^3 without considering the practical details of layering, construction etc.

The effect of a light weight fill is in general found highly marginal for the cases where it has been investigated. For this reason this measure is generally not recommended.

3.3 PRINCIPLES OF STABILITY CALCULATIONS

3.3.1 VERTICAL SEAWALL

A principle sketch of a vertical seawall with a reclaimed fill behind is shown on Figure 3.1. The sketch shows a caisson penetrated through the marine mud and the alluvium clay. However, in principle the caisson can be replaced by a cellular cofferdam, a tied back sheet pile wall or other structures combined with ground improvement of the soft mud/clay layers.

To assess the stability of a structure with or without a zone of soil improvement to some depth, the resulting global forces acting at the base of the structure/improved zone are calculated. Bearing capacity and sliding at this level are then estimated as described below, and necessary width and depth of the structure/improved zone to obtain a lumped safety factor, SF, of 1.2 are established.

Figure 3.2 shows the earth pressures and water pressures acting on a structure installed in the marine mud and the underlying alluvium clay. The global resulting forces P_V, P_H and M acting at the base are found by adding the submerged weights and the active and passive earth pressures.

Sliding in the clay/improved zone is checked against the requirement:

$$P_H \leq \overline{s}_u \cdot b/SF; \quad \overline{s}_u = \text{average shear strength}$$

$$b = \text{zone width}$$

Sliding in the sand is checked by requiring:

$$P_H \leq q \cdot b_o \cdot \tan\rho; \; q = P_V/b_o$$

$$b_o = b - 2 \cdot M/P_V$$

$$\tan\rho = \tan\phi/SF; \; \phi = 40^\circ$$

Sliding in the gravel underneath a caisson resting on the sea floor is checked by requiring:

$$P_H \leq P_V \cdot \tan\rho \; ; \; \tan\rho = \tan\phi/SF \; , \; \phi = 40^\circ$$

Stability in the clay/improved zone is calculated as:

$$q \leq N_c \cdot \overline{s}_u/SF - \sqrt{2} \cdot P_H/b_o \; ;$$

N_c = bearing capacity factor, taken as 5.14 herein

Bearing capacity in the sand is calculated from:

$$q \leq N_q \cdot p_V' + \tfrac{1}{2} \cdot N_\gamma \cdot \gamma' \cdot b_o$$

The bearing capacity numbers N_q and N_γ are taken from Janbu et al. (1976).

3.3.2 SLOPING SEAWALL

Figure 3.3 shows a principle sketch of a sloping seawall with a reclaimed fill behind. Some measures are needed to stabilize the fill on top of the mud. The sketch shows an improved zone through the mud and the clay at the toe of the fill, but the improved zone can be replaced by a caisson, penetrating through the mud, into the clay or into the sand.

For the sloping seawall alternative, a piled quay can subsequently be installed, if required.

The slope stability of the rubble mound wall alternative is assessed by the use of a computer program BEAST for limit equilibrium analysis by the method of slices (Clausen, 1988). Required width of the improved zone or the caisson to obtain a lumped safety factor, SF, of 1.2 is established.

4. ASSESSMENTS OF ALTERNATIVE DESIGN CONCEPTS

4.1 CONCEPTS CONSIDERED, GENERAL ASPECTS

Figures 4.1 and 4.2 illustrate the main seawall design concepts that have been investigated in this study. This includes the following:

(a)	Vertical seawalls	-	Caisson into sand
		-	Caisson into alluvium clay
		-	Caisson resting on sea floor
		-	Pile quay in front of fill
		-	Cellular cofferdam
		-	Tied back sheetpile wall
		-	Current practice in Hong Kong
(b)	Sloping seawalls	-	Fill on improved subsoil
		-	Fill with counterberms
		-	Fill on relief piles
		-	Current practice in Hong Kong

For the concepts found of most interest, parametric studies to assess effects of changes in water depths, thickness of mud and strength of alluvium clay has been carried out. Special measures to improve the seawall stability have also been considered in some cases, ref. Section 3.2 above.

The following sections deal separately with each main design concept, and are organized under the following subheading :

Design analyses
Construction phasing
Cost aspects
Technical recommendations, limitations etc.

Figures 4.3 to 4.9 show the results of the sliding and bearing capacity calculations for various input parameters and concepts investigated. Results of stability calculations for one concept are applicable also for other concepts, even if the concepts as such, are different. This means that some of the figures are referred to for more than one concept.

4.2 VERTICAL SEAWALLS

4.2.1 CAISSON INTO SAND

(a) Design Analyses

- Stability

Figure 4.3 shows the results of the stability calculations for base case input parameters (10 m water depth, 10 m marine mud thickness and in situ undrained compression strength of the alluvium clay of 40 kPa). The stability calculations are illustrated by an example in Appendix D. This analysis is based on vertical drains behind the wall and full consolidation of the mud and the clay for the fill weight.

As can be seen from the Figure 4.3, bearing capacity in the sand layer, rather than horizontal sliding, is critical. A width of 18 m is necessary for the caisson.

In Figure 4.4, no drains and thus no consolidation of the mud and clay underneath the fill is assumed. In this case, the required width of the caisson is 28 m, and sliding is the critical failure mode.

In Figure 4.5, a light weight fill in combination with no drains is introduced behind the wall. As seen, the caisson must be about 15 m wide. Thus, the effect of light weight fill is highly marginal (15 m versus 18 m caisson width).

Figures 4.6 to 4.9 show the effects of variations in water depth and marine mud thickness. As shown by the figures, bearing capacity in the sand layer requires caisson width's ranging from about 14 m to 21 m to obtain a SF of 1.2. In all cases, drains are assumed installed behind the wall, as this is found cost effective.

Variation in the shear strength of the clay in front of the wall has a negligible effect on the geometries of the different concepts, and are thus disregarded. In addition, varying the in situ strength of the clay behind the wall from 20 to 60 kPa is not relevant, as consolidation under the fill weight, is considered to be beneficial. This consolidation will give a strength larger than 60 kPa.

- Penetration analyses

Detailed analysis of penetration resistance of the caissons for the base case input parameters are presented in Appendix D. It is concluded that the caisson will penetrate through the soft marine mud under selfweight. Penetration through the alluvium clay can be achieved by ballasting the caisson and applying underbase suction. Satisfactory penetration into the sand, say 1-2 m, in order to obtain proper interaction between the caisson and the sand layer may require special precautions in order to reduce the tip resistance, such as wedge shaped skirt tips, steel skirts into the sand, filters at skirt tip, jetting at skirt tip etc.

Variations in the mud layer thickness will change these conclusions marginally due to the small resistance through this layer. 15 m water depth will improve the skirt penetration due to more room for ballast and more potential suction. 5 m water depth may imply problems with penetration through the clay layer unless provision for extra ballast is made.

- Settlements

Appendix D gives a detailed description of computed consolidation settlements for base case input parameters. Accumulated consolidation settlements 50 years after the fill was installed is calculated to be 3.3 m.

With band drains installed in a grid of 1.5 m by 1.5 m, the primary consolidation settlements will be completed within 2 years after placing the fill.

5 m and 15 m water depth gave settlements of 2.8 m and 3.7 m, respectively. 5 m and 15 m marine mud thickness resulted in calculated consolidation settlements of 2.7 m and 3.7 m, respectively. A light weight fill is calculated to give consolidation settlements of the order 1.8 m.

The caisson itself resting on the sand layer will only settle about 20-30 cm within the first few years due to consolidation of the underlying strata, see Appendix D.

(b) Construction Phasing

The caisson is foreseen to be built on a semisubmersible barge. The barge should be moored alongside a quay during construction to facilitate the casting. The walls will be slipformed. Due to the great height, the slipforming will be susceptible to wind.

The completed caisson will be towed to the construction site on board the semisubmersible barge where it will be received by a moored barge to keep it stable during off-floating and the installation phase.

The progress schedule is based on a rate of approximately one caisson every fortnight.

The technology behind the penetration operation is well known, but the concept is relatively elaborate and involves many construction phases, such as air cushion during placement, suction and ballasting during penetration of the caissons.

The caissons must be guided during the installation due to instability.

The solution is susceptible to water depth variations. Depths less than 10 m may also cause problems due to draft requirements of the lifting vessels.

(c) Cost Evaluations

Cost figures, with reference to details in Appendix E, are given on Figures 4.10 - 4.12, for water depths 5, 10 and 15 meters, respectively.

For variations in mud thickness the caisson height varies in the same way as for the water depth variations. The stability calculations show that the caisson width may change marginally as compared to the cases with water depth variations. Consequently, with approximately the same caisson, contributing about 50% to the total cost, the effects of mud thickness variations will be close to those given above for water depth variations.

To facilitate the penetration of the caisson into the sand, it is suggested to substitute the lower few meters of the concrete wall with steel. This will increase costs by 5 - 10% as compared to the costs given in Figures 4.10 - 4.12.

(d) Technical Recommendation, Limitation etc.

The alternative is not recommended. There are uncertainties related to the penetration and special measures are required. The caisson is susceptible to variations in the depth to the sand layer. Finally, the cost is among the highest estimated.

4.2.2 CAISSON INTO ALLUVIUM CLAY

(a) Design Analyses

- Stability

For a caisson only penetrating through the top marine mud, drains or ground improvement by deep cement mixing are required in the underlying alluvium clay, depending on the geometry of the problem (water depth, mud thickness). Figure 4.3 shows that a caisson width of at least 40 m is required for base case input parameters, if the clay under the caisson is not improved other than by drains and consolidation. This solution is therefore rejected.

With the use of ground improvement placed in vertical wall panels at 4 m spacing, it is assumed an average strength of 200 kPa in the improved zone, ref. Appendix C. Figure 4.3 indicates a required caisson width of about 15 m to satisfy the bearing capacity criteria, and a width of the improved zone of about 18 m to prevent failure in the underlying sand (equivalent to width of caisson into sand).

Figure 4.4 shows that by omitting the drains underneath the fill behind the caisson, the width of the caisson must be increased to 20 m, and the width of the improved zone to 28 m. Further, Figure 4.5 shows that replacing the normal fill with light weight fill material gives dimensions as for the base case with drains.

As for the concept with a caisson into the sand layer, Figures 4.6 to 4.9 show the effects of varying water depth and marine mud thickness. A caisson width ranging from 10 m to 20 m is needed if the clay layer is DCM improved, with corresponding widths of the improved zone of 14 m to 21 m.

For the case where only drains are installed below the caisson, caisson widths of 20 m and 30 m will satisfy global stability criteria for 5 m water depth and 5 m mud thickness, respectively. Drains alone will not give feasible solutions when the water depth or mud thickness are 15 m.

- Penetration

Detailed skirt penetration calculations were carried out for the "Caisson into sand" concept. From these calculations it is found that moderate amounts of ballast is required to penetrate a caisson a few meters into the alluvium clay. The penetration will stop when the skirt tips reach the improved soil panels.

- Settlements

The primary consolidation settlements of the fill behind the caisson have already been calculated and presented in Section 4.2.1 and Appendix D.

Settlements of the caisson are calculated to 20-30 cm if resting on improved soil. No settlements of the improved zone are assumed. If drains are installed below the caisson, Appendix D (Section D2.2) shows that the caisson will settle about 1 m for base case input parameters. This consolidation settlement will occur within one or two years after installation.

- Structure/soil interaction

Ground strength improvement raises some questions: Normally the strength is improved in vertical panels some distance apart, herein taken as 4 m. The interaction between discrete panels and the continuous caisson walls can be questionable. Both the vertical bearing on discrete points underneath the skirt tips, and interaction under lateral loading from the fill behind and wave loads, need further evaluations. This interaction problem can be avoided by continuous strength improved panels below the cellular wall. Then, however, the cost aspect of the strength improvement becomes important, see below under Section 4.2.5 Cellular cofferdam.

(b) Construction Phasing

No special considerations have been made.

(c) Cost Evaluations

See Section 4.2.5.

(d) Technical Recommendation, Limitation etc.

On the basis of the technical objections above and the possible cost consequences, this solution is not found as attractive as "Caisson resting on sea floor", and has therefore been disregarded.

4.2.3 CAISSON RESTING ON SEA FLOOR

(a) Design Analyses

- Stability

For a caisson resting on the sea floor, the marine mud and alluvium clay below must be improved to ensure global stability for base case input parameters. Figure 4.3 shows that a caisson width of about 8 m will fulfil the bearing capacity requirements in the improved zone, and that this zone must be 18-20 m wide to obtain sufficient bearing capacity in the sand layer. However, a caisson width of 9 m is needed to avoid sliding in the gravel bed assumed placed in the transition between the caisson and the improved clay.

Omitting the drains beneath the fill will not influence the layout of the caisson, but the width of the improved zone must be increased by about 10 m to prevent sliding in the sand layer.

The introduction of a light weight fill will reduce the required width of the caisson to 6-7 m, and the width of the improved zone to 15 m, as shown in Figure 4.5. This case can also in theory be combined with soil improvement through the mud only, and drains in the clay below. The width of the improved zone should then be 35-40 m.

From Figure 4.6, it is seen that only a 5 m wide caisson is needed on the improved soil for 5 m water depth to avoid sliding in the gravel. The improved zone should be about 14 m wide if it extends down into the sand layer, and 20 m wide if the alluvium clay is drained but not strength improved.

Figure 4.7 gives the corresponding values for 15 m water depth. The caisson should be about 14 m wide, and the improved zone below should be 20-25 m wide. In this case, it is not feasible to only drain the alluvium clay.

A variation of marine mud thickness ranging from 5 m to 15 m will not influence the geometry of the caisson resting on the seabed. Figure 4.8 shows that for 5 m mud thickness, the width of the improved zone should be 15-20 m if it extends down into the sand and 30-35 m if it only includes the mud. For 15 m mud, Figure 4.9 shows that the improved zone should be about 20 m wide to ensure bearing capacity in the sand. A solution based only on drains is not feasible.

- Settlements

The primary consolidation settlements of the fill behind the caisson have already been calculated and presented in Section 4.2.1 and Appendix D.

Caisson settlements are estimated to 20-30 cm if resting on a DCM improved zone going through both the marine mud and the alluvium clay. If only the mud is improved and drains are installed in the clay, the caisson will settle about 1 m, ref. Appendix D, Section D2.2.

- Soil/structure interaction

Proper interaction between the bottom slab of the caisson and the strengthened panels underneath will require a gravel bed in between the caisson slab and the panels. Further, a geotextile is needed to prevent that the gravel bed sinks into the soft mud underneath. Evaluations of the required geotextile quality are made in Appendix D.

(b) Construction Phasing

The caisson is foreseen to be built on a fabrication yard and launched into the sea on a slipway. The completed caissons will be stored afloat, awaiting necessary preparation at the installation site. The caissons will be towed afloat to the installation site for positioning and ballasting. This alternative is based on well known principles and has great flexibility.

The progress schedule is based on a production rate of two caissons a week for the small ones and one caisson a week for the bigger ones.

Since the smaller caissons are about half the size of the bigger ones the progress will be practically unchanged for variations in water depth.

The construction of the caissons and superstructure involve relatively few operations and are well proven. The solution is flexible regarding production progress since practically all components can be prefabricated. The solution is also recommended for larger quay loads.

The amount of soil improvement and concrete volume do not increase much with increased water depth.

Except for the gravel bed, only sand fill is required.

(c) Cost Evaluations

Cost figures, with reference to details in Appendix E, are given on Figures 4.13 - 4.15, for water depths 5, 10 and 15 m, respectively.

Mud thickness variations have not been considered, for the same reasons given in Section 4.2.1. However, the soil improvement cost given in Figures 4.13 to 4.15 should be corrected by 0.75 (for 5 m thickness) and 1.25 (for 15 m thickness), simply due to the volumes involved. The concept is believed to be less sensitive to mud thickness variations than the standard alternative.

(d) Technical Recommendation, Limitation etc.

This alternative is highly recommended as it is cost effective, it has few limitations and it is flexible with respect to parameter variations.

4.2.4 FILL WITH SUBSEQUENTLY INSTALLED CONCRETE QUAY

(a) Design Analyses

- Stability

A slope of 1:1.5 at the front of the fill is assumed in the stability calculations. This is the steepest slope that it is realistic to assume, and giving the most favourable cost figures for this alternative.

For base case input parameters, a caisson or improved zone of about 5 m width and extending down into the sand layer is found to give sufficient global stability for all slip surfaces that go through the caisson/improved zone. The input/output from BEAST (1988) are given in Appendix D for base case input parameters. The critical slip surface goes through the improved zone.

However, the improved zone or caisson itself, resting on the sand layer is not stable if it is only 5 m wide. Satisfactory stability against tilting or sliding of the caisson/improved zone, requires about the same width as for the vertical seawall alternatives, i.e. 15-20 m.

A caisson or strength improved zone underneath each side of the "primary" fill is considered beneficial. On the basis of the cost figures in Sections 4.2.1 - 4.2.3, strength improvement is clearly more attractive than a caisson.

Since global stability calculations yield a required improved zone of 15-20 m width beneath both ends of the primary fill, it is recommended to improve the mud and clay across the entire width of the primary fill. With a preliminary slope at the back of the fill of 1:1.15, total width of the improved zone ranging from 35 m to 50 m is needed, depending on water depth.

The advantage of having a connection between the concrete quay and the reclaimed area behind nearly free of settlements also goes in favour of soil improvement underneath the entire primary fill.

- Interaction between fill and strengthened panels

In order to avoid that the fill material penetrates into the soft mud, a geotextile on top of the strength improved panels is assumed. Requirements to the geotextile are evaluated in Appendix D, Section D3.1.

(b) Construction Phasing

The rock fill berm with erosion protection is formed on top of the improved soil. This rock fill berm is in fact a sloping seawall and can be used as such.

The piles are driven through the rockfill berm and down to bed rock or hard layers. Only steel tube piles are considered to be feasible for this piling. The piles will be filled reinforced concrete to 10 meters below seafloor. The lower part is filled with sand.

Construction sequences
- Soil improvement/drains
- Rock berm
- Piling
- Superstructure
- Fill

To meet the construction schedule a progress per week of 2 bays at 7 m width for the superstructure is required.

The construction progress of the quay structure is largely independent on the water depth.

The quay line could be constructed whenever necessary after installation of the rock berm. This is considered to be of great importance.

This open concrete quay solution is a well proven solution. And large scale prefabrication is possible in order to increase construction progress as necessary.

The quay width and corresponding concrete volume and pile numbers are, however, strongly dependent on the water depth in front of the quay. Increased design loads on the quay will also require a corresponding increase in concrete volume and pile dimensions.

(c) Cost Evaluations

Cost figures, with reference to details in Appendix E, are given on Figures 4.16 - 4.18, for water depths 5,10 and 15 m, respectively.

Mud thickness variations have not been considered, for the same reasons given in Section 4.2.1. However, the soil improvement cost given in Figures 4.16 to 4.18 should be corrected by 0.75 (for 5 m thickness) and 1.25 (for 15 m thickness), simply due to the volumes involved.

(d) Technical Recommendation, Limitation etc.

The concept is considered to be of interest, even if it is more costly than other concepts. However, the concept is based on proven design and it is flexible with respect to parameter variations. As the cost, due to ground improvement, increases rapidly with increasing water depth, it is probably of less interest for larger water depths.

4.2.5 CELLULAR COFFERDAM

(a) Design Analyses

- Stability

No stability analyses have been performed as we have different technical arguments against a cellular cofferdam solution.

- Cellular cofferdam performance

A *properly working* cellular cofferdam is in principle the same stabilizing element as a caisson, either penetrated into the sand or into the clay.

A cellular cofferdam penetrated into the sand implies sheet piles of at least 30 m length, for both the base case alternative as well as for the alternatives with 5 m water depth or 5 m mud thickness. Practical limitations with respect to handling of the sheet piles are by contractors normally set to 25-30 m. Thus, sheet piles of more than 30 m length is beyond normal practice.

If sheet piles of this length can be installed, the next question is how a cellular cofferdam can be established. A properly working cofferdam requires that the mass inside is compacted such that the hoop stresses are mobilized in the cofferdam wall. Thereby the wall obtains its stiffness. With the top soft mud and the slightly stiffer clay below it can be questioned whether these materials will provide sufficient inside support to the cofferdam wall. Most likely the mass inside has to be replaced by material suitable for compaction. This solution, however, hardly serves the purpose of this study.

A cellular cofferdam penetrated into the clay could satisfy the limitations on the length of the sheet piles. This solution will, however, require strength improvement of the clay below. This is shown by the stability analyses performed for the caisson penetrated into the clay, Section 4.2.2. Strength improvement by deep mixing equipment will imply new elements of operational challenges as the equipment as of today is not designed for operation inside a pre-installed cofferdam.

Ground strength improvement raises additional questions. Normally the strength is improved in vertical panels some distance apart, say 4 m. The interaction between discrete panels and the circular, continuous cellular wall can be questionable and may affect the feasibility of the concept. This interaction problem can be avoided by continuous strength improved panels below the cellular wall. Then, however, the cost aspect of the strength improvement becomes important, see below.

A cellular cofferdam penetrated into the clay may, in the same way as a cofferdam penetrated into the sand, suffer from the lack of inside support due to the soft in-situ material.

(b) Construction Phasing

No evaluations have been made.

(c) Cost Evaluations

Assuming strength improvement of the entire clay volume beneath the cellular cofferdam, the cost becomes:

Width: typical 20 m
Layer thickness: 10 m
i.e. volume 200 m^3/m wall

In addition comes the wall itself. Roughly, 14 tons steel per m wall are considered necessary. With a price of HK$12000/ton, the total price is larger than

improvement:	HK$	118 000/m wall
cellular walls:	HK$	168 000/m wall
	HK$	286 000/m wall

With other cost components the total cost will exceed HK$ 300 000/m wall.

(d) Technical Recommendations

Technical aspects and cost figures as given above rule out this alternative.

4.2.6 TIED BACK SHEET PILE WALL

(a) Design Analyses

- Performance

A tied back sheet pile wall has to be penetrated at least a few meters into the dense sand layer to ensure sufficient global stability. The length of each sheet pile will then be 30-40 m, both for the base case alternative as well as for the alternatives with optional water depth and mud thickness. As already mentioned in Section 4.2.5, practical limitations in length with respect to handling of the sheet piles is normally set to 25-30 m by contractors.

The active earth pressure from the vast fill overlying the soft subsoils requires sheet piles with a section modulus way above what is normally used (moments of 5000-10000 kNm).

Placing the fill and installing the anchoring system for the sheet pile wall are very difficult to conduct, bearing in mind that the soft soils hardly can carry any weight until consolidated or improved.

(b) Construction Phasing

No evaluations have been made.

(c) Cost Evaluations

A design moment of the magnitudes calculated above can very well require a wall dimension corresponding to 0.7 tons/m wall. With a cost of HK$ 1200/m wall installed the total cost for the wall itself can very well exceed HK$ 300 000/m.

(d) Technical Recommendations

The concept is not recommended.

4.2.7 STANDARD ALTERNATIVE, CURRENT PRACTICE

For comparison, cost figures are given on Figures 4.19 - 4.24, for 5, 10 and 15 m water depth, both for a block wall solution and a caisson solution.

Rough global stability calculations of the block wall solution are presented in Appendix D, Section D7, and show that the global stability of the block wall placed on the rubble mound meets the criteria of a lump sum safety factor SF $\geq$ 1.2. Further, Figure 4.6 shows that a base width of the block wall of 6-7 m also ensure sufficient capacity against a sliding failure in the gravel bed.

It should be noted that no evaluation of the internal stability of the block wall itself has been performed as a part of this study. The layout of the block wall was given by GEO (1993).

4.3 SLOPING SEAWALLS

4.3.1 FILL ON STRENGTH IMPROVED SUBSOIL

(a) Design Analyses

The geometry of this concept is assumed to be the same as for a fill with subsequently installed concrete quay. Thus, the design analyses presented in Section 4.2.4 are valid. To omit large settlements in the primary fill between the improved zones (differential settlements in excess of 3 m) and to ease the construction of the fill, it is recommended to improve the marine mud and the alluvium clay across the entire width of the primary fill, as describe in Section 4.2.4. Global stability and settlements are then taken care of.

(b) Construction Phasing

The formation of the sloping wall will be done in a traditional way by dumpbarges and end tipping.

The sequences of construction are:
- Soil improvement/drains
- Dumping from splitbarges
- End tipping from top
- Adjustment of front slope
- Erosion protection

A capacity of around 1500 - 2000 m^3 per day of rockfill should be considered to meet the construction schedule.

The required amounts of soil improvement and fill volume varies substantially with variations in water depth with corresponding variations in required production capacities.

The solution requires large amounts of quarry-run rock for the berm construction.

Larger stone blocks are also required for the erosion protection layer.

(c) Cost Evaluations

Cost estimates are given in Figures 4.25 to 4.27, with details in Appendix E.

(d) Technical Recommendation, Limitation etc.

The alternative is recommended. This solution is similar to the rockfill berm for the concrete quay solution, Section 4.2.4, and is a cost effective solution. This solution can also be chosen as an interim solution where a piled concrete quay can be installed later, if desired.

4.3.2 FILL WITH A COUNTERFILL OF VERY GENTLE SLOPE IN FRONT

(a) Design Analyses

Theoretically, soil strength improvement underneath the primary fill may be omitted if the strength improvement zone is replaced by a counterfill of sufficient height and length. However, it is difficult to come up with a design of this counterfill ensuring satisfactory local stability at the very end of this fill, bearing in mind that there is practically no strength in the top of the mud. Thus, also for a counterfill, we would recommend to stabilize under the toe by use of soil strength improvement.

Stability calculations show that if full consolidation for the primary fill is assumed, a counterfill of at least 5 m height is needed for base case assumptions. Most likely, only partial consolidation of the marine mud and alluvium clay can be relied upon within a reasonable construction time period (about one year). Thus, the height of the counterfill must be about 7 m.

For 5 m and 15 m water depth, the required height of the counterfill is judged to 3 m and 10 m, respectively.

(b) Construction Phasing

It is essential to establish the strength improved zones first. Thereafter the band drains are installed. A geotextile is placed on the sea floor in order to support the fill. With the geotextile in place, a careful filling of a thin sand layer (0.5-1 m) can start.

The construction of most of the fill will require use of barges.

The counterfill and the primary fill up to the same height as the counterfill is then installed and left to settle for about one year. Then the primary fill is installed to its full height.

(c) Cost Evaluations

The cost estimate given in Figures 4.28 to 4.30 indicates that the alternative is cost effective. It can be even better with respect to cost if some waste material can be used as counterfill material (filter criteria and erosion may prevent this). On the other hand, if satisfactory construction progress of the primary fill requires closer spacing between the drains, the cost can increase considerably. According to comments from GEO, this latter aspect is not of major interest for comparison of the concept with other concepts.

(d) Technical Recommendation, Limitation etc.

The concept will need approximately 7 m high counterfill for 10 m water depth in order to obtain sufficient stability with only partial consolidation of the mud/clay for the fill weight. With 7 m counterfill the available water depth thus appears to be considerably limited. To satisfy stability requirements one must build up the fill in steps and allow for consolidation and strength gain in the subsoil. With a c/c distance of 1.5 m between drains the required construction time is minimum 1 year.

Otherwise, the concept is flexible and makes use of well known technology.

4.3.3 FILL ON RELIEF PILES

(a) Design Analyses

- Stability

Rough estimates of pile bearing capacity and horizontal sliding have been performed and presented in Appendix D, Section D6.

For a reasonable c/c distance between the piles of 4 m, they have to carry about 3000 KN in axial load from the fill weight (base case). This axial loads can hardly be taken by conventional, driven concrete piles with pile tips in the sand strata. Assuming the piles driven to 30 m depth, proper end bearing may require a pile cross section of 0.35 m^2, or 0.6 x 0.6 m. For this reason it will probably be better to drive piles to bedrock (at 40 m depth below seabed according to base case soil profile).

To resist the earth pressure from the fill, a width of the piled area of about 80 m is required. Behind the piled area the mud is then assumed consolidated for the fill weight by use of vertical drains.

A water depth of 5 m will reduce the required width of the piled area to about 60 m. Axial loads on the piles reduce to about 2200 KN.

Reduction in mud layer thickness will insignificantly affect the pile layout as compared to the base case alternative. The design is mainly governed by the fill height and the strength of the mud.

- Interaction between fill and piles

Interaction between fill and pile caps requires use of geotextile to transfer loads from the fill to the pile caps. The earth pressure from the fill will, for base case condition, require use of geotextile with an ultimate strength of ≈ 1000 KN/m and a design strength of approximately 50% of the ultimate strength.

(b) Construction Phasing

No evaluations have been made due to the cost figures and technical recommendations below.

(c) Cost Evaluations

HK$ 0.6/KN,m pile installed is judged to be a realistic figure. The likely cost range is 0.2-1.0 HK$/KN,m.

Pile costs	=	360 000 HK$/m
Drainage	=	1 000 HK$/m
Geotextile	=	15 000 HK$/m
		376 000 HK$/m

(d) Technical Recommendation, Limitation etc.

The cost estimate indicates that the alternative is not so attractive. In addition there are also technical arguments against this solution. The piles need to be driven to bedrock. The bedrock level will introduce some uncertainty. The piles need a design capacity at the limit or even beyond the standard practice known to us. The required strength of the geotextile is also very high, at the limit of today's practice.

For these reasons the alternative is not recommended.

4.3.4 STANDARD ALTERNATIVE, CURRENT PRACTICE

Figures 4.31 to 4.33 show cost figures for comparison.

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LIST OF TABLES

Table 2.1 - Base Case Design Soil Parameters

Stratum	Depth (m)	Soil Description	γ_{tot} (kN/m^3)	s_u^C (kPa)	s_u^{DSS} (kPa)	s_u^E (kPa)	m	c_v (m^2/yr)	M (MPa)	ϕ' (deg.)	c' (kPa)
I	0-10	Soft clay	15	$0.35 \cdot p_o'$	$0.28 \cdot p_o$	$0.21 \cdot p_o'$	10	4		28	0
II	10-20	Silty clay	18	40	32	24	15	10			
III	20-25	Sand	19						50	40	0
IV	25-30	Clay	19	60	48	36	15	5			
V	30-40	Sand	19						50	38	0
	>40	Rock									

Legend :

γ_{tot} = Total unit weight
s_u^C = Undrained triaxial compression shear strength
s_u^{DSS} = Undrained direct simple shear strength
s_u^E = Undrained triaxial extension shear strength
m = Dimensionless modulus number = $((1 + e_o)/c_c) \cdot \ell n\ 10$
c_v = Coefficient of vertical consolidation
M = Constrained deformation modulus
ϕ' = Triaxial peak drained friction angle
c' = Cohesion (in terms of effective stresses)

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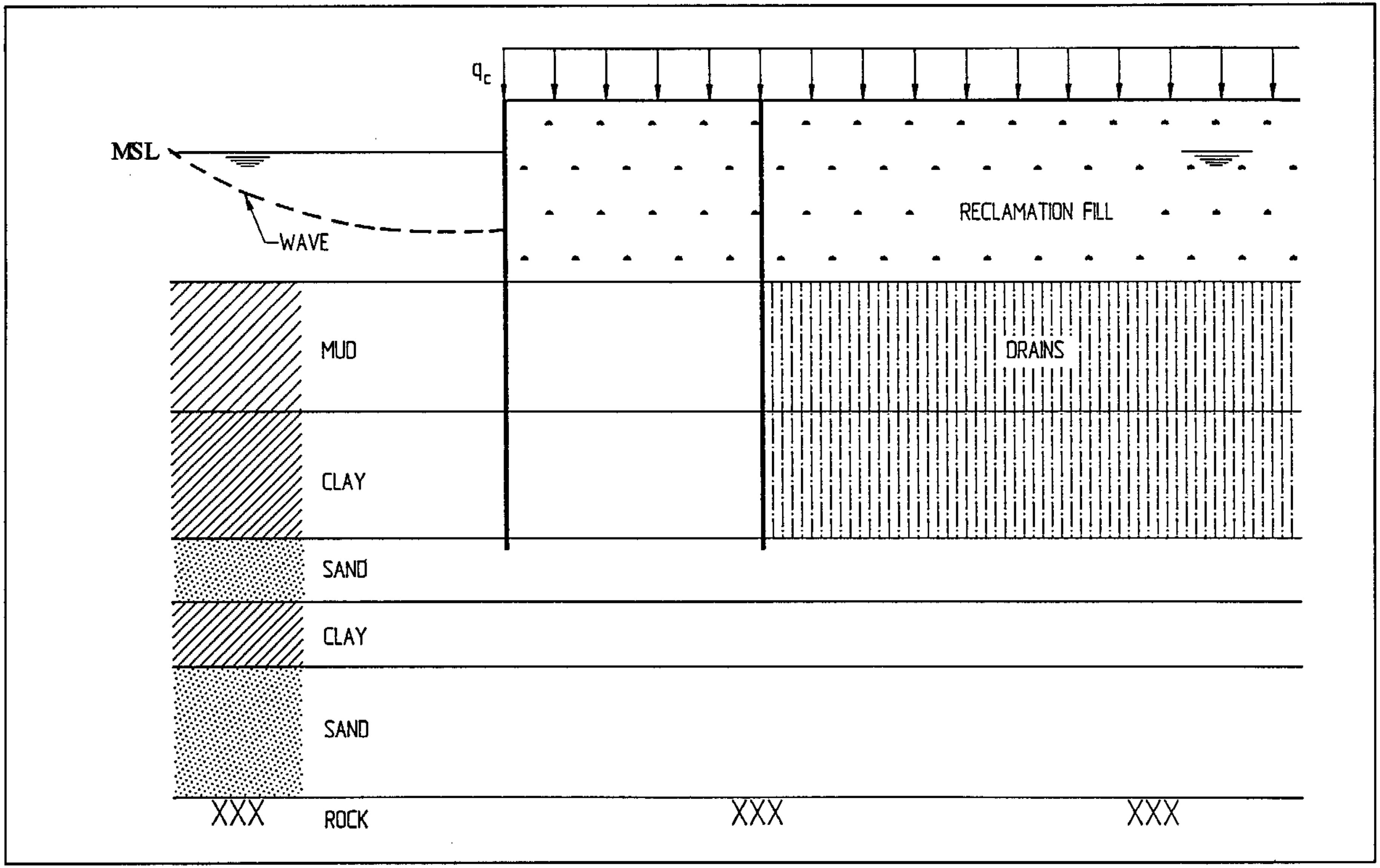


Figure 3.1 - Principle Sketch of Vertical Seawall

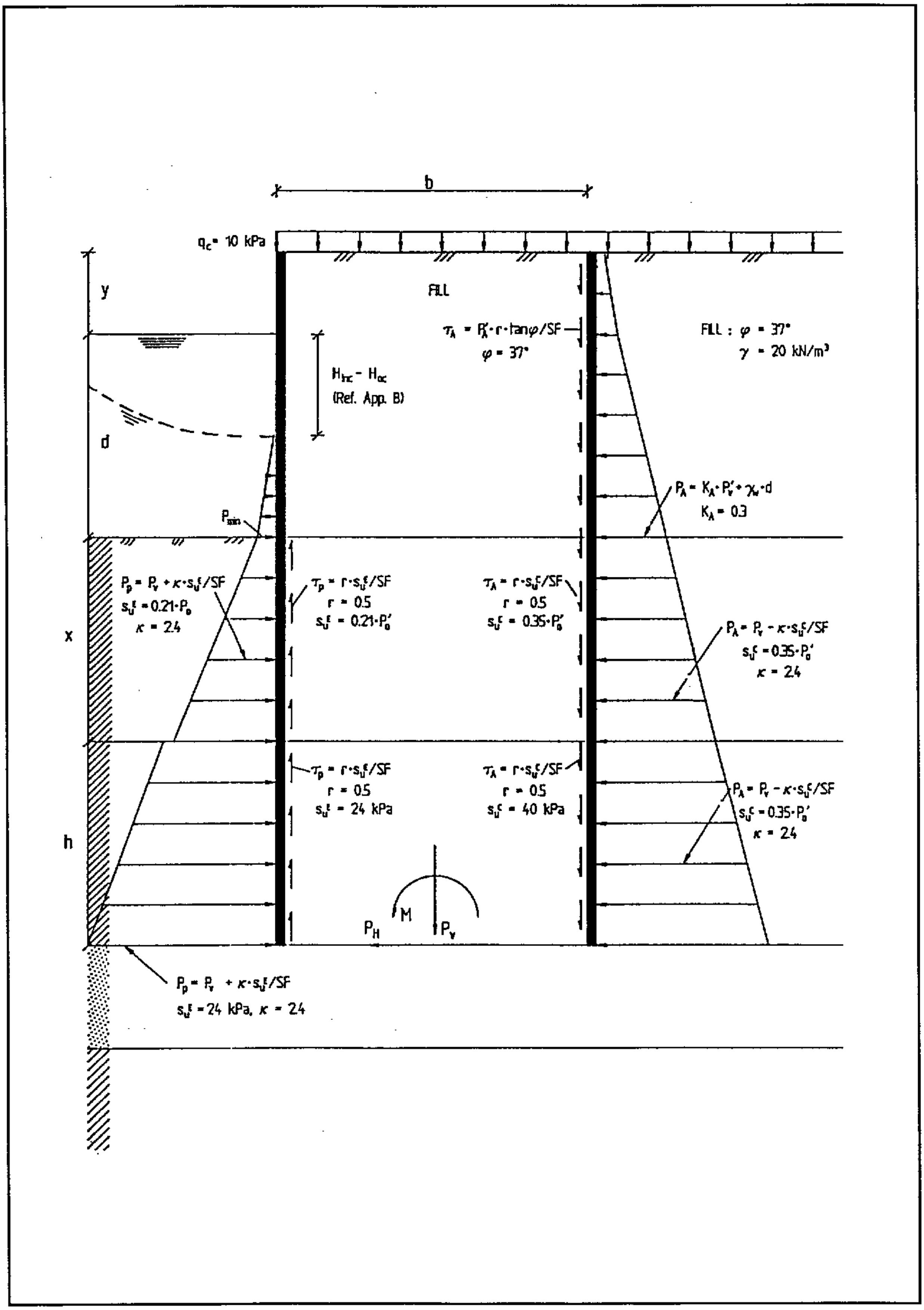


Figure 3.2 - Principle Sketch of Earth Pressures and Water Pressures

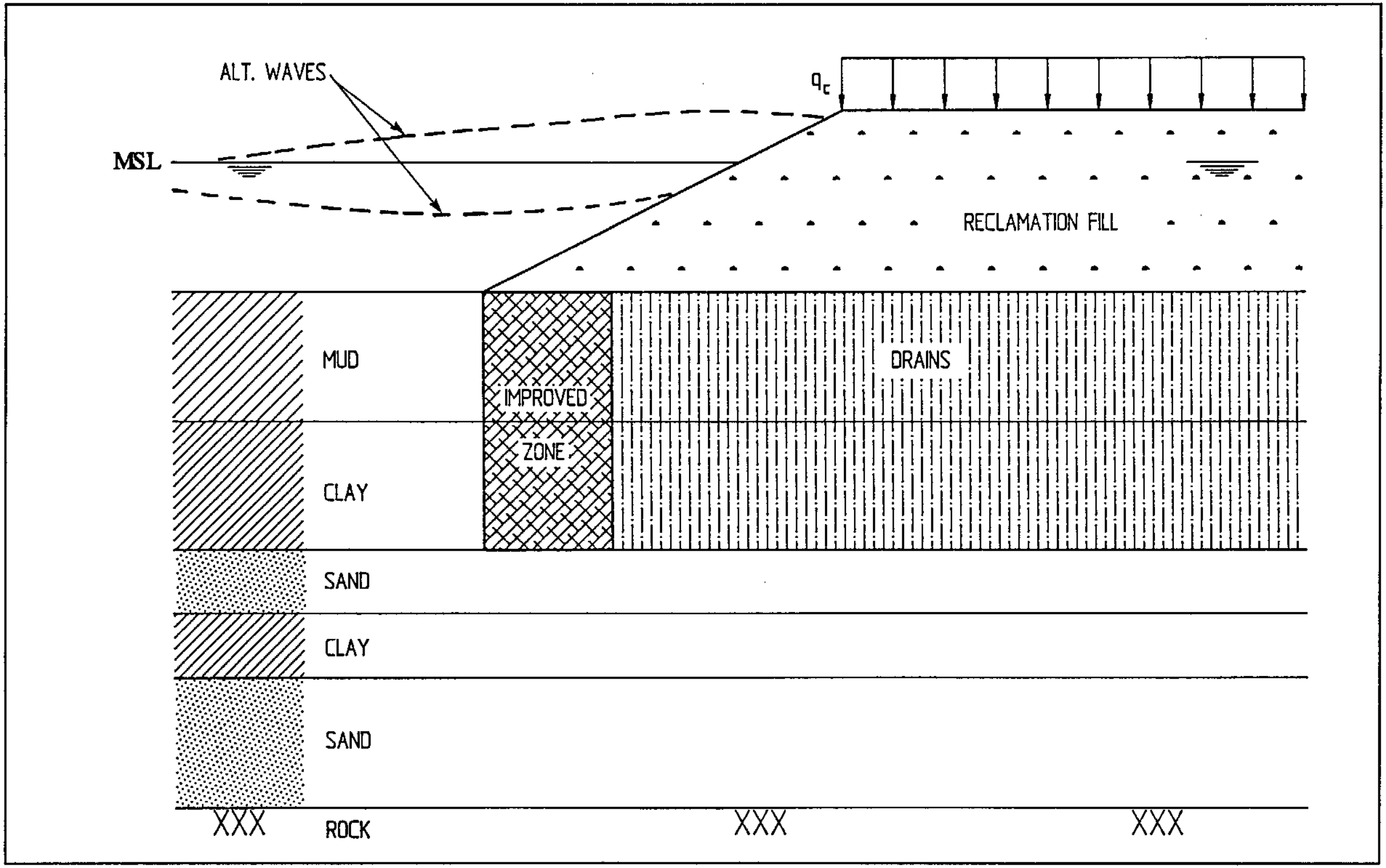


Figure 3.3 - Principle Sketch of Sloping Seawall

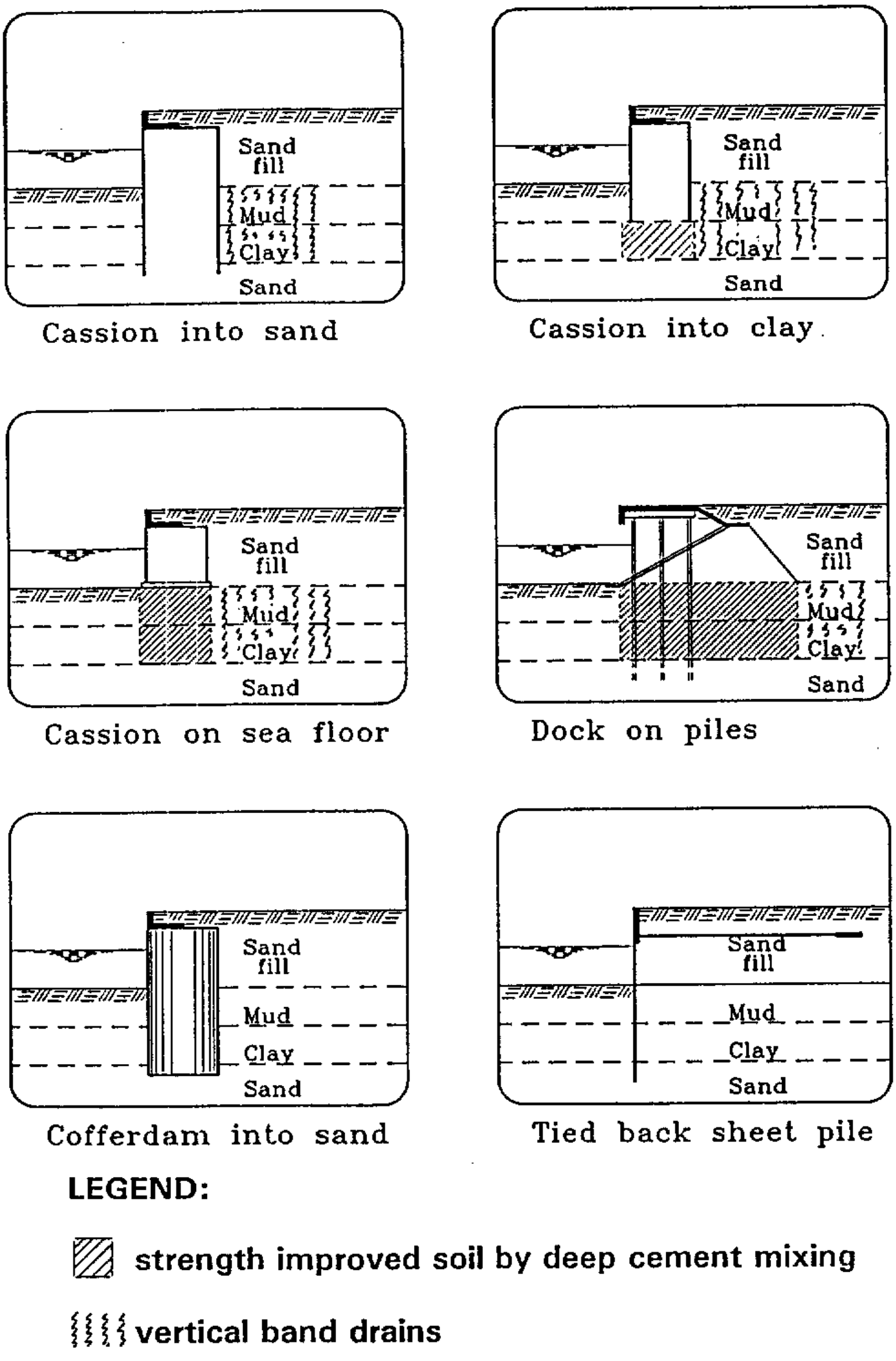
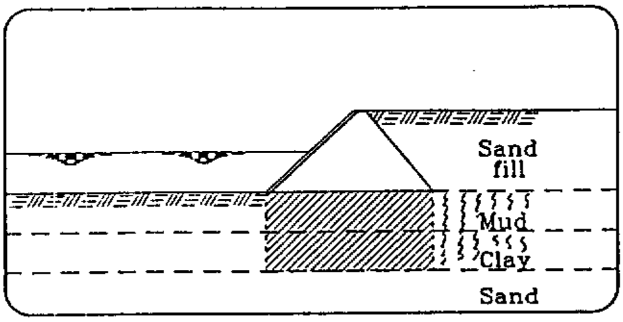
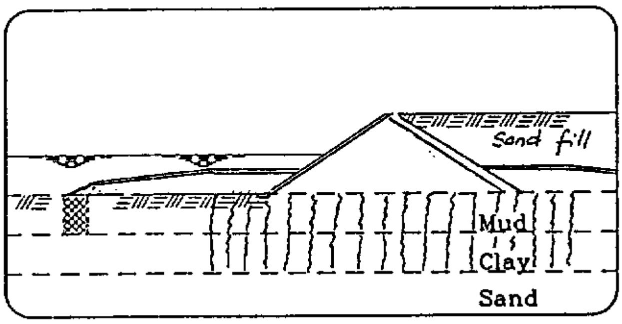


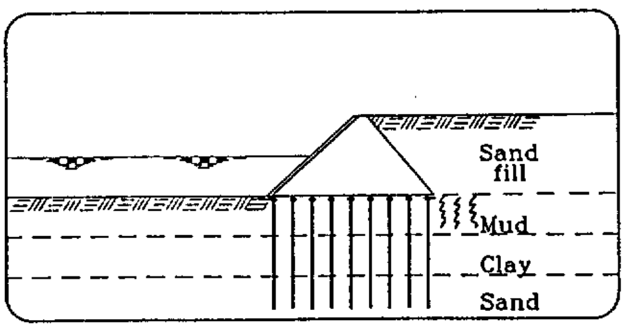
Figure 4.1 - Vertical Seawall, Concepts Evaluated



Sloping wall



Sloping wall – gentle counter fill



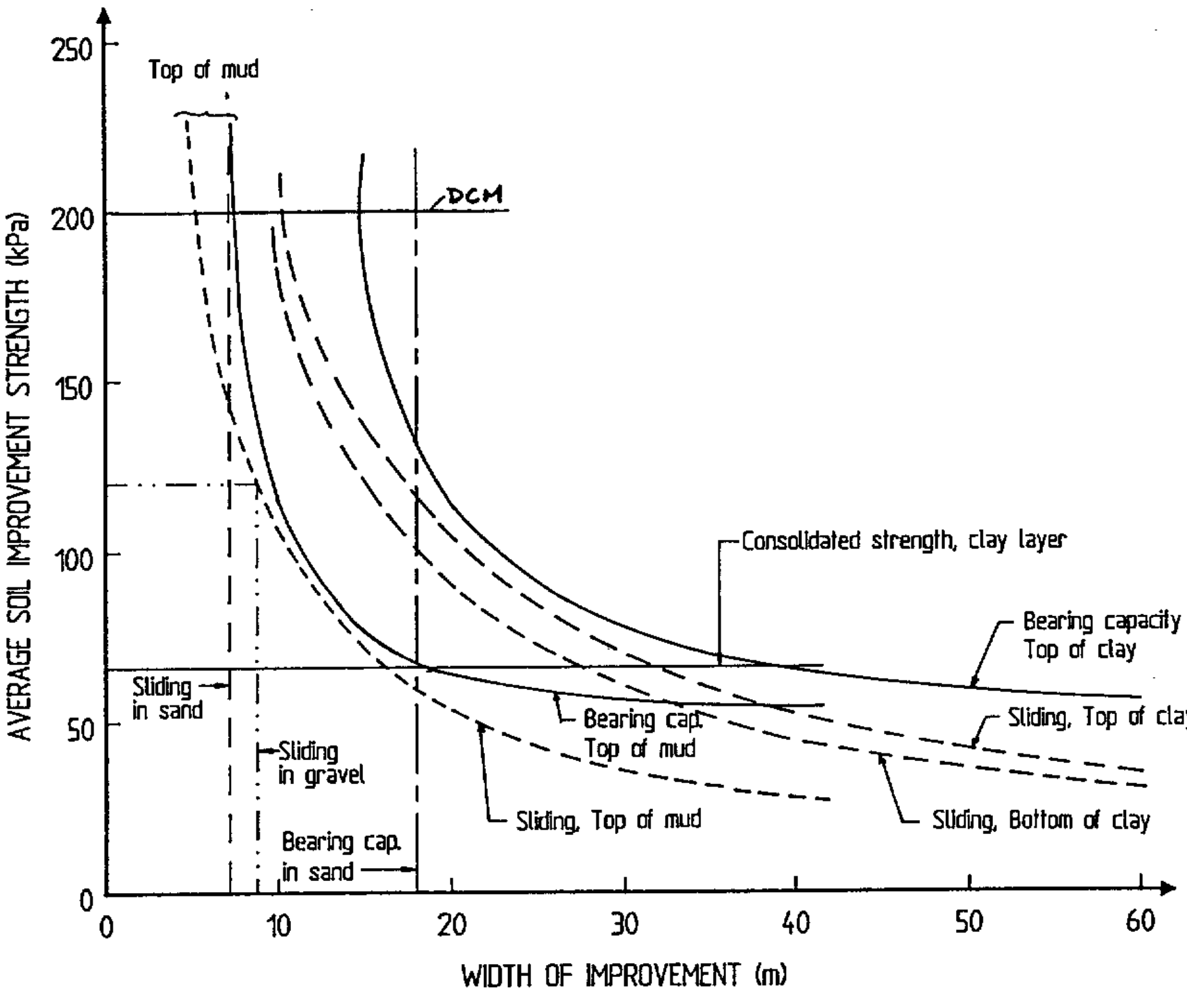
Sloping wall – relief piles

LEGEND:

strength improved soil by deep cement mixing

vertical band drains

Figure 4.2 - Sloping Seawall, Concepts Evaluated



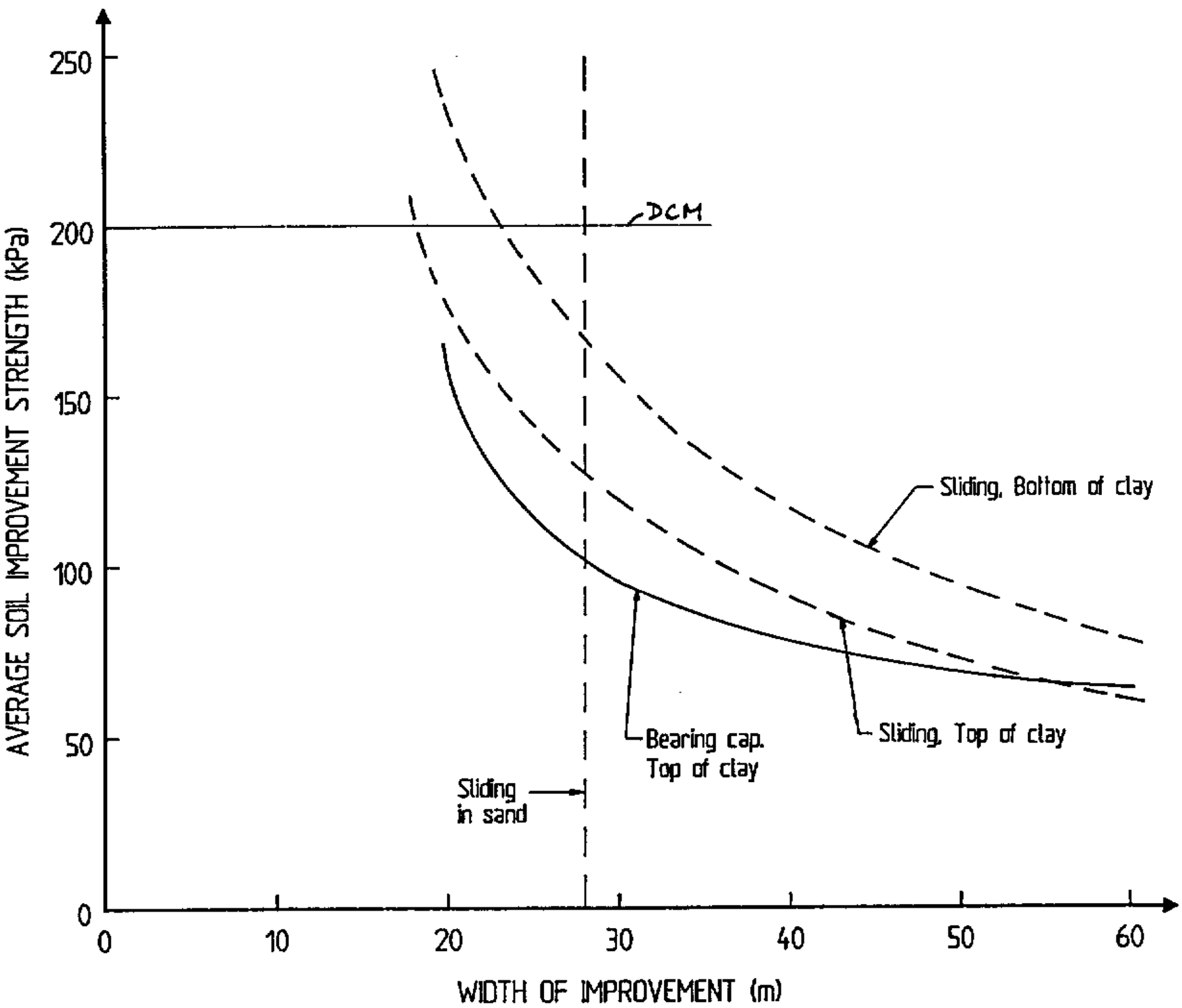
Water depth : 10m

"Mud" thickness : 10m

Fill : γ = 20 kN/m^3 , φ = 37°

Comment : q_c = 10 kPa, SF = 1.2

Figure 4.3 - Stability Calculations, Base Case Input Parameters



Water depth : 10m

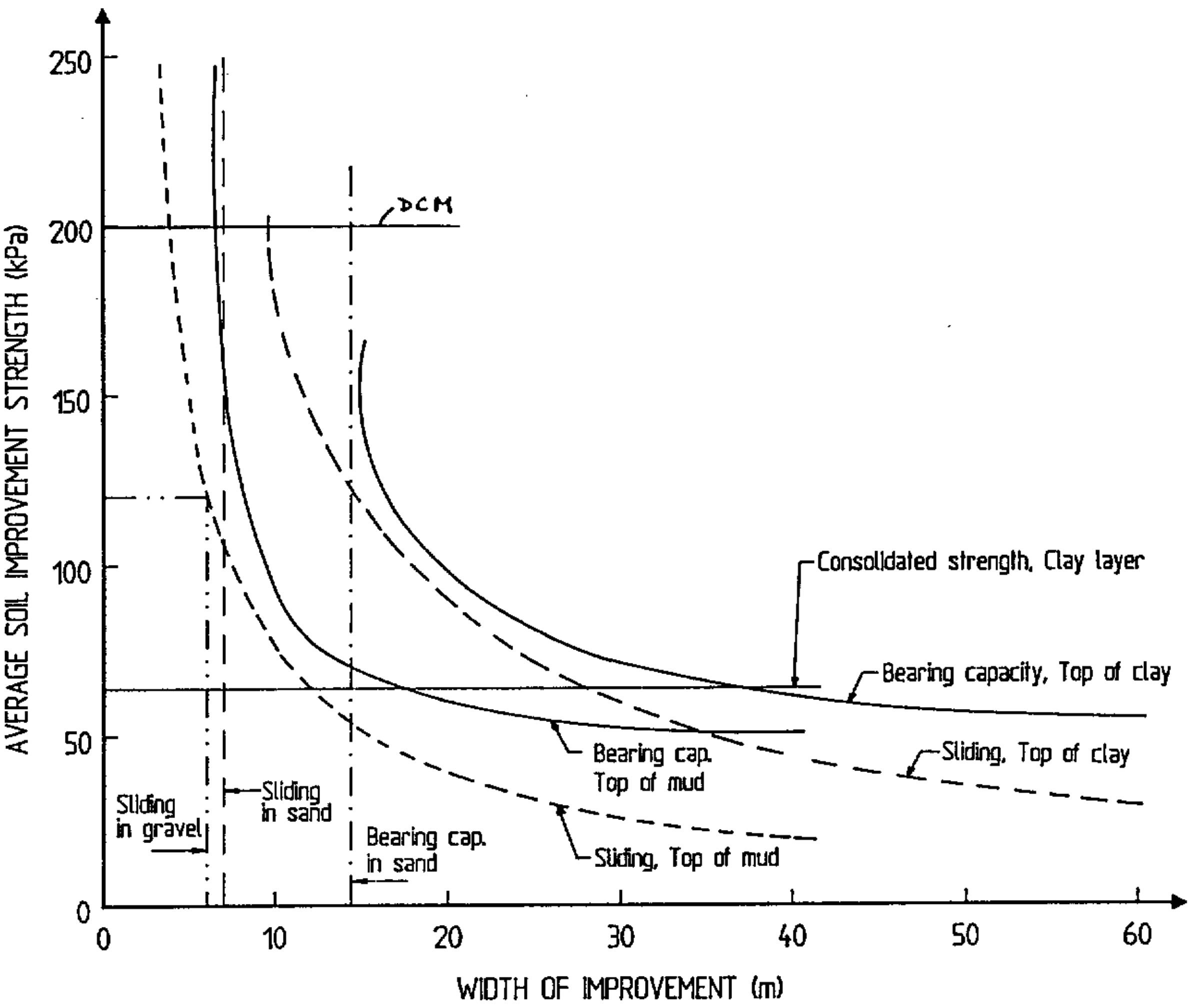
"Mud" thickness : 10m

Fill : γ = 20 kN/m^3 , φ = 37°

Comment : q_c = 10 kPa, SF = 1.2

No consolidation

Figure 4.4 - Stability Calculations, No Drains behind the Vertical Wall



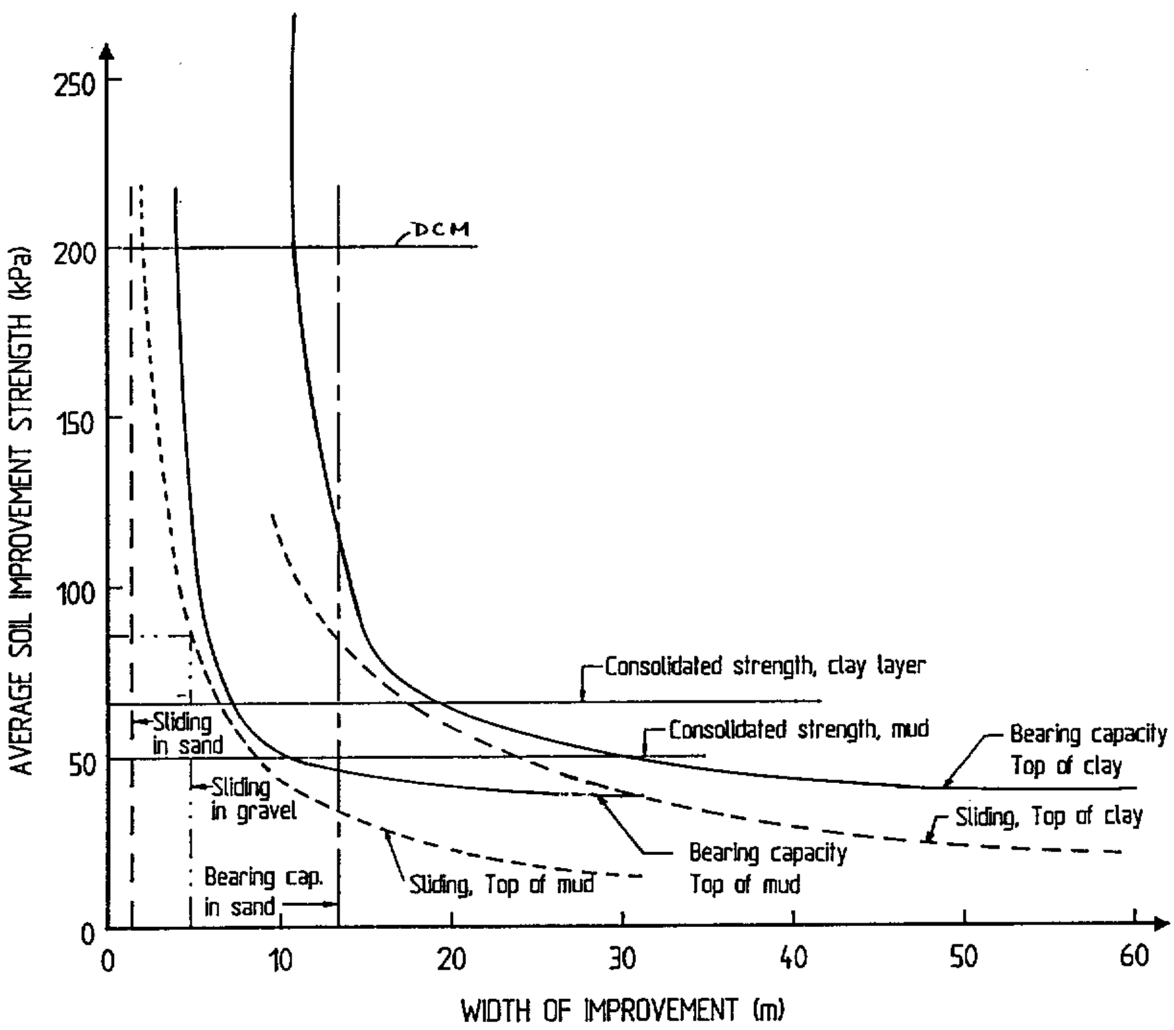
Water depth : 10m

"Mud" thickness : 10m

Fill : γ = 11 kN/m³ , φ = 37°

Comment : q_c = 10 kPa, SF = 1.2
No consolidation

Figure 4.5 - Stability Calculations, Light Weight Fill behind the Vertical Wall



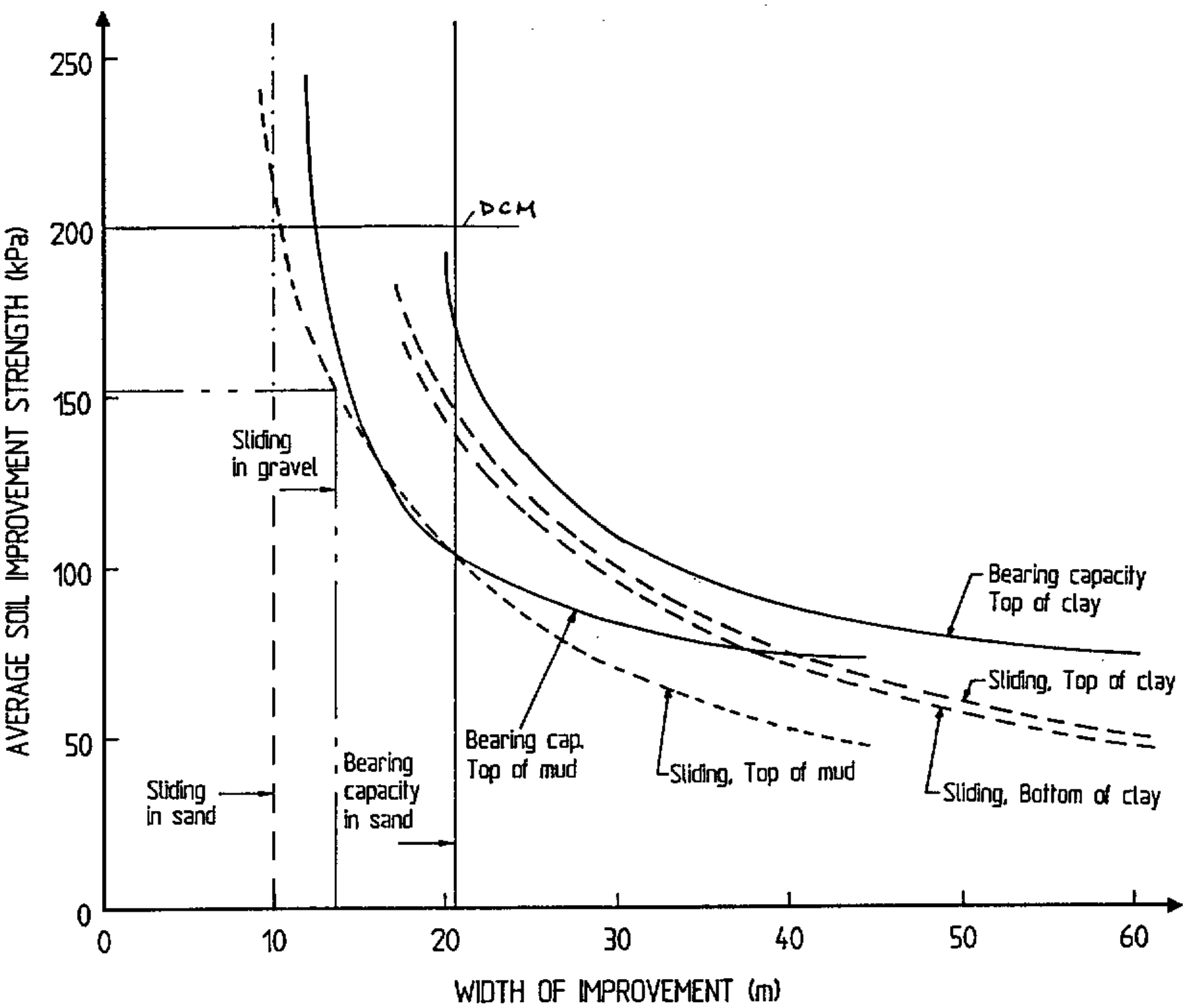
Water depth : 5m

"Mud" thickness : 10m

Fill : γ = 20 kN/m^3 , φ = 37°

Comment : q_c = 10 kPa, SF = 1.2

Figure 4.6 - Stability Calculations, 5 m Water Depth



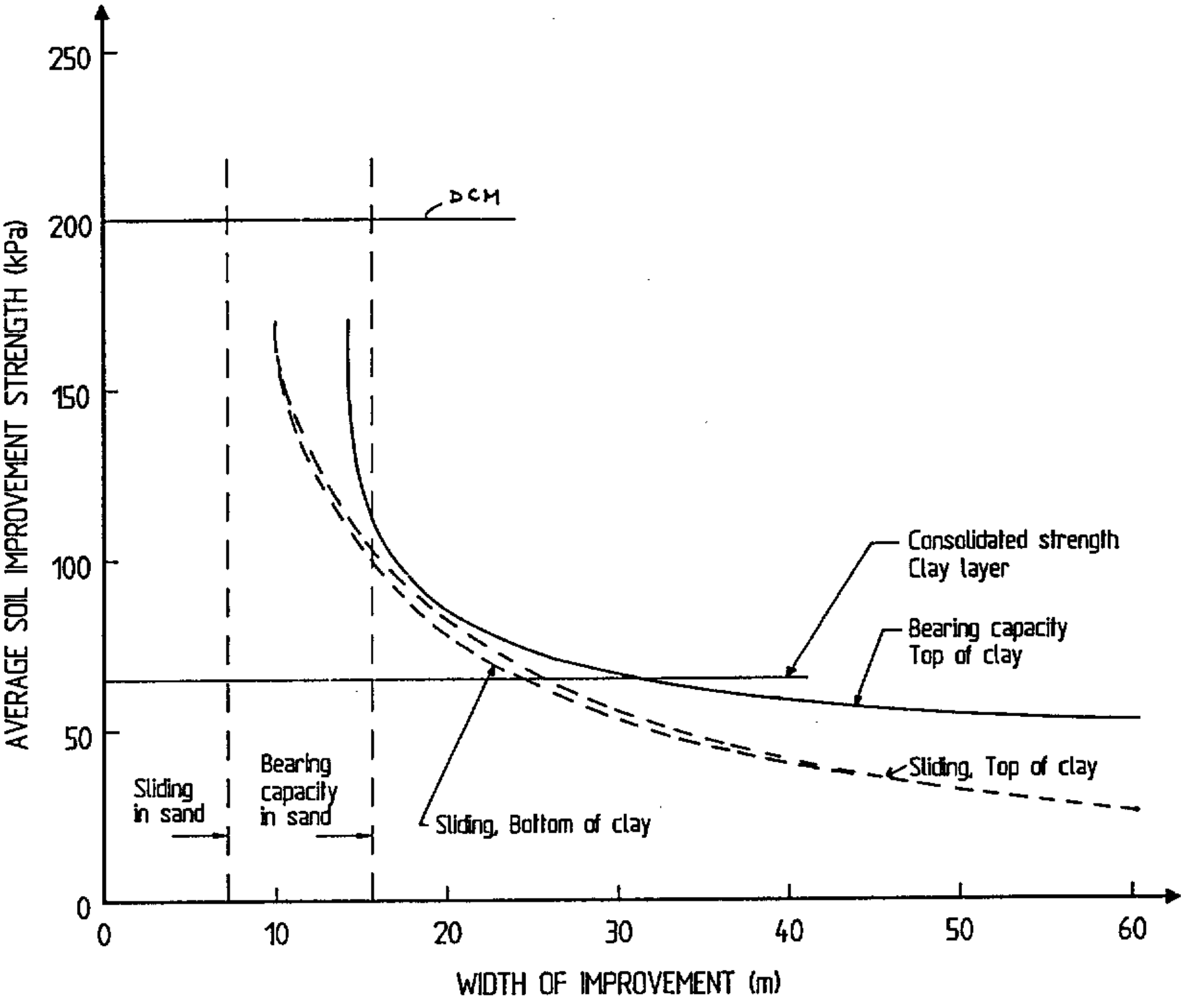
Water depth : 15m

"Mud" thickness : 10m

Fill : γ = 20 kN/m^3 , φ = 37°

Comment : q_c = 10 kPa, SF = 1.2

Figure 4.7 - Stability Calculations, 15 m Water Depth



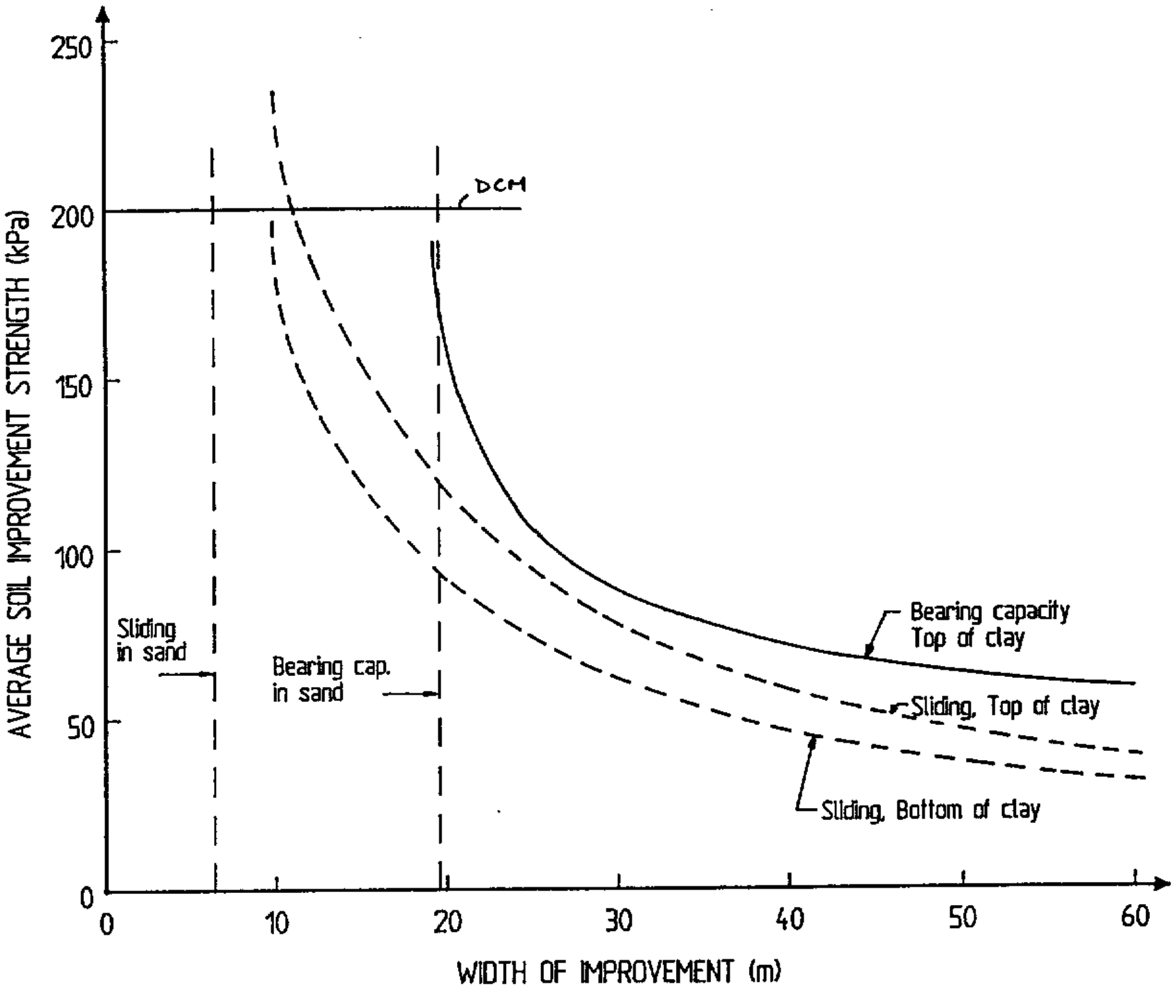
Water depth : 10m

"Mud" thickness : 5m

Fill : γ = 20 kN/m^3, φ = 37°

Comment : q_c = 10 kPa, SF = 1.2

Figure 4.8 - Stability Calculations, 5 m Marine Mud Thickness



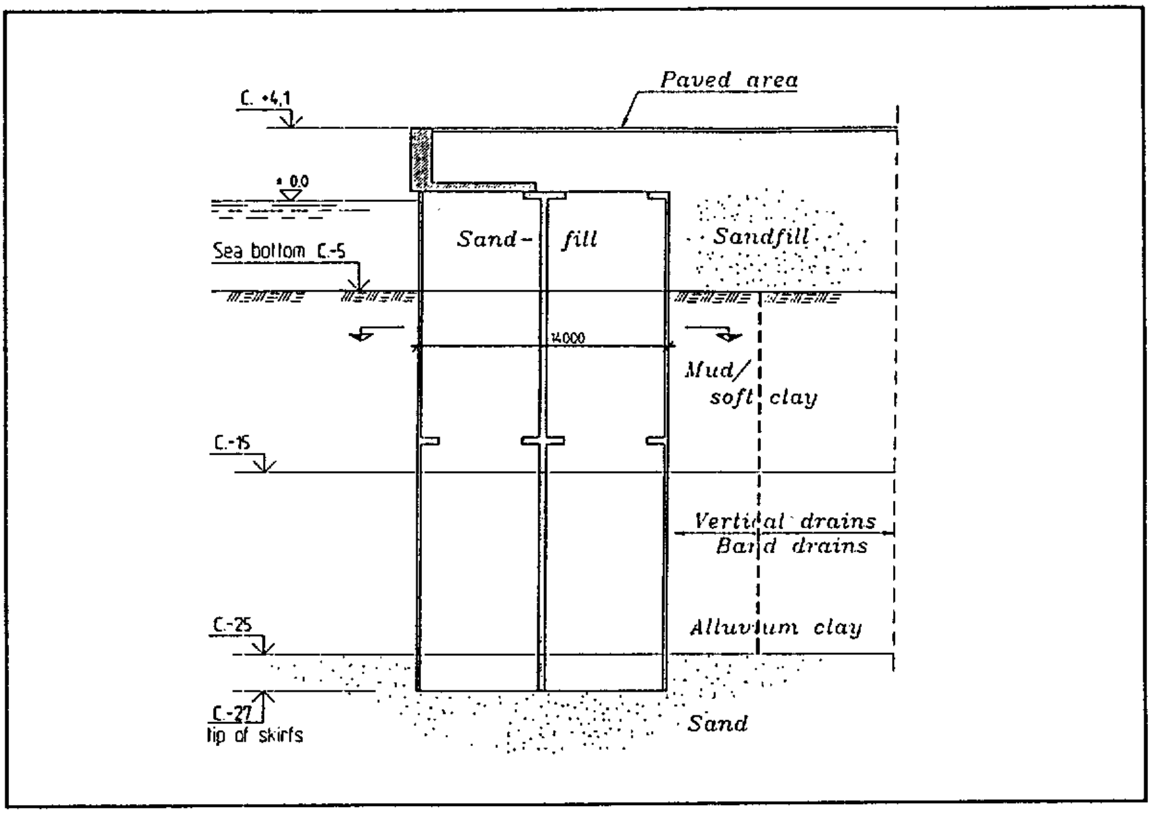
Water depth : 10m

"Mud" thickness : 15m

Fill : γ = 20 kN/m^3 , φ = 37°

Comment : q_c = 10 kPa, SF = 1.2

Figure 4.9 - Stability Calculations, 15 m Marine Mud Thickness

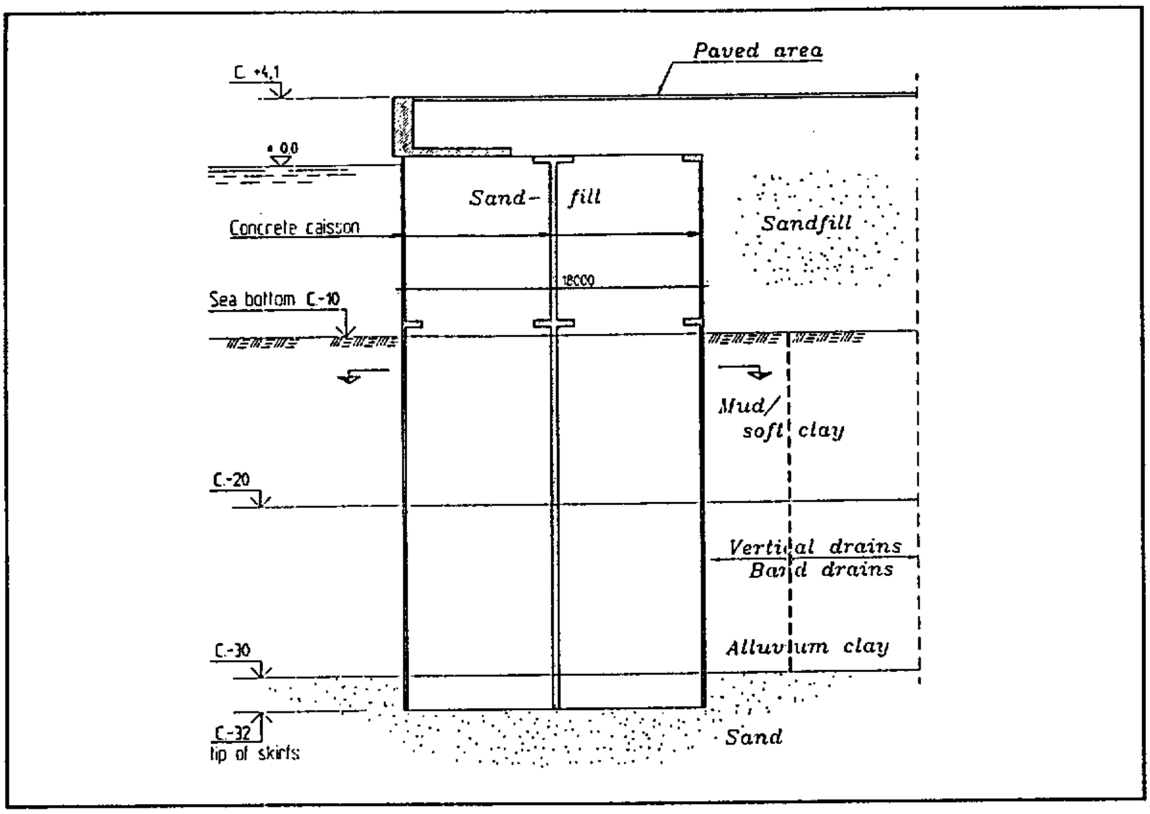


Comments:			
	General cost:	HK$	35,000,000
	Concrete works:	"	133,000,000
	Foundations works:	"	22,000,000
	Filling works:	"	41,000,000
	Drains:	"	13,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	244,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.10 - Cost Figure for Caisson into Sand, 5 m Water Depth

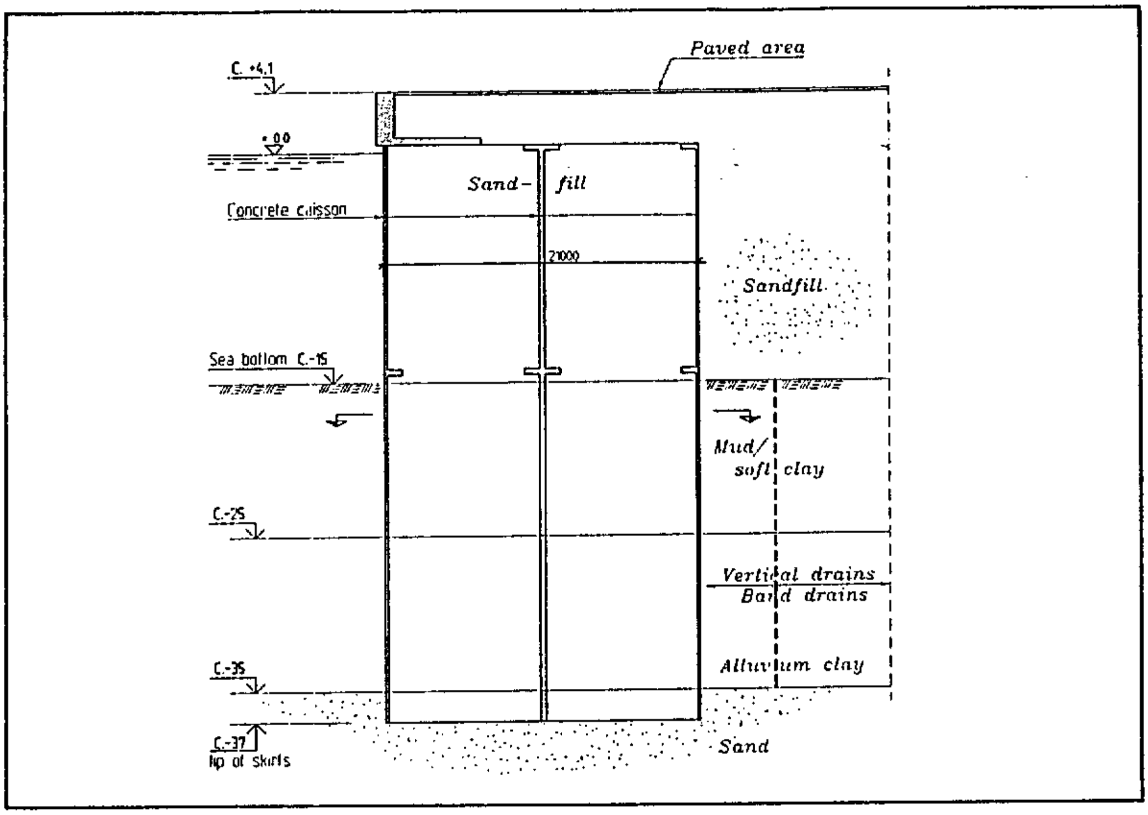


Comments:			
	General cost:	HK$	35,000,000
	Concrete works:	"	149,000,000
	Foundations works:	"	19,000,000
	Filling works:	"	61,000,000
	Drains:	"	11,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	275,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1		2		3		4	
Ground improvements								
Foundation works								
Concrete works								
Filling works								

Figure 4.11 - Cost Figure for Caisson into Sand, 10 m Water Depth

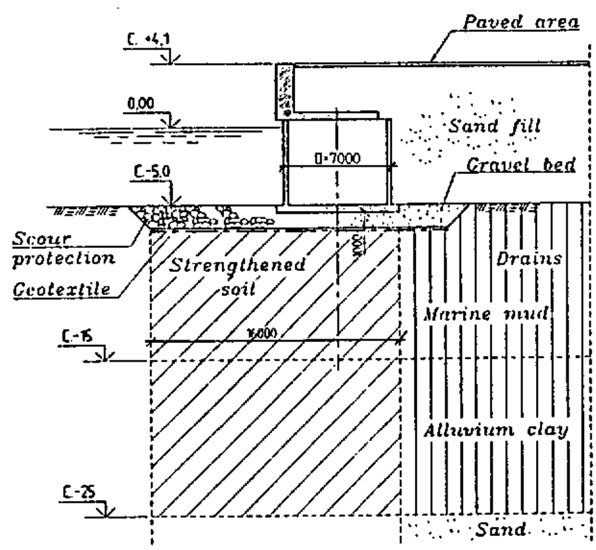


Comments:	General cost:	HK$	35,000,000
	Concrete works:	"	165,000,000
	Foundations works:	"	19,000,000
	Filling works:	"	80,000,000
	Drains:	"	10,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	309,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.12 - Cost Figure for Caisson into Sand, 15 m Water Depth

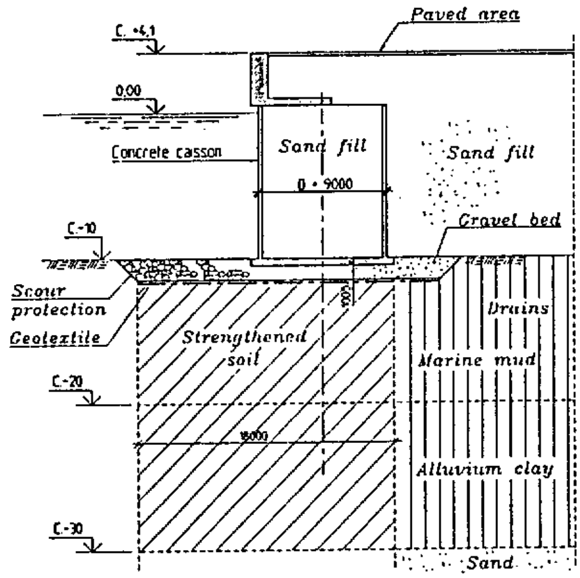


Comments:			
	General cost:	HK$	41,000,000
	Concrete works:	"	37,000,000
	Foundations works:	"	10,000,000
	Filling works:	"	40,000,000
	Drains:	"	15,000,000
	Ground improvements:	"	41,000,000
	SUM: 1000 m	HK$	184,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.13 - Cost Figure for Caisson on Improved Soil, 5 m Water Depth

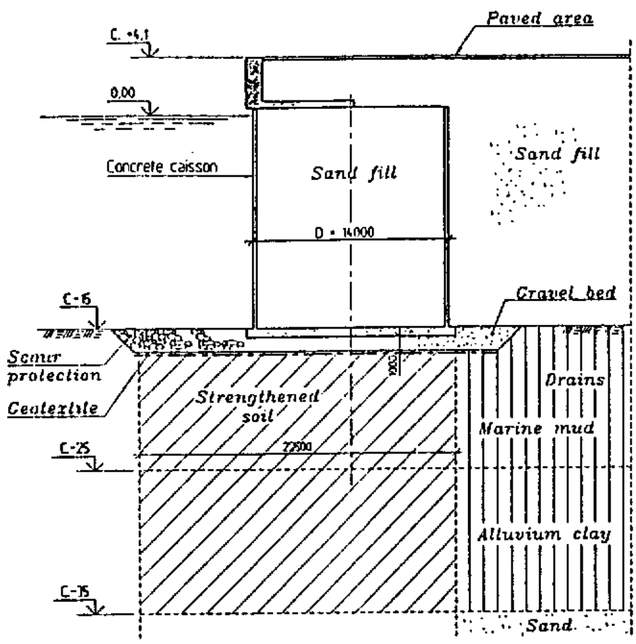


Comments:			
Cost effective, technically flexible, well known technology, not sensitive to variation in soil data and layering	General cost:	HK$	41,000,000
	Concrete works:	"	52,000,000
	Foundations works:	"	12,000,000
	Filling works:	"	56,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	54,000,000
	SUM: 1000 m	HK$	229,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.14 - Cost Figure for Caisson on Improved Soil, 10 m Water Depth

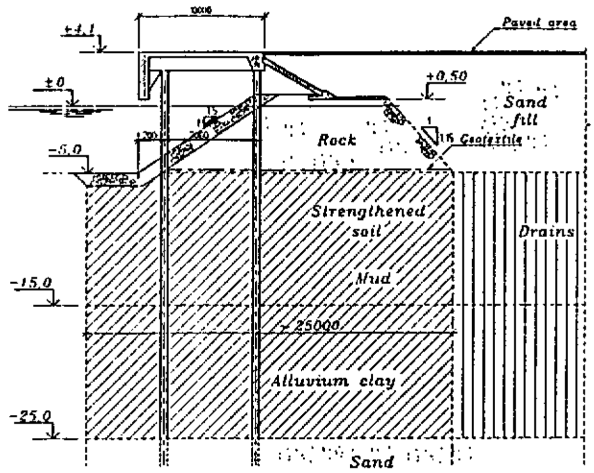


Comments:			
	General cost:	HK$	41,000,000
	Concrete works:	"	85,000,000
	Foundations works:	"	14,000,000
	Filling works:	"	70,000,000
	Drains:	"	13,000,000
	Ground Improvements:	"	64,000,000
	SUM: 1000 m	HK$	287,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.15 - Cost Figure for Caisson on Improved Soil, 15 m Water Depth

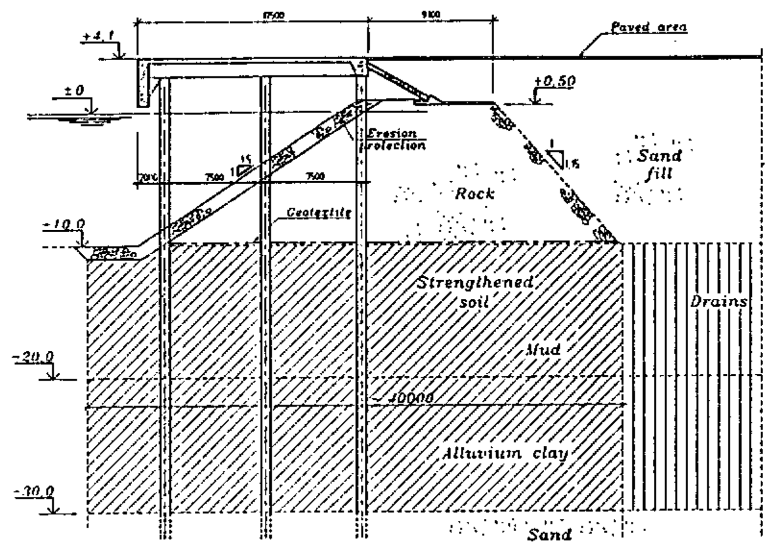


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	29,000,000
	Foundations works:	"	31,000,000
	Filling works:	"	45,000,000
	Drains:	"	9,000,000
	Ground improvements:	"	79,000,000
	SUM: 1000 m	HK$	229,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.16 - Cost Figure for Piled Quay on Improved Soil, 5 m Water Depth

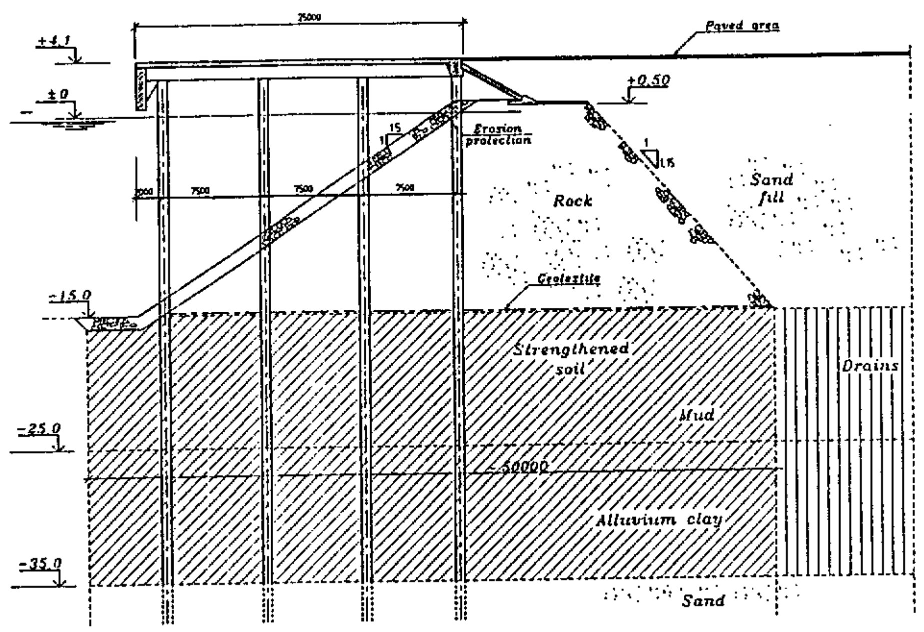


Comments:	General cost:	HK$	36,000,000
Technically flexible, well known technology, quay front free of settlements, Costs sensitive to water depth and depth to bed rock	Concrete works:	"	35,000,000
	Foundations works:	"	48,000,000
	Filling works:	"	61,000,000
	Drains:	"	5,000,000
	Ground improvements:	"	120,000,000
	SUM: 1000 m	HK$	305,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.17 - Cost Figure for Piled Quay on Improved Soil, 10 m Water Depth

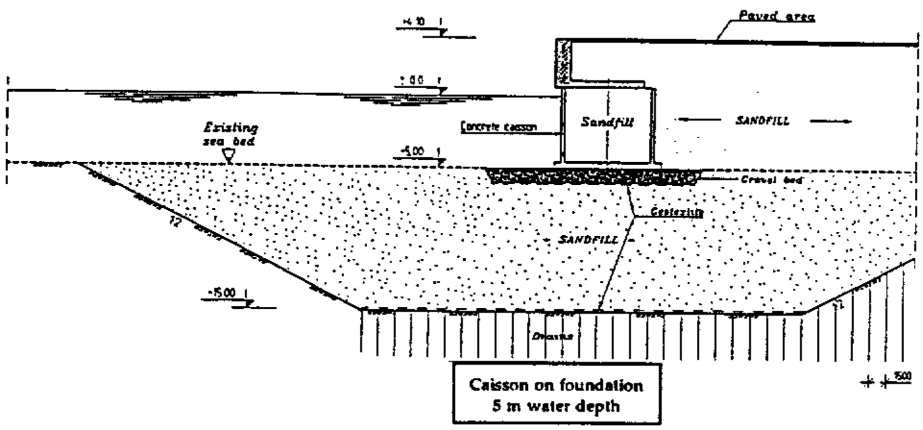


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	40,000,000
	Foundations works:	"	68,000,000
	Filling works:	"	79,000,000
	Drains:	"	1,000,000
	Ground improvements:	"	159,000,000
	SUM: 1000 m	HK$	383,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.18 - Cost Figure for Piled Quay on Improved Soil, 15 m Water Depth

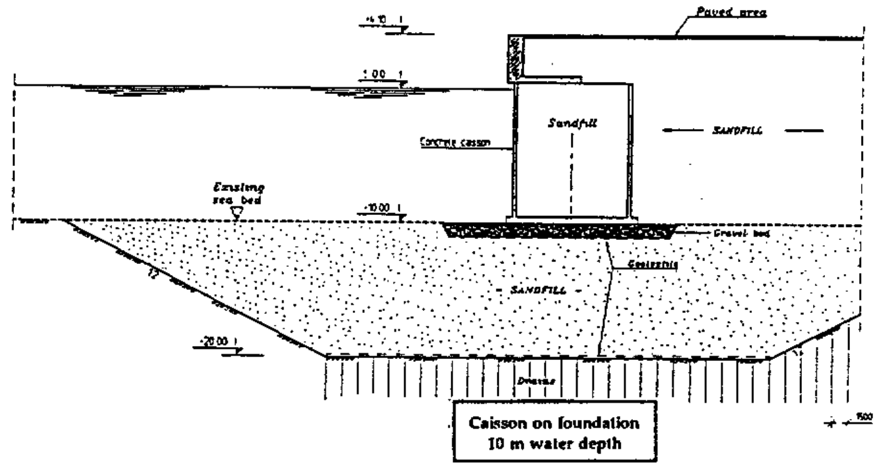


Comments:			
	General cost:	HK$	41,000,000
	Concrete works:	"	37,000,000
	Foundations works:	"	66,000,000
	Filling works:	"	39,000,000
	Drains:	"	15,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	198,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1		2		3		4	
Ground improvements								
Foundation works								
Concrete works								
Filling works								

Figure 4.19 - Cost Figure for Caisson on Dredged Foundation, 5 m Water Depth

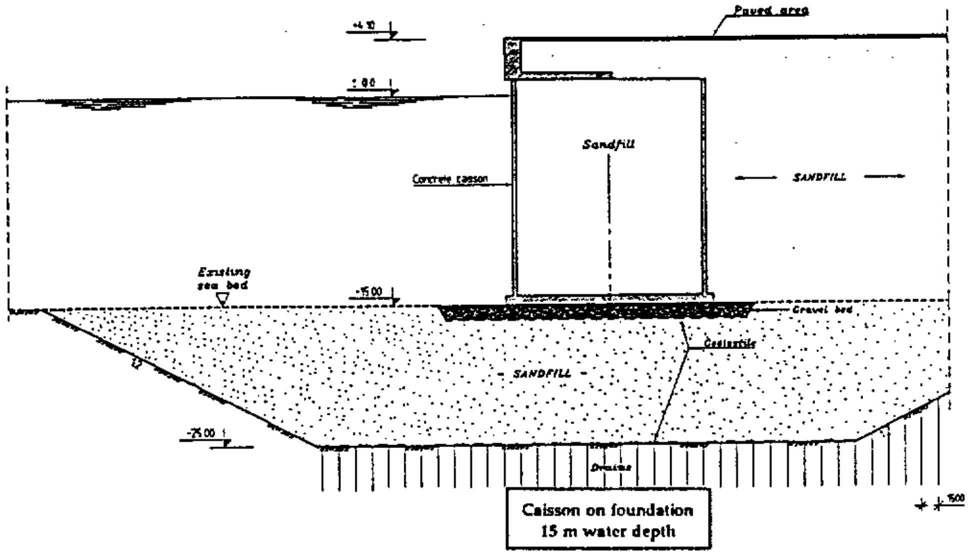


Comments:			
	General cost:	HK$	41,000,000
	Concrete works:	"	52,000,000
	Foundations works:	"	67,000,000
	Filling works:	"	56,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	230,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works	━━	━━	━	
Concrete works	━━	━━	━	
Filling works		━	━━	

Figure 4.20 - Cost Figure for Caisson on Dredged Foundation, 10 m Water Depth

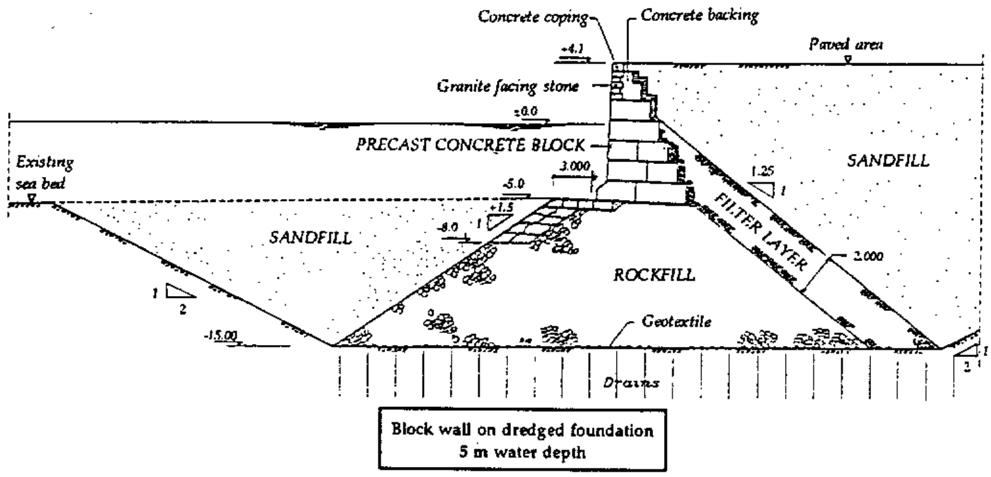


Comments:			
	General cost:	HK$	41,000,000
	Concrete works:	"	83,000,000
	Foundations works:	"	66,000,000
	Filling works:	"	73,000,000
	Drains:	"	13,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	276,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1		2		3		4	
Ground improvements								
Foundation works								
Concrete works								
Filling works								

Figure 4.21 - Cost Figure for Caisson on Dredged Foundation, 15 m Water Depth

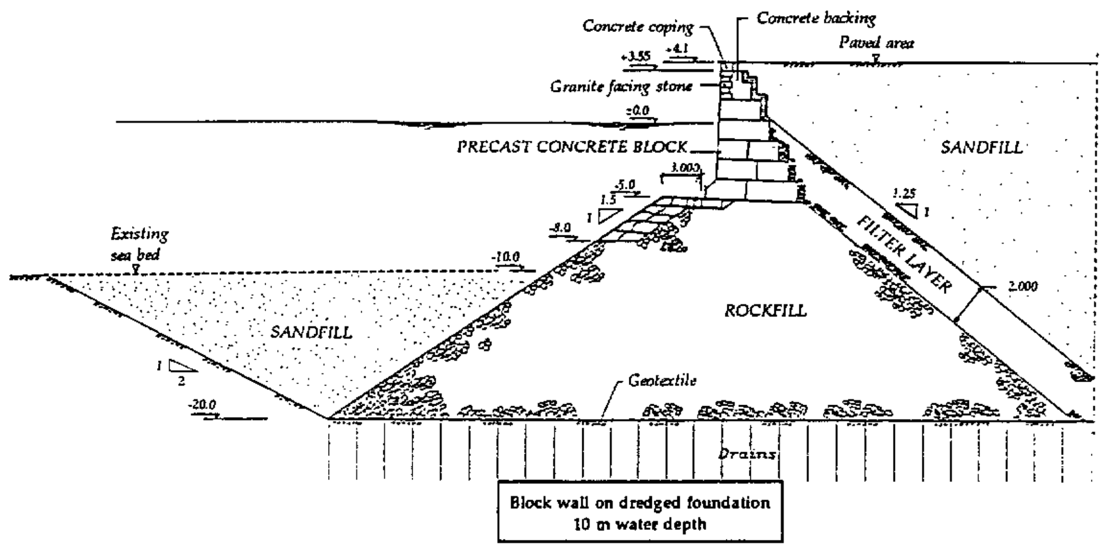


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	33,000,000
	Foundations works:	"	46,000,000
	Filling works:	"	60,000,000
	Drains:	"	15.000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	190,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.22 - Cost Figure for Blockwall on Dredged Foundation, 5 m Water Depth

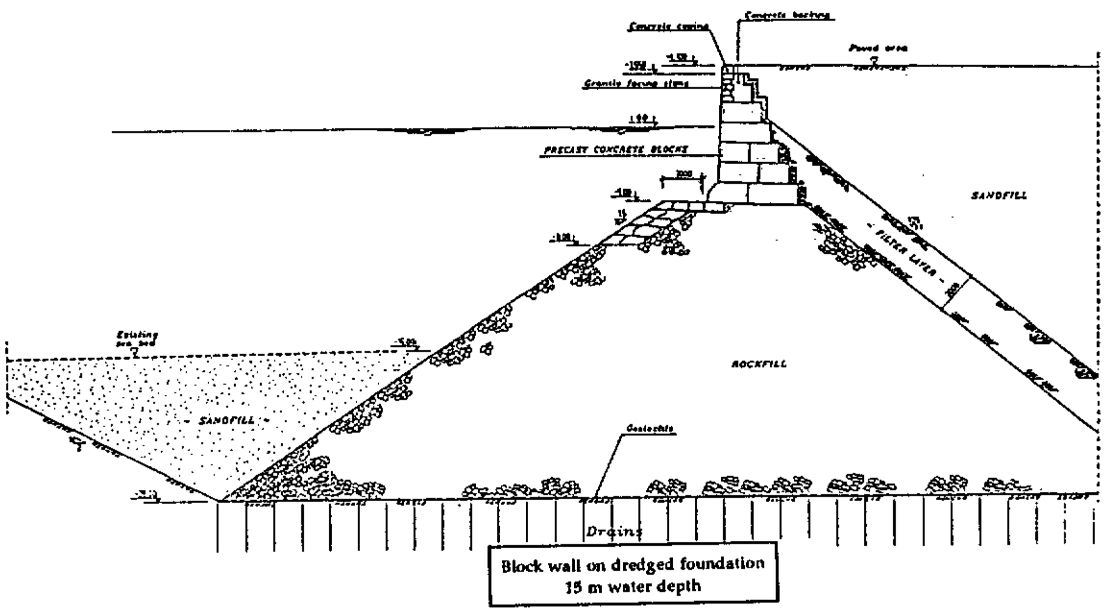


Comments:			
	General cost:	HK$	35,000,000
	Concrete works:	"	33,000,000
	Foundations works:	"	74,000,000
	Filling works:	"	74,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	230,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.23 - Cost Figure for Blockwall on Dredged Foundation, 10 m Water Depth

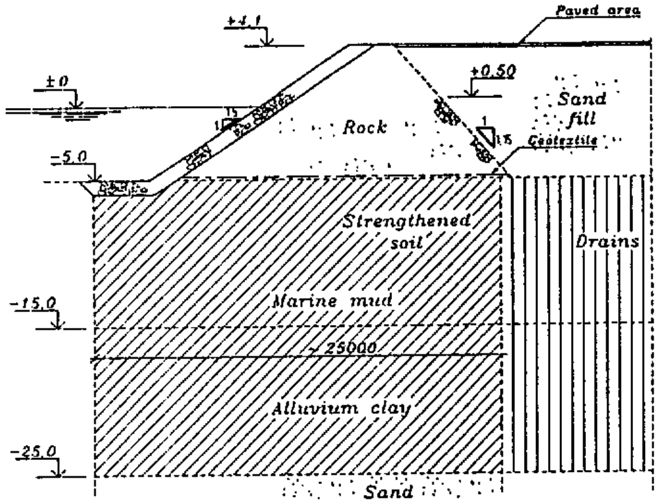


Comments:			
	General cost:	HK$	35,000,000
	Concrete works:	"	33,000,000
	Foundations works:	"	104,000,000
	Filling works:	"	83,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	269,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.24 - Cost Figure for Blockwall on Dredged Foundation, 15 m Water Depth

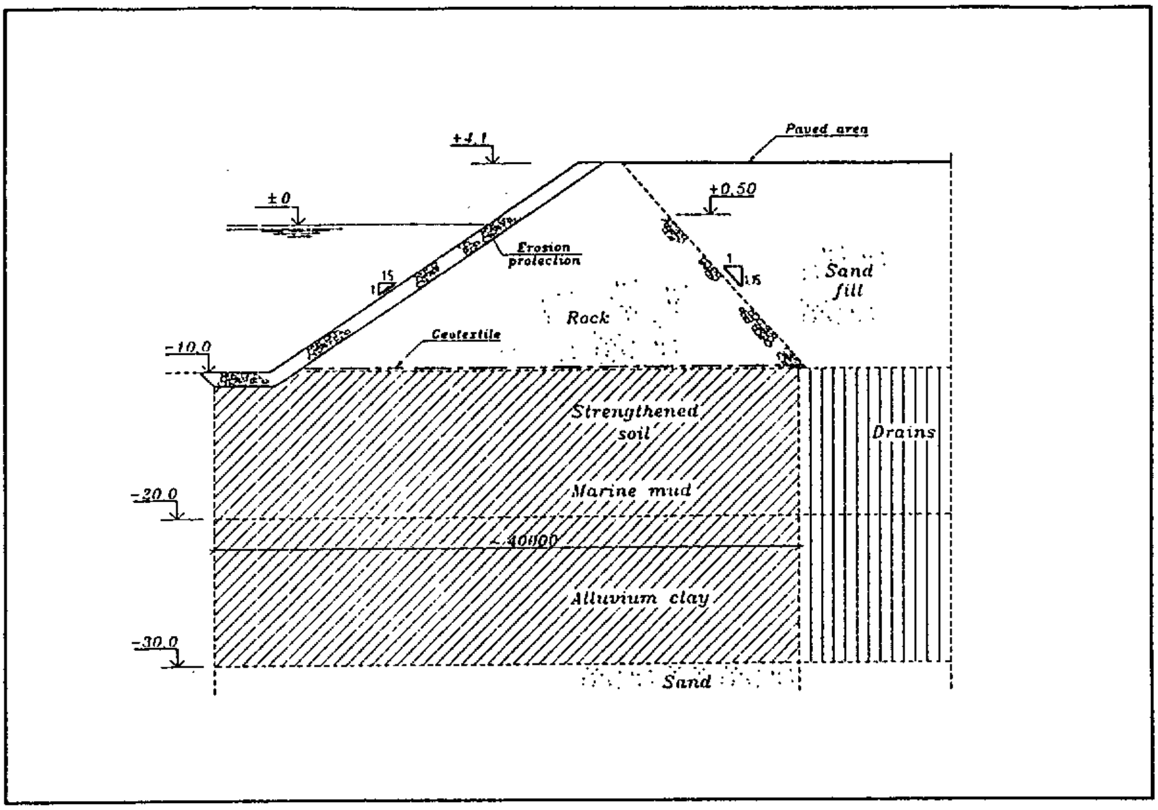


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	-
	Filling works:	"	45,000,000
	Drains:	"	9,000,000
	Ground improvements:	"	79,000,000
	SUM: 1000 m	HK$	169,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.25 - Cost Figure for Sloping Wall, Improved Soil, 5 m Water Depth

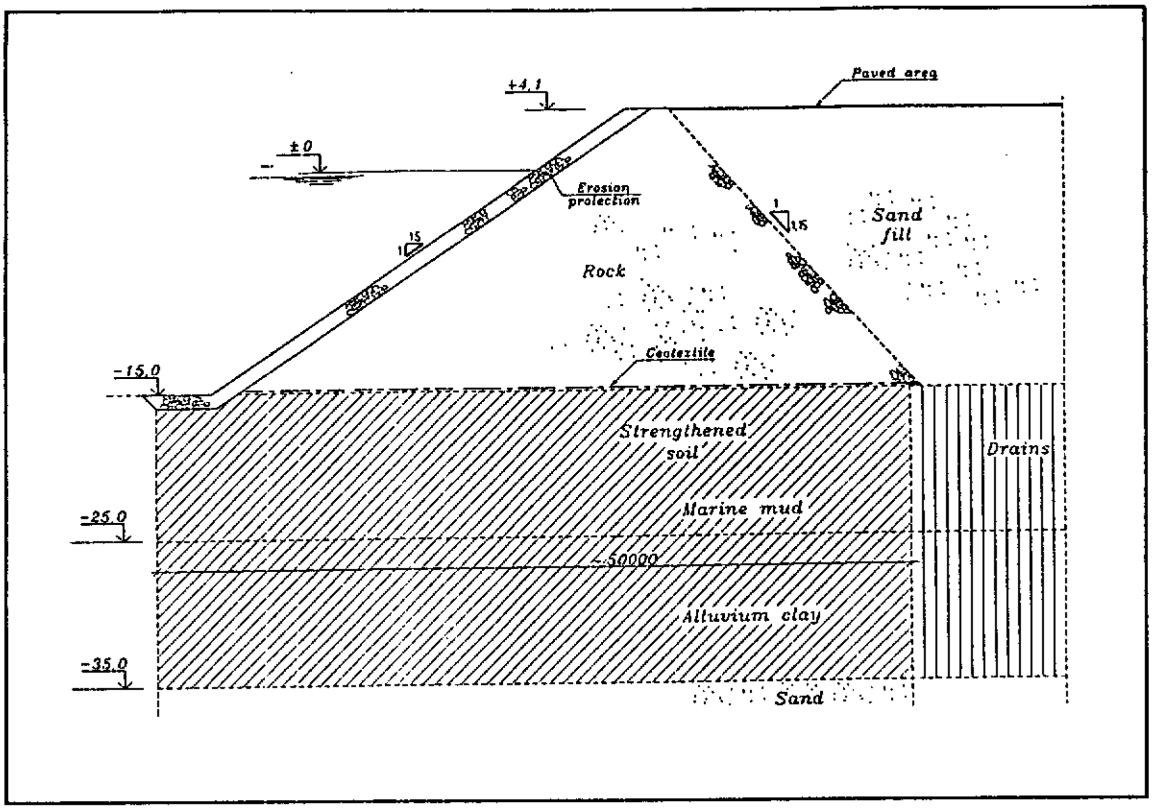


Comments:			
Cost effective, technically flexible, well known technology, sea wall free of settlements	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	-
	Filling works:	"	61,000,000
	Drains:	"	5,000,000
	Ground improvements:	"	118,000,000
	SUM: 1000 m	HK$	220,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.26 - Cost Figure for Sloping Wall, Improved Soil, 10 m Water Depth

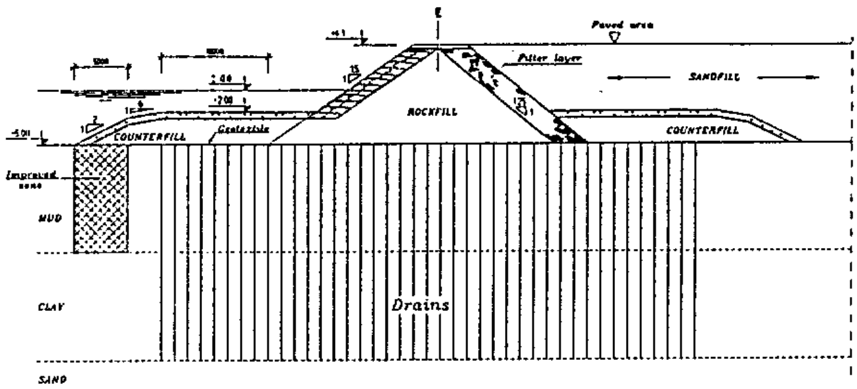


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	-
	Filling works:	"	82,000,000
	Drains:	"	1,000,000
	Ground improvements:	"	147,000,000
	SUM: 1000 m	HK$	266,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.27 - Cost Figure for Sloping Wall, Improved Soil, 15 m Water Depth



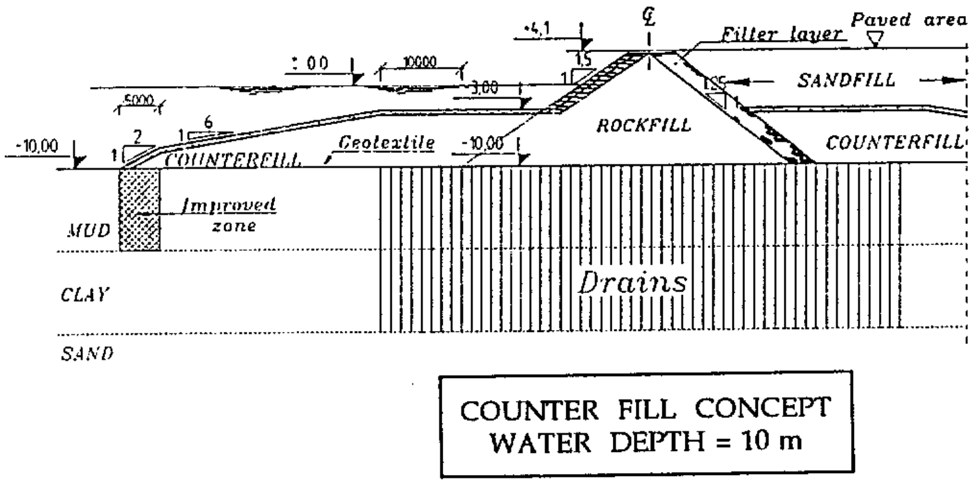
COUNTER FILL CONCEPT
WATER DEPTH = 5 m

Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	3,000,000
	Filling works:	"	49,000,000
	Drains:	"	17,000,000
	Ground improvements:	"	15,000,000
	SUM: 1000 m	HK$	120,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.28 - Cost Figure for Sloping Wall, Fill with a Counterfill of Very Gentle Slope in Front, 5 m Water Depth



Comments:

Technically flexible, cost effective, relatively long construction phase

General cost:	HK$	36,000,000
Concrete works:	"	-
Foundations works:	"	5,000,000
Filling works:	"	103,000,000
Drains:	"	26,000,000
Ground improvements:	"	15,000,000
SUM: 1000 m	HK$	185,000,000

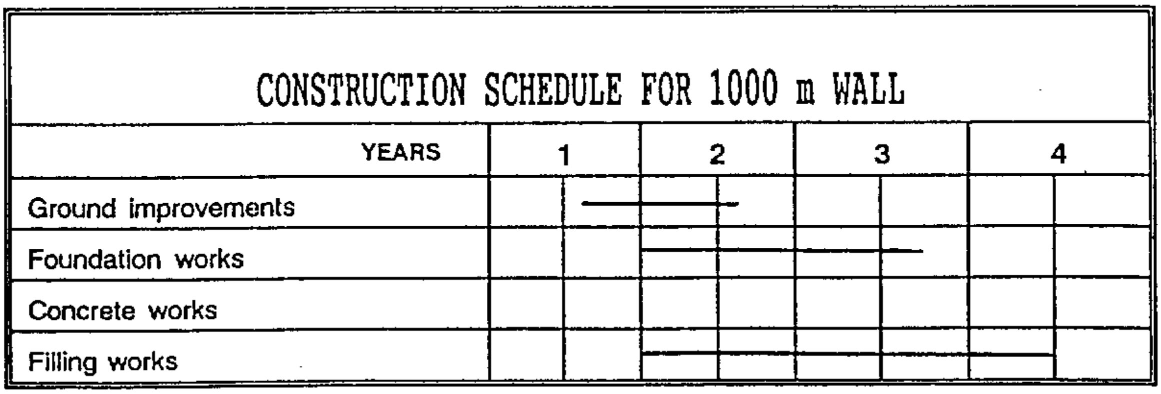
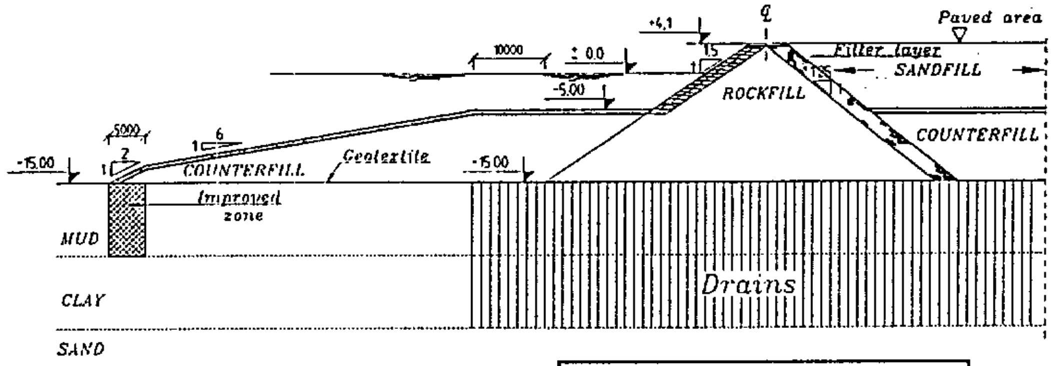


Figure 4.29 - Cost Figure for Sloping Wall, Fill with a Counterfill of Very Gentle Slope in Front, 10 m Water Depth



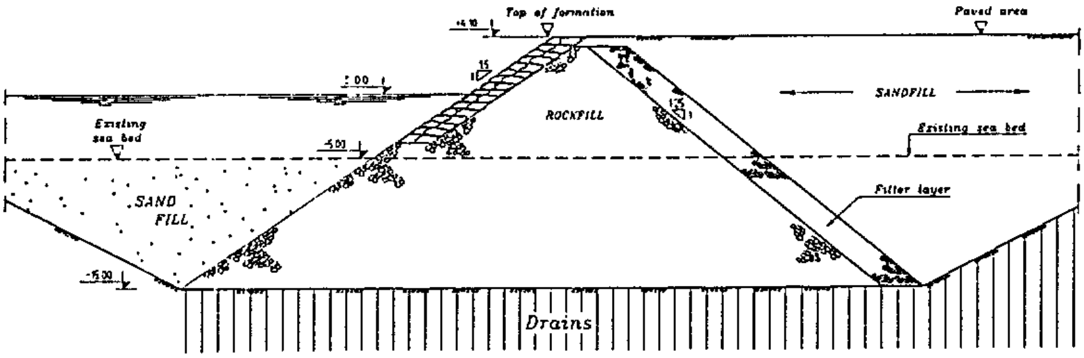
COUNTER FILL CONCEPT
WATER DEPTH = 15 m

Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	7,000,000
	Filling works:	"	173,000,000
	Drains:	"	26,000,000
	Ground improvements:	"	15,000,000
	SUM: 1000 m	HK$	257,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.30 - Cost Figure for Sloping Wall, Fill with a Counterfill of Very Gentle Slope in Front, 15 m Water Depth

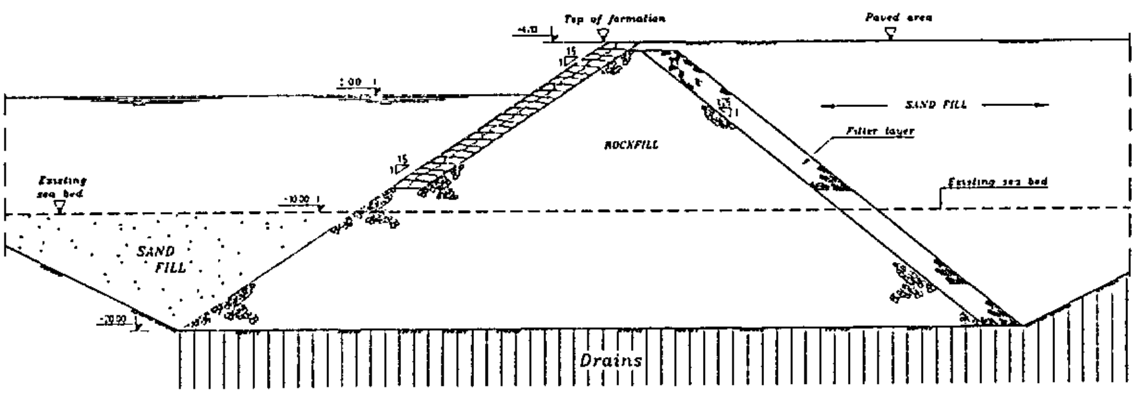


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	15,000,000
	Filling works:	"	98,000,000
	Drains:	"	15,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	164,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1	2	3	4
Ground improvements				
Foundation works				
Concrete works				
Filling works				

Figure 4.31 - Cost Figure for Sloping Wall, Dredged Trench, 5 m Water Depth

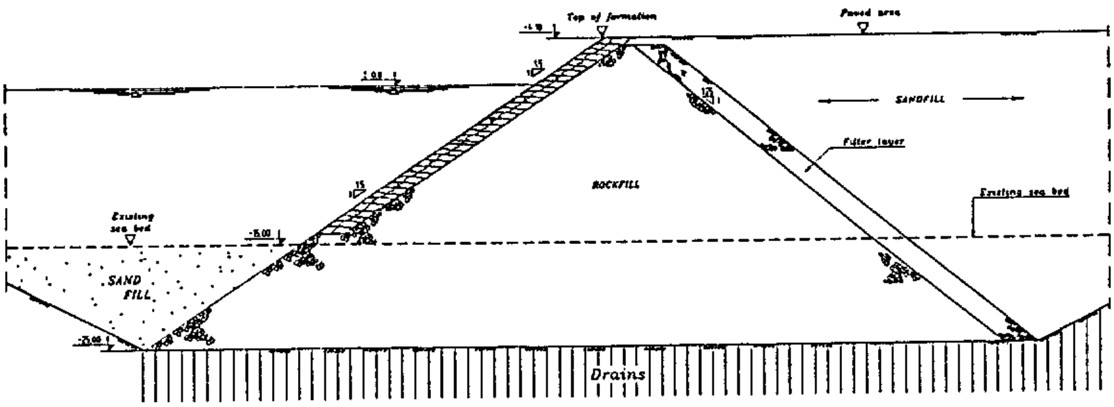


Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	18,000,000
	Filling works:	"	154,000,000
	Drains:	"	14,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	222,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1		2		3		4	
Ground improvements								
Foundation works	—	—	—	—	—			
Concrete works								
Filling works		—	—	—	—	—		

Figure 4.32 - Cost Figure for Sloping Wall, Dredged Trench, 10 m Water Depth



Comments:			
	General cost:	HK$	36,000,000
	Concrete works:	"	-
	Foundations works:	"	22,000,000
	Filling works:	"	206,000,000
	Drains:	"	12,000,000
	Ground improvements:	"	-
	SUM: 1000 m	HK$	276,000,000

CONSTRUCTION SCHEDULE FOR 1000 m WALL

YEARS	1		2		3		4	
Ground improvements								
Foundation works								
Concrete works								
Filling works								

Figure 4.33 - Cost Figure for Sloping Wall, Dredged Trench, 15 m Water Depth

APPENDIX A

DESIGN SOIL PARAMETERS

A.1 DESIGN SOIL PARAMETERS

The base case design soil parameters are given in Table 2.1. The following comments can be made to the different values:

- *Thickness of each stratum*

The soil stratigraphy was preliminary estimated in the NGI proposal for this study. A review of the 16 PCPTs performed at the Chek Lap Kok Airport site (GCL, 1991) generally shows a large scatter in soil layering, but as an average the depths given in the proposal seems reasonable and are therefore kept unchanged.

- *Total unit weight,* γ_{tot}

A total unit weight of 15 kN/m^3 in the soft top clay ("mud") is based on background material provided by GEO (1991). Values of 18 kN/m^3 in the silty clay below and 19 kN/m^3 in the deeper strata are estimated on the basis of experience.

- *Undrained shear strength,* s_u

The average normalized undrained shear strength in the soft clay, $s_u/p_o' = 0.35$ was provided by GEO (1991). s_u is taken as the uncorrected field vane measurements. The undrained vane shear strength is then corrected by applying methods outlined by Aas (1986) to arrive at the values given in Table 2.1.

An undrained triaxial compression shear strength of 40 kPa in the silty clay was assumed in the NGI proposal, based on suggestions by GEO in letter to NGI dated 19 November 1991. Interpretation of shear strengths from the PCPTs received has given values of s_u^C ranging from 15 kPa to 100 kPa in this stratum. The interpretation seems to confirm the preliminary assumption to be a conservative value, and $s_u^C = 40$ kPa is therefore kept unchanged. s_u^{DSS} and s_u^E as given in Table 2.1, are based on the same anisotrophy factors as for the top soft clay.

The undrained shear strengths in the clay layer in Stratum IV (25-30 m depth) are interpreted from PCPTs to be in the range 45 kPa to 120 kPa, and design values are tentatively taken as 50% higher than the strengths of Stratum II.

- *Dimensionless modulus number, m*

The modulus number is taken from Janbu (1970) based on an average water content of 90% in the top soft clay, and 40-50% in the clay layers below.

- *Coefficient of vertical consolidation,* c_v

Values of c_v ranging from 1 to 20 m^2/year in the top soft clay was provided by GEO

(1991), with an average of 4 m^2/year.

The dissipation tests performed in connection with the PCPTs (GCL, 1991) have been interpreted to obtain coefficients of horizontal consolidation, c_h, following guidelines outlined by Robertsen et al. (1990). The coefficients of vertical consolidation is then taken as 25-50% of the values of c_h, as may be reasonable for a layered soil.

- *Constrained deformation modulus, M*

Lunne and Christophersen (1983) have suggested the following relationship between cone tip resistance, q_c, and M for 10 MPa $< q_c <$ 50 MPa:

$$M = (2 \cdot q_c + 20)$$

Taking average q_c in the two sand strata as 15 MPa, we obtain M = 50 MPa.

- *Friction angle, ϕ'*

GEO (1991) has provided information stating that ϕ' in the top soft clay is 27°-29°. An average of $\phi' = 28°$ is therefore suggested used.

Schmertmann (1976) has given a relationship between q_c, p_o', D_r (relative density) and peak drained friction angle. Applying this theory to the PCPT data available, average values of 40° and 38° are obtained for the upper and lower sand strata, respectively.

- *Cohesion, c'*

The cohesion (in terms of effective stresses) is assumed to be zero in all strata.

The friction angle of the fill material has been taken as $\phi = 37°$, and the total unit weight has been taken as $\gamma = 20$ kN/m³.

When band drains are installed with a spacing of 1.5m, the soft marine mud and the underlaying alluvium clay will be fully consolidated under the weight of the fill after 1-2 years, as outlined in Section 3.2. The fully consolidated mud and clay are then assumed to have an average undrained shear strength of $0.28\sigma_v'$, where σ_v' is the vertical effective stress at a given depth.

Section 3.2 indicate a conservative undrained shear strength in the DCM improved panels of 800 kPa. Assuming that about 25 % of the marine mud and possibly the underlaying alluvium clay will be improved, an average undrained shear strength in the improved zones of 200 kPa is obtained.

A.2 REFERENCES

Gammon Construction Limited (GCL) (1991)
Site investigation report, Job. No. 1076, Chek Lap Kok Replacement Airport.

Geotechnical Engineering Office (GEO) (1991)
Hong Kong - Foundation Conditions for Seawalls and Breakwaters, Technical note given to K. Karlsrud during meetings in Hong Kong in October 1991.

Janbu, N. (1970)
Grunnlag i geoteknikk. Tapir Forlag, Trondheim.

Lunne, T., H.P. Christophersen and T.I. Tjelta (1985)
Engineering use of piezocone data in North Sea clays. International Conference on Soil Mechanics and Foundation Engineering, 11. San Francisco 1985. Proc. Vol. 2, pp. 907-912.

Robertson, P.K. (1990)
Soil Classification Using the Cone Penetration Test, Canadian Geotechnical Journal, Vol. 27, No. 1, pp. 151-158.

Schmertmann, J.H (1976)
An updated correlation between relative density, D_r and Fugro-type electric cone bearing q_c Waterways Experiment Station, Vicksburg, Miss. Contract report, DACW 39-76-M 6646, 145 p.

Aas, G., S. Lacasse, T. Lunne and K. Høeg (1986)
Use of in situ tests for foundation design in clay, ASCE Spec. Conf. IN SITU '86, Blacksburg, Virginia, pp. 1-30.

APPENDIX B

WAVE LOADS

B.1 WAVE LOADS

The significant wave height for a wave with a return period of three years within the Hong Kong Harbour area was taken as 3 m, based on information by GEO (1991).

The Civil Engineering Manual (CEM), Section VII.2.4.12.3, gives a recommended ratio between significant wave height and maximum wave height during a storm of 1.9.

Further, CEM, Section VII.2.4.12.2, states that the design wave parameters for the assessment of wave loads should be taken as the height and period of the average maximum incident wave having a return period of 100 years.

Experience from the North Sea has given a ratio between the max. wave height for waves with a return period of 100 years and 3 years of about 1.25. This ratio is also assumed to be valid for Hong Kong conditions.

This approach has given a 100 year design wave of:

$$H_{inc} = 3 \cdot 1.9 \cdot 1.25 = 7.2 \text{ m}$$

The period of a 7.2 m wave is normally 10 - 14 s, and $T = 12$ s is therefore chosen.

The wave length of the 100 year design wave can be found as:

$$L = (g \cdot T^2/2 \cdot \pi) \cdot \tanh(2 \cdot \pi \cdot d/L)$$

British Standard BS 6349 : Part 1 : 1984, Section 39.4.2, is then applied to find the Clapotis set-up and the maximum and minimum wave pressures on a vertical seawall.

$$H_{oc} = (\pi \cdot H_{inc}^2/L) \cdot \coth(2 \cdot \pi \cdot d/L)$$

$$P_{max} = \rho \cdot g \cdot d + \rho \cdot g \cdot H_{inc}/\cosh(2 \cdot \pi \cdot d/L)$$

$$P_{min} = \rho \cdot g \cdot d - \rho \cdot g \cdot H_{inc}/\cosh(2 \cdot \pi \cdot d/L)$$

The following values of wave lengths and pressures are found :

Water depth, d (m)	Wave length, L (m)	H_{oc} (m)	P_{max} (kPa)	P_{min} (kPa)
5	82.1	4.93	115	0
10	113.3	2.32	159	37
15	135.4	1.44	203	91

B.2 REFERENCES

GEO (1991)
Hong Kong - Foundation Conditions for Seawalls and Breakwaters Technical note given to K. Karlsrud during meetings in Hong Kong in October 1991.

APPENDIX C

GROUND STRENGTH IMPROVEMENT

(This Appendix is available for viewing in the Civil Engineering Library)

APPENDIX D

DESIGN ANALYSES

(This Appendix is available for viewing in the Civil Engineering Library)

APPENDIX E

DETAILED COST ESTIMATES

(This Appendix is available for viewing in the Civil Engineering Library)