Port Engineers Department
Port of Cape Town

AN INVESTIGATION WITH RECOMMENDATIONS OF THE PRESENT CONDITIONS SURROUNDING THE CAISSON EXTENSION TO THE MAIN BREAKWATER AT THE PORT OF CAPE TOWN.

JUNE 1997

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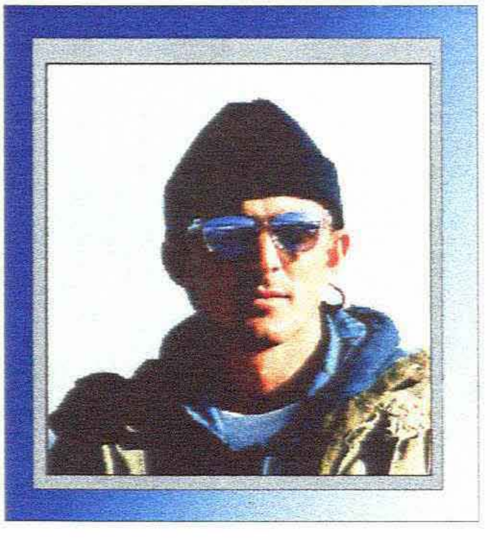
Deon Willem Lourens is a Engineering Technician employed by Portnet. He has been with Portnet since 1989. He was born in the Transvaal in 1968 and has vowed never to return there, hence his interest in Port Engineering. This thesis will complete his Masters Diploma course on a interesting note.



DECLARATION

I, the undersigned declare that the work contained in this thesis is my own work and has not previously been submitted to any University or Technicon for the purpose of obtaining any qualification.

SIGNATURE: ________ DATE: 15/10/97

ACKNOWLEDGEMENTS

I would like to thank the following individuals and organisations for their support and assistance with this study:

- Portnet, for the use of their equipment, financial assistance and information.

- Council for scientific and industrial research (CSIR), in particular Mr H Moes and Mr D Phelp for all the assistance and information provided.

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- Dr Clifford for his guidance.

- For my Father in Heaven who gave me the energy and endurance to complete this study.

- For my wife Karin, for her support and encouragement.

- My Doctor, for my stomach ulcer treatment.

EXECUTIVE SUMMARY

The idea for this study occurred when movement of the caisson extension to the breakwater was observed.

The major concern was, what would happen if the caisson breakwater extension failed? What would the financial implications be to the port?

The CSIR have carried out a number of studies with regards to the safety of the structure. The consequences of caisson failure and the possible effects on the port were however not investigated.

When it was determined that settlement was taking place, information concerning the condition of the caisson structure and factors influencing the structure were gathered. Investigations on the following were done:

(a) Extent of caisson settlement.

(b) Sediment movement around the structure.

(c) Foundation condition.

(d) Wave impacts of long and short period waves on the Ben Schoeman Dock in the event of caisson failure. (Refraction and diffraction).

(e) Financial implications due to possible container operation downtime at Ben Schoeman Dock in the event of caisson failure.

(f) The tourist potential of the structure.

The conclusion reached in this study was that the Ben Schoeman Dock would not be adversely effected if partial or complete failure of the main breakwater should take place. One could even question the length of the extension and whether it was actually required.

The recommendations of this study would be as follows follows:

- Maintain the caisson extension in good condition as it will be important for possible future extensions to the port.

- Develop the breakwater as it is an asset which has potential for tourism to Cape Town.

OPENING STATEMENT

Various studies worldwide have been performed with reference to caisson breakwaters over the years. See examples in chapter 2 A (PIANC 1992 chapter 113 and 124). It was however difficult to find information that would assist us with the Port of Cape Town breakwater. All breakwater studies are unique with regard to their own topographic and sea conditions experienced around that breakwater. The type of caisson constructions studied also varies considerably.

The CSIR have done investigations with regard to the Main breakwater at the Port of Cape Town. These studies concentrated specifically on the stability of the caissons and the possibility of caisson failure (CSIR C/SEA 8508). The CSIR literature were the major source used to gather information assisting this literature. All wave data figures used were also gathered from CSIR reports.

Information collected on the topography of the surrounding area around the breakwater were obtained from the CSIR and Portnet. Pieter Goldy (1993) was the source for information with reference to the stability of the Table Bay coastline.

The Shore Protection Manual (U.S.Army 1977) was the source to obtain theoretical information and charts for the diffraction of waves around structures.

Below is an humorous extract which will give the reader of this document a good idea of the diverse opinion that people have on the subject of breakwaters. The extract refers to rubble mound breakwaters but the underlying message can equally be applied to any type of breakwater.

What is a **rubble mount breakwater**? It is in fact all things to all men.

To the chairman of a **Port Authority** it is a large heap of rock dumped, at great expense, in the sea to protect an area of water, which he, the Client, wants to use at all times and in all weather because he has sold his harbour to shipping companies as providing just that. He sometimes fails to understand why engineers and scientists contrive to make the design sound so difficult and sophisticated because even he, as a layman, knows that breakwaters have been around for hundreds of years. Why therefore, he asks himself, do these damn things still fall apart and who is this chap Hudson who keeps on cropping up in every conversation?

To a **Mariner** it is a navigational hazard which on occasions or when designed by an inexperienced engineer, somehow seems to make the seas greater inside the harbour than out. He has been told that it is rather like an iceberg with 9/10 th of it under water - somewhere - so he gives it a wide berth and ends up driving his ship on to the Lee breakwater which, of course, was put there for just that purpose.

To a **Scientist** it is a random collection of individual particles having no cohesion and subject to random loading. It is thus fair game for all sorts of his favourite statistical analysis, probability distributions or even joint probability distributions - and in 2 and 3 dimensions. The Scientist usually considers that unless a harbour engineer has at least 10 years of wind and wave recording at this site, he is a fool ever to take the job in the first place.

To a **Contractor** a rubble mound structure is just one big "muckshifting" job of pouring endless loads of rubble into the sea.

He is cynically amused by the specification which calls for tolerances which he knows cannot be achieved but is comforted by the thought that once in place can rarely be properly inspected and measured. He views the scientist with deep suspicion and wishes he would, just occasionally, leave the "rarefied atmosphere" of his laboratory and find out how it is really done. He has heard that scientists are somehow concerned with the design which he believes explains all his problems.

To the **Structural Engineer** it is a stunningly crude structure, which as it is not based on a Code of Practice is therefore despicable and has no right to stand up anyway.

To the **Architect** who has designed a perfectly proportioned yacht marina development it is an aesthetic disaster which usually ends up going green and smelling horribly.

To the **Harbour Engineer** it is rather like his mother-in-law.....

"Great to get away from but occupying a unique place in his heart."

(With acknowledgement to I.W. Stickland, Breakwaters, 1984)

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Heavy seas during a Cape storm.

Waves pounding the breakwater wall, the spray reaching a height of about 20 meters.

Photograph group - A

1. PREAMBLE

1.A. INTRODUCTION

At the time of writing this thesis the student was employed by Portnet in the Port of Cape Town. This study developed out of the concern for the Port due to deterioration of the caisson extension of the Main breakwater.

The function of the Main breakwater is to enclose and provide a stable entrance, providing additional wave or sediment extrusion protection to the Port.

This study investigates the following:

1. The present condition of the caisson extension.

2. The possible causes of deterioration of the structure.

3. The implications to the Port in the event of caisson failure.

4. Possible repairs and maintenance.

5. The tourist potential of the structure.

This thesis will not present an in-depth overview of all the factors involved which effect the performance of the caisson extension. It will however summarise previous studies and apply their conclusions to the present condition of the caisson structure. Having this information available, will enable Portnet to make better decisions with regards to any future extensions to the Port as well as future maintenance requirements.

1.A.1 History

When the early seafarers came to the Southern Tip of Africa, little protection was provided for ships against the bad weather conditions encountered there. Today Harbours and marine structures are used by shipping for trade and other activities.

The most important factor in the establishment of a harbour is the creating of a barrier to provide protection against severe weather conditions. These barriers are called breakwaters.

The history of the Breakwater in the Port of Cape Town dates back to 1860. Prince Alfred, second son of Queen Victoria, tipped the first truck of stone on September 17, thus officially starting the construction of the Table Bay Breakwater. (Peter Newall 1993, Cape Town Harbour, 1652 to the present)

The first part of the Breakwater was constructed out of stone dumped from railway wagons, to form a natural slope against the force of the waves. This part of the Breakwater was completed after 10 years of construction. The Breakwater was 570 metres in length.

Today the Table Bay Breakwater is nearly 900 metres in length, constructed mainly of rock, blockwork and the caisson extension. The 130m extension consists of seven caissons and will be the major focus of this investigation.

The last breakwater extension was constructed in 1972 as a vertical caisson type breakwater. This extension was necessary for the protection of the proposed Ben Schoeman Dock. The dock would provide the infrastructure to establish, amongst others, container handling facilities in the Port of Cape Town. This extension to the Breakwater consists of seven caissons which were constructed in the Sturrock graving dock. They were floated into position and sunk on a stone bed foundation.

1.A.2 Construction of the caisson breakwater

The area occupied by the proposed extension was dredged to remove all unsuitable material and obtain the necessary depths. A stone foundation layer was constructed in which the seven caissons were bedded. A polypropylene blanket of the required width was placed in the dredged trench prior to the placing of the stone. Graded stone (1 to 450 kg) was then placed as per calculated profiles.

After the stone foundation was placed a screed layer of small stone was placed and levelled with a bed leveller.

The individual caissons were constructed in the Sturrock dry dock where they were floated out and then towed to site. The caissons were accurately positioned by the use of barges and cables. They were then carefully sunk into position. Care had to be taken not to disturb the stone bed when sinking the caissons.

After each caisson settled they were filled with sand. Grout socks were inserted in the joints between the caissons. (See fig 1)

The capping was cast on top of the structure including rubber seals placed between the concrete sections. (See fig1) Concrete blocks were placed at the caisson toe, to provide additional protection to the foundation.
(See photograph group B)

1.B VISIBLE DETERIORATION

Rubber membranes were placed by the contractor between individual caissons to facilitate with the monolithic abilities of the structure. (See fig 1) During an inspection in 1993, the author noticed that some deterioration of caisson rubber joints was taking place. The concrete where the rubber membrane was placed, was deforming and pushing the membrane out of the slot. This observation confirmed that there was movement between the individual caissons. A level survey was conducted and compared with similar measurements taken prior to the winter storms. It was noticed that there was movement between caissons of up to 15mm since the previous survey (4 months earlier). These results confirmed the author's suspicion that movement or some form of settlement is taking place between caissons.

The caissons as already mentioned, serve an important function in that they protect the Ben Schoeman dock. It was therefore felt that a comprehensive investigation should be carried out to highlight the importance of the structure and to conduct proper maintenance planning concerning the future of the caisson extension.

In this investigation the author intends:

1. To determine the present conditions surrounding the caisson movement.

2. To investigate the factors affecting the structure and what their influence is. (For example: movement, foundation damage and structural instability).

The author further intends to predict the possible consequences of structure failure and also to determine what measures are to be taken to extend the design life of the structure.

1.C COMPANY INVOLVEMENT

The involvement of Portnet in any investigation concerning breakwater structures is important. Since its construction investigations of the caisson extension have been carried out by the Council for Scientific and Industrial Research (CSIR).

Portnet is involved in the maintenance of marine structures. It supports research which may improve methods of maintaining such structures.

The calculation of the safety factors of the caisson structure against sliding and overturning was performed by the CSIR (1985) C/SEA 8508. The results thereof will be discussed in Chapter 3.

1.D ACCESS TO INFORMATION

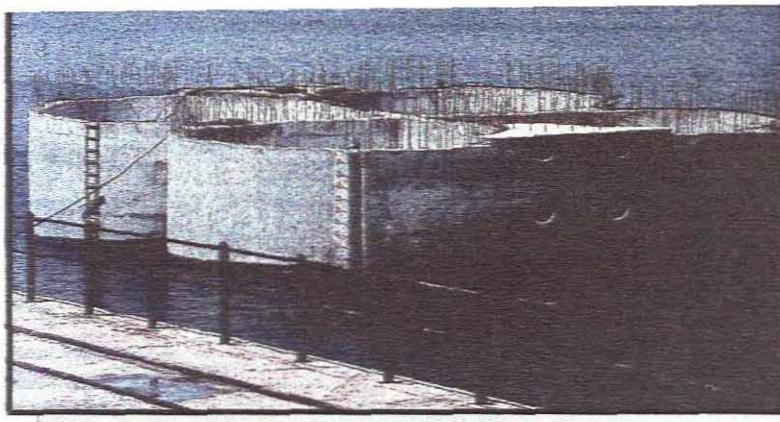
The information obtained for this thesis was provided by the library services of Portnet and the CSIR.

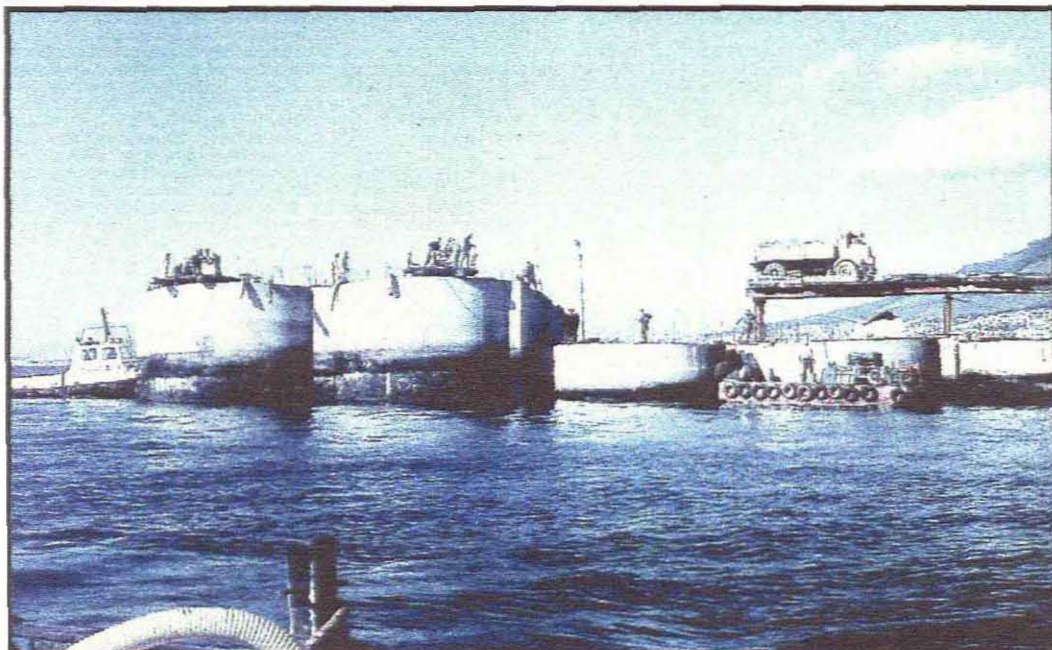
1.E ENVIRONMENTAL DISASTER

One of the important factors for compiling this thesis is to make those concerned, aware of the possible implications to the Port of Cape Town in the event of the failure of the structure. This includes environmental and economic implications.

What are the possible environmental effects on the Port and the surrounding areas in case of caisson failure? How severe would these effects be? Would Portnet be prepared for the consequences of possible failure of the structure and the environmental effects that might follow as a result of failure?

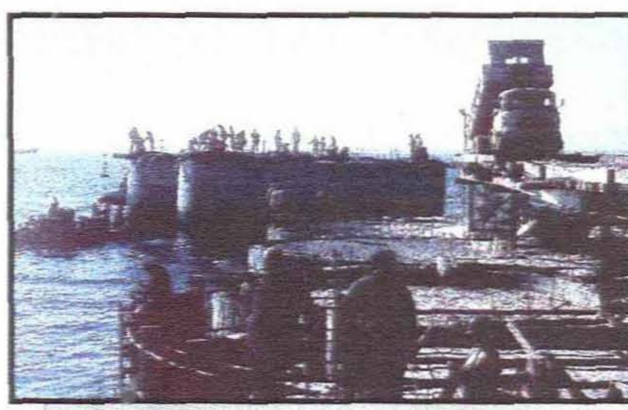
A caisson ready to be placed into position



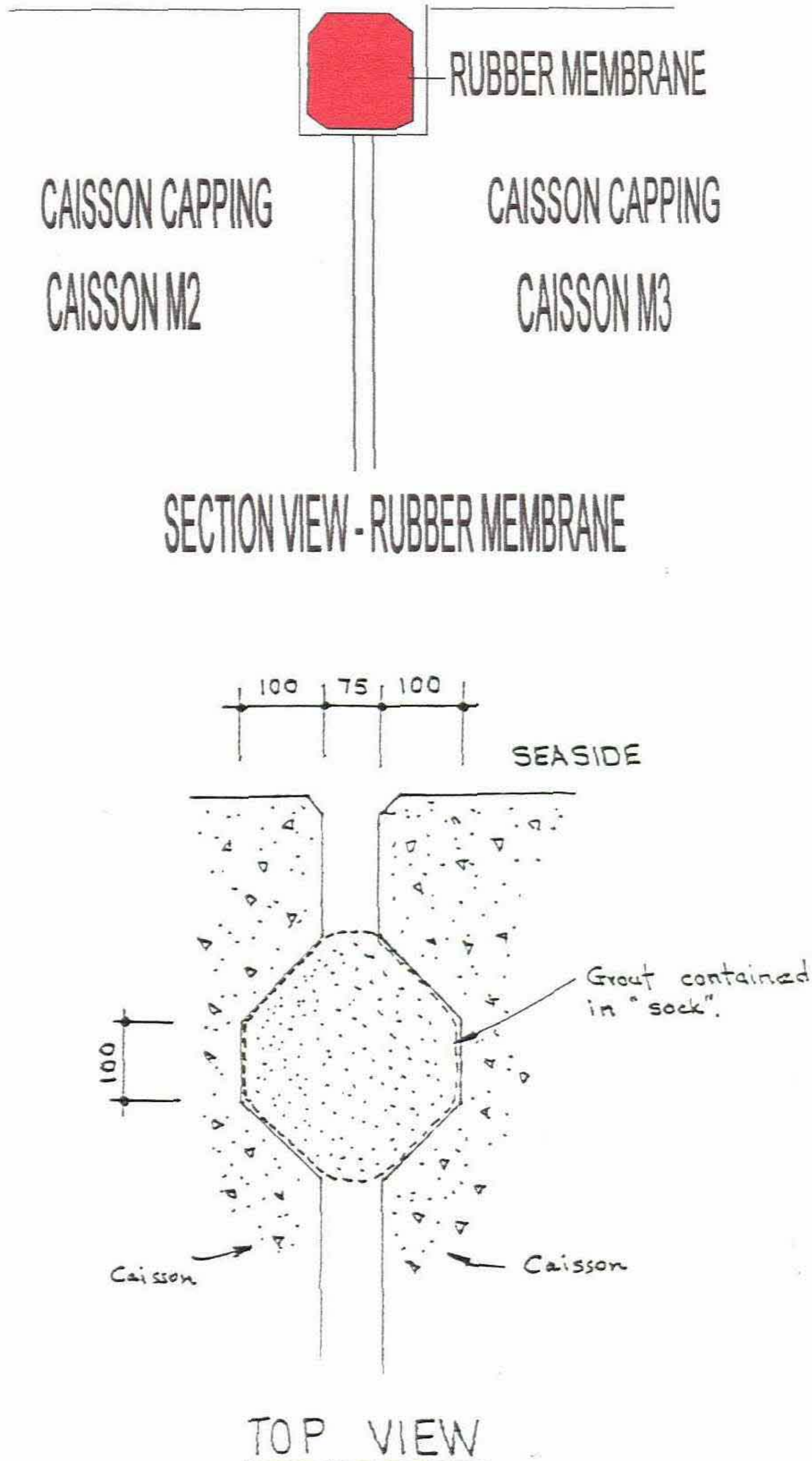


The caisson sunk into position

A temporary bridge was used by trucks to dump the sand into the caissons



Photograph group - B



Rubber membrane and Grout sock detail

Fig 1

2. TESTS TO DETERMINE MOVEMENT

2.A MEASURING DEVICES

2.A.1. Introduction

The first form of deterioration was caused by movement and/or settlement of the structure. The following questions will be addressed: Why is there movement of the structure? What factors contribute to the movement of the structure?

The first step to follow with regard to movement is to determine the following:

(a) What type of movement is taking place (vibration or "rocking motion").

(b) What pattern of movement is present between the individual caissons.

(c) Which caissons are affected?

The first difficulty encountered was to measure and determine what type of motion is taking place between the individual caissons.

Various methods and instruments were used, but none were found to be suitable under the conditions experienced.

2.A.2. Conditions experienced

(a) The breakwater structure is exposed to major wave action and inclement weather. This complicates the fixing of any form of measuring device on the structure.

(b) Direct or radio communication with these devices are vital if any useful information is to be gathered. In the present case cost of such communication would not be justified when weighed against the value of the information.

(c) Vandalism and theft are also real problems and must be considered when any sort of instrumentation is placed on an open structure accessible to the public.

2.A.3 Alternative analysing methods (1)

Various methods of analysing and designing caisson breakwater structures to determine the safety factors against sliding and overturning are available and used worldwide. New methods are still being developed to gain a better understanding of the various elements involved in breakwater design. A good example of new methods is study that was done by the University of Germany (Frazius Institut). They compared some physical quantities or relationships obtained from the measurements of physical model tests to those obtained by computation.

The alternative analysing methods are included to give a background of the methods use around the world in the design of caisson breakwaters. The conclusions made with reference to the alternative methods will give us a better understanding of the factors involved in caisson design. It also compares the results obtained in theoretical calculation with those obtained using model studies.

2.A.3.1. Dual cylindrical caissons

A full scale experiment was performed with a new type of caisson design, the dual cylindrical caisson. This field experiments were

Tanimoto, H. Endoh, S. Takahashi from Japan (ICCE 1992, chapter 124).

The caisson has dual cylindrical walls with the outer wall being perforated with a impermeable centre core. (see fig 2.1)

Various prototype experiments were performed before the full scale prototypes were constructed and tested. The advantages of this design are in its low reflection abilities and its high stability.

Three caissons were used for the field test. One was designed for a wave height with a mean return period of one year while the other two were designed for wave heights with a fifty year return period.

The aim of this experiment was to measure the displacements of the caisson and the wave pressures due to sliding in high wave conditions. This was accomplished on 17 February, 1991.

The caisson designed for a wave height return period of one year slid out of position.

After the field test of sliding was completed, a 1/21 scale model was used to reproduce the field conditions experienced (ICCE 1992, chapter 124).

2.A.3.2. Conclusion Remarks

The conclusions made in this study were as follows:

1. The sliding of the caisson can be determined by the following: its safety factor using the measured wave forces, weight of the caisson and friction factor in the field and in the laboratory.

2. The measured wave forces having an effect on the caisson, were similar to the wave forces calculated. The sliding of the caisson experienced in the field can therefore be judged by using the safety factor of the calculated wave forces.

2.A.4. Alternative analysing methods (2)

2.A.4.1. Dynamic response of caisson breakwaters

The main objectives of this study, the impact loading and dynamic response of caisson breakwaters, were to establish and develop guidelines as well as the evaluation of counter-measures for increasing the overall stability of caisson structures and their foundations.
(ICCE 1992, chapter 113)

The underlying philosophy of this investigation was to show that a caisson breakwater is a problem in dynamics. This cannot be treated as a simple static problem. For this purpose a hydraulic test and pendulum test were performed on caisson breakwaters supplemented by a dynamic analysis of the structure.

2.A.4.2. Setup and test conditions

The hydraulic model test was performed in the Hanover (GWK) wave flume on this specific type of caisson breakwater, with a rubble mound on a sand foundation. (ICCE 1992, chapter 113)(see fig 2.2)

The following measurements were carried out simultaneously:

1. Incident and reflected waves.
2. Impact pressure on the caisson front.
3. Uplift pressure.

4. Wave induced pore-water pressure in the foundation.
5. Total wave induced stress in the sand layer.
6. Dynamic response of the caisson.
7. Total forces.

2.A.4.3. The pendulum test

The pendulum test was a supplement to the hydraulic model test and used the same caisson/foundation in different water depths. (see fig 2.2)

The objective of this test was to determine the hydronamic mass, the damping and the subgrade reaction coefficients to be taken into account in the dynamic analysis of the caisson/foundation system. The test was conducted under dry conditions and wet conditions at different water depths.

2.A.4.4. Summary of the results and conclusions

1. The suggestion commonly made that the effects of impacts on the stability of caisson breakwaters is not significant, could not be substantiated in this study.

2. The characteristics of the impact load are governed by the shape of the wave breaking against the structure. The most critical loads found were those in double peak impact forces which are induced by plunging breakers against the structure.

3. Dynamic uplift pressures caused by wave impacts are not linearly distributed and appear to be important for the dynamic stability analysis of the structure.

4. Impulsive loads due to breaking waves cause free damped nonlinear oscillations of the structure foundation. These nonlinear oscillations are probably due to the plastic deformations of the foundation as well as the hydrodynamic mass and the geodynamic mass. The latter two masses increase with the increase in the amplitudes of the oscillations of the structure.

5. Sharp peak stresses of the total stress recorded are induced in the foundation. The oscillations are transmitted from the structure to the foundation. This is followed by smaller oscillations which correspond with the free-rocking oscillations of the structure.

6. From the total stresses recorded two types of stresses are found in the sand layer and rock foundation beneath the caisson structure. One is caused by the free oscillation of the structure following impact and the other is caused by the shock wave propagation in the soil foundation.

2.A.5. Summary

After considering the use of laser technology, radio transmitting and sensitive measuring devises the following conclusions were made.

1. The actual information required was the movement of each caisson in relation to the others in unfavourable wave conditions. Information of these movements, as well as the swell forces and wave heights influencing the movements of the structure would have had to be gathered simultaneously. This was not achieved.

2. It was concluded that the information to be gathered and the methods required were of a specialised nature thus requiring specialised research. The cost to obtain this information could also not be justified.

The questions posed with regards to the types of movement present in the caisson structure and which caissons are effected are still unanswered. The following assumption can however be made. Signs of settlement are present throughout the whole caisson structure, indicating that all the caissons are either settling or moving during adverse wave conditions. The type of movements present in the structure, vibration or rocking is still unknown. It is also known that all the caissons are affected by settlement which could be due to vibration and/or rocking motion.

Consequently, emphasis will now be placed on standard measuring methods to analyse the movement of the caissons.

2.B SOUNDINGS

2.B.1. Introduction

Sounding is the measurement of the vertical depth of the ocean floor, in this case to identify any build up, or erosion around the structure which may influence the stability of the structure.

Soundings around the caisson structure were carried out on 24 February 1994. (see fig 2.3)

The soundings profiles achieved on 27 February 1987 indicates a large depletion of sand on the outside of the breakwater compared with the soundings taken on February 1994. (see fig 2.4)

No large scale erosion of the foundation itself could be detected from the sounding or diving inspections.

2.B.2. Sand migration in Table Bay

Various studies in the last 50 years have been carried out primarily by the CSIR to investigate various aspects of the Table Bay coastline. The main focus of these investigations was to establish the effects of harbour extensions to the Port of Cape Town on the movement of sand, coastline stability and wave onslaught on the Table Bay coastline. It is known that the beaches at Hout Bay, Sandy Bay and Clifton are stable. The coastline in this area has a very steep slope, indicating that the Benguela current, which flows in a northerly direction does not influence the morphology existing in Table Bay. (P. Goldie 1993).

According to G. Rosental (1992) there is an overall circulation of sediment from offshore sources to the Bay. He further stated that the breakwater is reflecting sand destined for the beach at Roggebaai far offshore. If this is the case, regular changes to the profile of the sea bed at the Main breakwater would occur.

The extensions at Granger Bay and the reclamation work at the Victoria and Alfred Waterfront makes it very difficult to make any assumptions about the movement of sand at the Main Breakwater.

In a CSIR bathymetric and topographic survey report CSIR (1993) EMAS-C 94032, it was concluded that the sediment process has stabilised over the years 92/93 at the Table Bay Breakwater. What must however be taken into account is the lack of significant storm events during the monitoring period of this report. The local effects are caused by reclamation in and around the port, and the effects of sediments present in the stormwater run-off. (See Port development plans - fig 2.5 and fig 2.6).

There are however external factors present which influence the sand accretion in the Port.

These factors, in our case, are small and cause more discomfort in the Port than outside the Port. These factors must however be monitored for the effects they might have outside the breakwater.

The influence that sand accretion could have on the wave impacts around and against a breakwater structure could be critical. These effects will be investigated in chapter 3.

2.B.3. Summary

To predict the pattern of sediment movement in and around the area around the Table bay breakwater requires more time. With large scale developments present at Granger Bay and the area surrounding the Victoria & Alfred Waterfront, certain changes could still be experienced. (see fig 6.1 for V & AW future development plan).
It is therefore very unlikely that any accurate predictions could be made concerning this issue.

There is however no indication that erosion of the caisson foundation is taking place. The foundation slopes are stable and no damage is present.

Accretion and erosion of sand along the vertical section of the breakwater is constantly changing. Up to the present there has been no change in the wave patterns affecting the breakwater as a result of sand accretion.

In this study, the possibility of increased wave impacts on the breakwater due to sand movement can be safely ignored.

2.C SOIL CONDITIONS

The foundation of the caisson extension was constructed as follows:

(a) The area was dredged to remove unsuitable material. The material dredged was shale (see fig 2.7).

(b) After dredging was a polypropylene filter blanket was placed.

(c) One to four hundred and fifty kilogram rock was then dumped on site to the profile as shown on fig 2.8.

(d) A screed layer of small stone was placed as a final layer to allow for maximum contact area with the caisson foundation.

If settlement of the caisson structure is taking place, as was found, it could only be due to foundation settlement or damage.

The two factors involved would be:

(a) Localised stresses developing in the existing stone foundation due to excessive movement of the caissons resulting in localised settlement of the foundation.

(b) The forming of a slip circle in the existing soil due to failure thereof. The stone foundation will therefore settle deeper into the sea bed causing movement of the caisson extension.

The present rate of settlement is however still at a stage where no definite signs are present indicating the reasons for the settlement. Diving inspections have only revealed localised foundation failure caused by incorrect insertion of the grout socks of the caissons.

(See damage report: Annexure C)

2.D INSPECTIONS AROUND THE STRUCTURE

2.D.1. Below the water surface

Inspections of the caisson structure below water were carried out in the past and also for this investigation. Various defects and problem areas were found. The most serious problem encountered was the undermining of the foundation stone layer. This is as a result of the deterioration of the grout socks between the caissons.

2.D.2. Grout socks

The main purpose of the grout socks is to seal the joint between the caissons. The most common use is in quay walls, where the purpose of the grout sock is to prevent the leaching of sand from behind the structure.

In the case of the caisson breakwater the purpose of the grout sock is to prevent the flow of water between the caissons.

If a grout sock should fail, large quantities of water would move between the caissons. The amount of damage caused to the foundation of the caisson structure depends on the location of the breach in the grout sock. If the breach in the grout sock is located close to the foundation layer serious damage could take place.

Scouring of the foundation stone would occur and the resulting damage would in turn affect the stability of the structure. Definite decrease of the safety factors related to sliding and overturning would take place. If this undermining is not prevented, failure of the monolithic ability of the structure could take place.

The stability factors (sliding and overturning) are also greatly reduced when the stone bed is affected by erosion.

The effects of this erosion are discussed in chapter 3.

2.D.3. Above water surface inspection

My opinion is that the structural condition of the concrete capping is acceptable. This opinion is based on the general condition of the concrete, the absence of major cracks or obvious spalling.

The capping is approximately two metres thick and has a service tunnel to provide services to the end of the structure.

2.D.4. Alkali aggregate reaction

Definite alkali aggregate reaction is present in the concrete capping, causing the concrete to crack. (See photograph group C)

What is Alkali Aggregate reaction? This normally involves the formation of a gel at the aggregate-paste interface. This gel attracts water molecules causing high pressures to develop in the concrete. (Fulton 1993)

The reaction occurs when the following three conditions are satisfied simultaneously.

(a) Exposure to water.
(b) Reactive silica present in the aggregates.
(c) High sodium oxide content in the concrete.

It is important to reduce the possibility of this reaction occurring in concrete (Fulton, 1993) when structures are constructed:

(a) Avoid aggregates that are potentially reactive.
(b) Alkali levels must be kept within the safe zone.
(c) As partial replacement for cement, blend either 20% Fly ash, or 10% condensed silica fume, or 40% Ground granulated blast furnace slag in the concrete.

My opinion is that the consequences of the deterioration of the concrete mass capping on the Port side of the caisson breakwater is not critical, as the capping is approximately two metres thick and the concrete in a sound structural condition.

It is important to maintain the capping on the sea side of the caissons. The geometry of the breakwater structure itself is very significant in the determination of wave loading (Port Engineers Handbook, 1994)

The alkali aggregate reaction present can weaken the structure and the risk of capping failure.

It is important therefore to keep the concrete capping in good repair, especially on the sea side of the structure, as it has an influence on the stability of the structure.

Further facts regarding the concrete capping and the failure thereof can be found in chapter 3 of this investigation.

2.E INSTRUMENT MEASUREMENTS

2.E.1. Results of vertical movement

During the construction phase of the Caisson extension various levels were taken on the caissons. The levels taken during the construction phase were taken on the caisson corners and not on the brass pegs grouted into the concrete capping.

The results of these surveys were calculated relative to low water ordinary spring tide (LWOST). (the top corner of the caisson relative to LWOST).

Harbour side Sea side

*	*		
B1 = 2.719 M	B2 = 2.716 M	M1	18 MARCH 71
C1 = 2.722 M	C2 = 2.722 M		
D1 = 2.612 M	D2 = 2.640 M	M2	14 APRIL 71
E1 = 2.716 M	E2 = 2.725 M		
F1 = 2.778 M	F2 = 2.848 M	M3	19 APRIL 71
G1 = 2.646 M	G2 = 2.697 M		
H1 = 2.923 M	H2 = 2.899 M	M4	7 MAY 71
I1 = 2.682 M	I2 = 2.643 M		
J1 = 2.923 M	J2 = 2.972 M	M5	7 MAY 71
K1 = 3.036 M	K2 = 2.944 M		
L1 = 2.841 M	L2 = 2.990 M	M6	16 JUNE 71
M1 = 2.853 M	M2 = 2.910 M		
N1 = 2.752 M	N2 = 2.728 M	M7	19 JULY 71
O1 = 2.704 M	O2 = 2.640 M		

This table is an indication of the levels (relative to LWOST) present on the caissons just before the placement of the concrete capping. A certain amount of settlement had already taken place prior to the placement of the concrete capping.

Minimal amounts of movement took place in the months following the placing of the capping. The maximum settlement being 3 mm in 3 months. (information obtained from construction surveys)

The graph in fig 2.9 indicates the final levels of the caissons before the capping was placed. It can be noted that large differences are present, up to 400 mm in the final levels of the caissons. (See fig 2.9)

The assumption can be made that the capping was placed in such a manner such that a level surface was obtained.

As far as is known no levels have been taken on the caisson structure since the time of construction. The first levels taken after the construction are presented in this thesis, together with the relevant settlements during the monitoring period.

A series of level surveys were carried out on the individual caissons at certain intervals during the year. The levels were taken on brass pegs placed in the capping of each caisson. Brass studs have been provided at the four corners of each caisson. Level surveys were carried out on the following dates:

(a) 21 June 1993
(b) 17 October 1993
(c) 30 June 1994
(d) 24 August 1995 (See fig 2.10 for a graphic display of the settlement)

The results obtained are tabulated below:

All measurements in the table were taken on the East side (Harbour side) of the structure. A1 was the bench mark and therefore has a zero value. The rest is a settlement in relation to the bench mark A1. (See fig 2.10)

JUNE 1993	OCTOBER 1993	JUNE 1994	AUG 1995
A1 = 0	A1 = 0	A1 = 0	A1 = 0
B1 = -13	B1 = -12	B1 = -12	B1 = -13
C1 = -35	C1 = -35	C1 = -39	C1 = -40
D1 = -20	D1 = -21	D1 = -25	D1 = -26
E1 = -59	E1 = -70	E1 = -67	E1 = -70
F1 = -65	F1 = -65	F1 = -73	F1 = -75
G1 = -68	G1 = -80	G1 = -80	G1 = -83
H1 = -78	H1 = -89	H1 = -89	H1 = -93
I1 = -71	I1 = -77	I1 = -79	I1 = -81
J1 = -80	J1 = -85	J1 = -87	J1 = -89
K1 = -100	K1 = -100	K1 = -104	K1 = -107
L1 = -85	L1 = -87	L1 = -91	L1 = -94
M1 = -107	M1 = -107	M1 = -108	M1 = -112
N1 = -97	N1 = -97	N1 = -97	N1 = -100
O1 = -128	O1 = -128	O1 = -127	O1 = -130

The following information shows cumulated differences based on the first survey performed, as with reference to the previous table. (Harbour side)

JUNE 1993	OCTOBER 1993	JUNE 1994	AUGUST 1995
A1 = 0	A1 = 0	A1 = 0	A1 = 0
B1 = 0	B1 = -1	B1 = -1	B1 = 0
C1 = 0	C1 = 0	C1 = 4	C1 = 5
D1 = 0	D1 = 1	D1 = 5	D1 = 6
E1 = 0	E1 = 11	E1 = 8	E1 = 11
F1 = 0	F1 = 0	F1 = 8	F1 = 10
G1 = 0	G1 = 12	G1 = 12	G1 = 15
H1 = 0	H1 = 11	H1 = 11	H1 = 15
I1 = 0	I1 = 6	I1 = 8	I1 = 10
J1 = 0	J1 = 5	J1 = 7	J1 = 9
K1 = 0	K1 = 0	K1 = 4	K1 = 7
L1 = 0	L1 = 2	L1 = 6	L1 = 9
M1 = 0	M1 = 0	M1 = 1	M1 = 5
N1 = 0	N1 = 0	N1 = 0	N1 = 3
O1 = 0	O1 = 0	O1 = -1	O1 = 2

All measurements in the following table were taken on the West side (Sea side) of the structure on the brass studs. (See fig 2.10)

JUNE 1993	OCTOBER 1993	JUNE 1994	AUG 1995
A2 = -25	A2 = -26	A2 = -25	A2 = -25
B2 = -35	B2 = -36	B2 = -36	B2 = -36
C2 = -58	C2 = -58	C2 = -62	C2 = -66
D2 = -61	D2 = -62	D2 = -65	D2 = -66
*	*	*	*
*	*	*	*
G2 = -92	G2 = -104	G2 = -103	G2 = -106
H2 = -94	H2 = -106	H2 = -106	H2 = -109
I2 = -98	I2 = -104	I2 = -107	I2 = -109
J2 = -113	J2 = -120	J2 = -122	J2 = -125
K2 = -126	K2 = -128	K2 = -132	K2 = -135
L2 = -120	L2 = -121	L2 = -125	L2 = -127
M2 = -105	M2 = -106	M2 = -108	M2 = -110
N2 = -90	N2 = -90	N2 = -92	N2 = -94
O2 = -125	O2 = -124	O2 = -123	O2 = -126

The following information is cumulated differences based on the first survey performed, as displayed in the previous table. (Sea side)

JUNE 1993	OCTOBER 1993	JUNE 1994	AUGUST 1995
A2 = 0	A2 = 1	A2 = 0	A2 = 0
B2 = 0	B2 = 1	B2 = 1	B2 = 1
C2 = 0	C2 = 0	C2 = 4	C2 = 8
D2 = 0	D2 = 1	D2 = 4	D2 = 5
*	*	*	*
*	*	*	*
G2 = 0	G2 = 12	G2 = 11	G2 = 14
H2 = 0	H2= 12	H2 = 12	H2 = 15
I2 = 0	I2 = 6	I2 = 9	I2 = 11
J2 = 0	J2 = 7	J2 = 9	J2 = 12
K2 = 0	K2 = 2	K2 = 6	K2 = 9
L2 = 0	L2 = 1	L2 = 5	L2 = 7
M2 = 0	M2 = 1	M2 = 3	M2 = 5
N2 = 0	N2 = 0	N2 = 2	N2 = 4
O2 = 0	O2 = -1	O2 = -2	O2 = 1

For a summary in graph form of caissons settlement during the monitoring period (see fig 2.11 - fig 2.13). The wave condition experienced during the tables during period are illustrated in fig 2.14 - 2.17. The wave information was gathered from the wave rider buoy situated at Koeberg. The Tables illustrate the significant wave height experienced. The wave period is unfortunately not coupled to these tables.

2.E.2. Results of horizontal movement

Horizontal measurements between the brass studs present in the caisson capping were measured to detect any horizontal movement between the caissons. The horizontal movement was however negligible and no results thereof was therefore placed .

2.E.3. Summary

The following conclusions can be derived from the information obtained in the level surveys and construction details.

(a) The worst settlements occurred during the monitoring period June 1993 and October 1993. Factors contributing to structure movements or damage during high wave conditions is the wave period and wave direction of the storm. A storm with a big wave height but with a small wave period will not have the same effect on the breakwater structure as a wave with a large wave height and period. (Shore Protection Manual 1977) The analysis of wave periods combined with wave height and wave direction and their effects on breakwater structures will have to be analysed before any assumptions could be made with reference to settlement of the structure. Little correlation between storm occurrences and settlement were observed.

(b) The caissons worst effected are M3 and M4 with a maximum cumulative settlement of 15 mm obtained during the monitoring period. They are followed by caissons M2 and M5 with cumulative settlements of up to 12 mm.

(c) The level difference over the length of the structure between caissons M1 and M7 on the harbour side of the structure is 130 mm. The Sea side level difference of the structure is 126 mm.

(d) The levels were obtained on brass pegs on the concrete capping of the structure. This is not a true reflection of the settlement taking place as deflection of the concrete capping could have distorted the values obtained on the brass pegs. It is assumed that the differences between the settlement of the caisson and that of the capping are the same as no major external damage is visible on the capping.

2.F CONCLUSION

From the data available it can be established that settlement of the caisson structure is definitely taking place. The factors contributing to the foundation settlement is due either to the movement of the caissons (due to wave action) or shock wave propagation in the foundation.

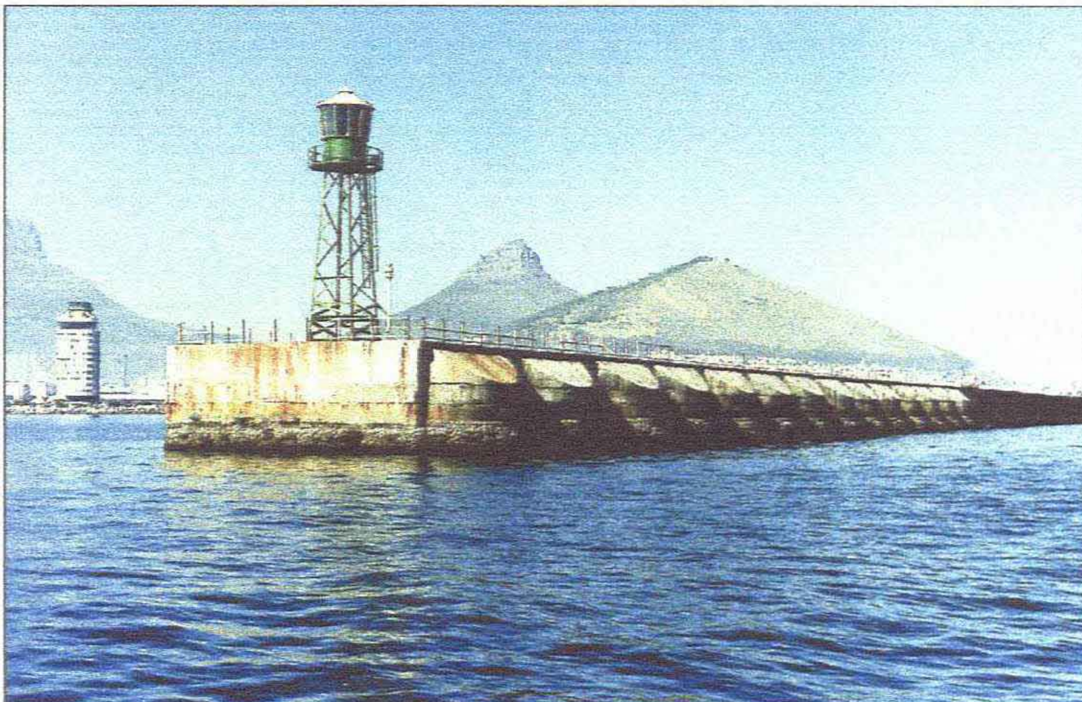
(a) The type of foundation failure could be either a localised foundation failure, or a sub foundation failure. Localised failure would be as a result of caisson movement causing a change in the foundation profile which will result in caisson settlement. Sub-foundation failure will be due to shear failure of the soil layers in the existing or undisturbed foundation material. This will be similar to the slip circle effect resulting in settlement of the entire structure.

(b) To determine the conditions responsible for the movement of the caissons or the factors present in the foundation system, requires extensive investigations. This could be a study in its own right as can be seen from the examples on the alternative methods of analysing caisson structures.

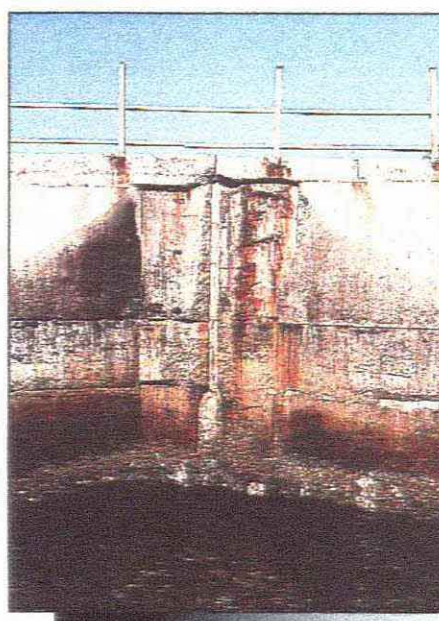
(c) Current erosion around the structure is localised, due to a grout sock failure between caisson M6 and M7 and is the cause of the damage to the foundation. Other forms of erosion, if present, are negligible.

(d) Constant changing of the sand profiles around the caissons appears to have no influence on the wave conditions exerted on the structure. As discussed in chapter 2.B.2, the possibility of the sand ever having any influence in the wave conditions around the structure is negligible.

Alkali aggregate reaction is present in the concrete capping. This has caused cracks in the capping with the subsequent deterioration thereof.



The concrete capping is also deteriorating due to the large number of wave attacks on the structure.



Photograph group - C

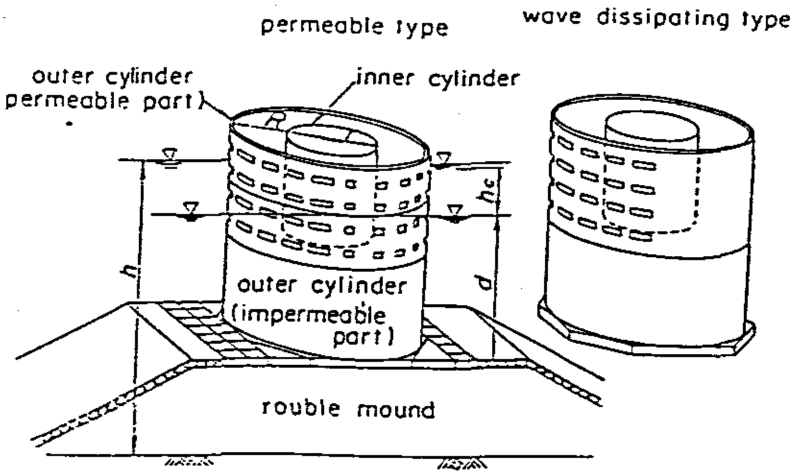


Fig. 1 Conceptual Figure of Dual Cylindrical Caisson

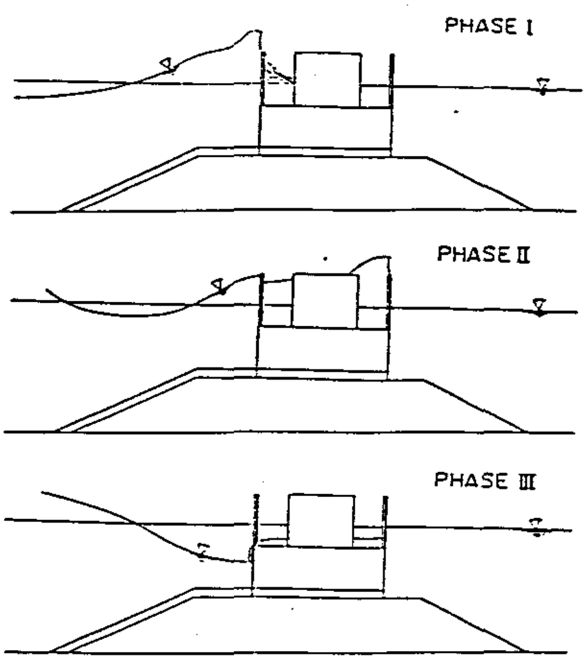


Fig. 2 Three Phases of Wave Action

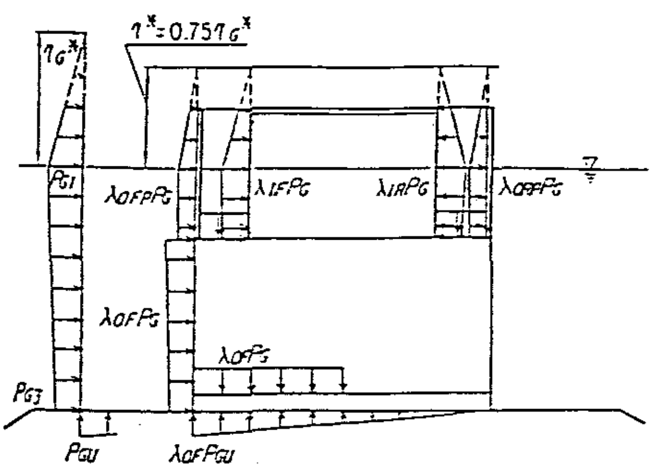


Fig. 3 Distribution of Wave Pressure

Dual cylindrical caissons- Three phases of wave action | Fig 2.1

a) Hydraulic Model Tests

The hydraulic model tests were conducted by using regular and irregular waves with wave heights and periods up to 1.20m and 7s, respectively. The measurements were simultaneously performed on two independent caisson structures. Total forces were measured on the first caisson. The second caisson was installed on a rubble mound foundation lying on a 1.4 m-thick sand layer (Fig. 1).

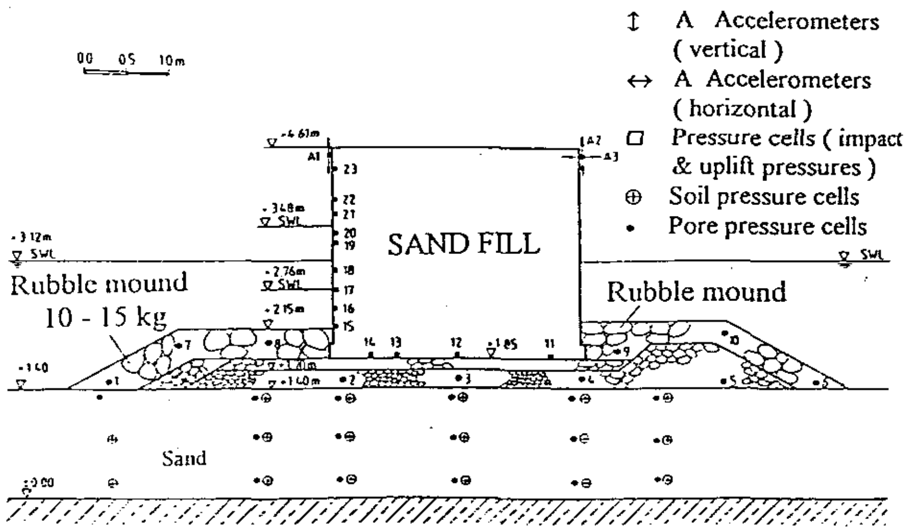
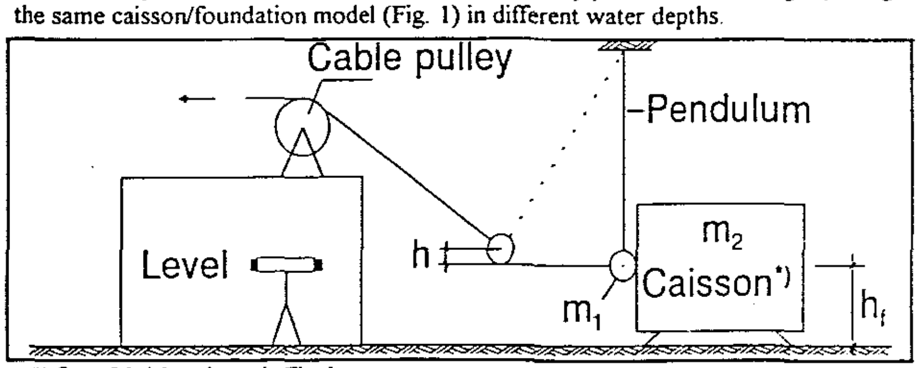


FIG. 1: CAISSON BREAKWATER TESTED IN THE LARGE WAVE FLUME (GWK)

Simultaneous measurements of the following items were carried out:
(a) incident and reflected waves, (b) impact pressure on the caisson front, (c) uplift pressure, (d) wave-induced pore-water pressure in the foundation, (e) total wave-induced stress in the sand layer, (f) dynamic response of the caisson (accelerations) and (g) total forces.

b) Pendulum Tests

The hydraulic model tests were supplemented by pendulum tests (Fig. 2) using the same caisson/foundation model (Fig. 1) in different water depths.



*) Same Model as shown in Fig. 1

FIG. 2: EXPERIMENTAL SET-UP FOR PENDULUM TESTS

The impulsive loads induced by the pendulum and the response of the structure and its foundation were simultaneously recorded. The main objective of these tests was to determine the hydrodynamic mass as well as the damping and the subgrade reaction coefficients to be considered in a dynamic analysis of the caisson/foundation system. Two test series were conducted on the caisson part lying on the rubble mound foundation: Tests under dry conditions and tests with different water depths.

The wave flume test & Pendulum test | Fig 2.2

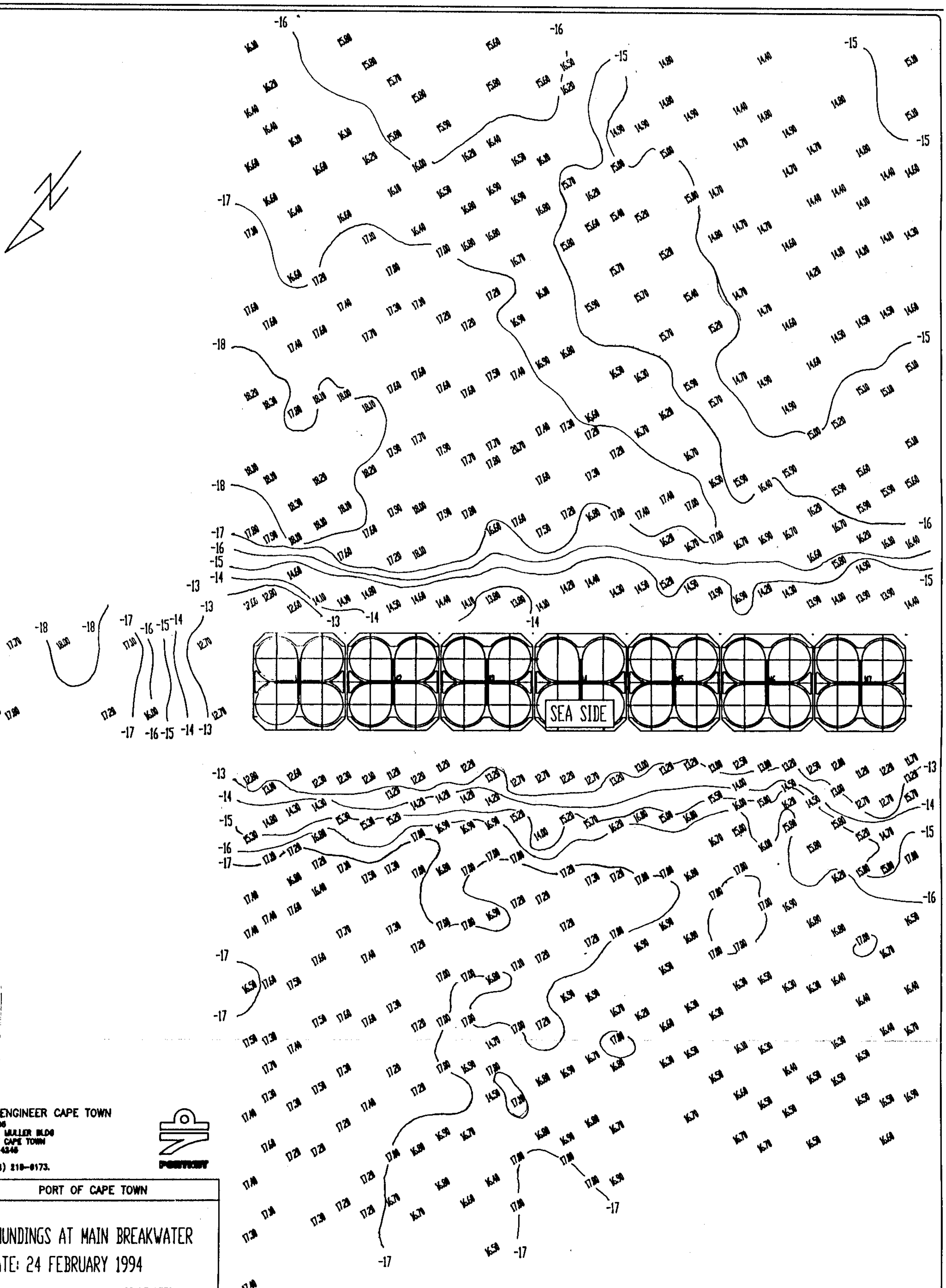


Fig 2.3

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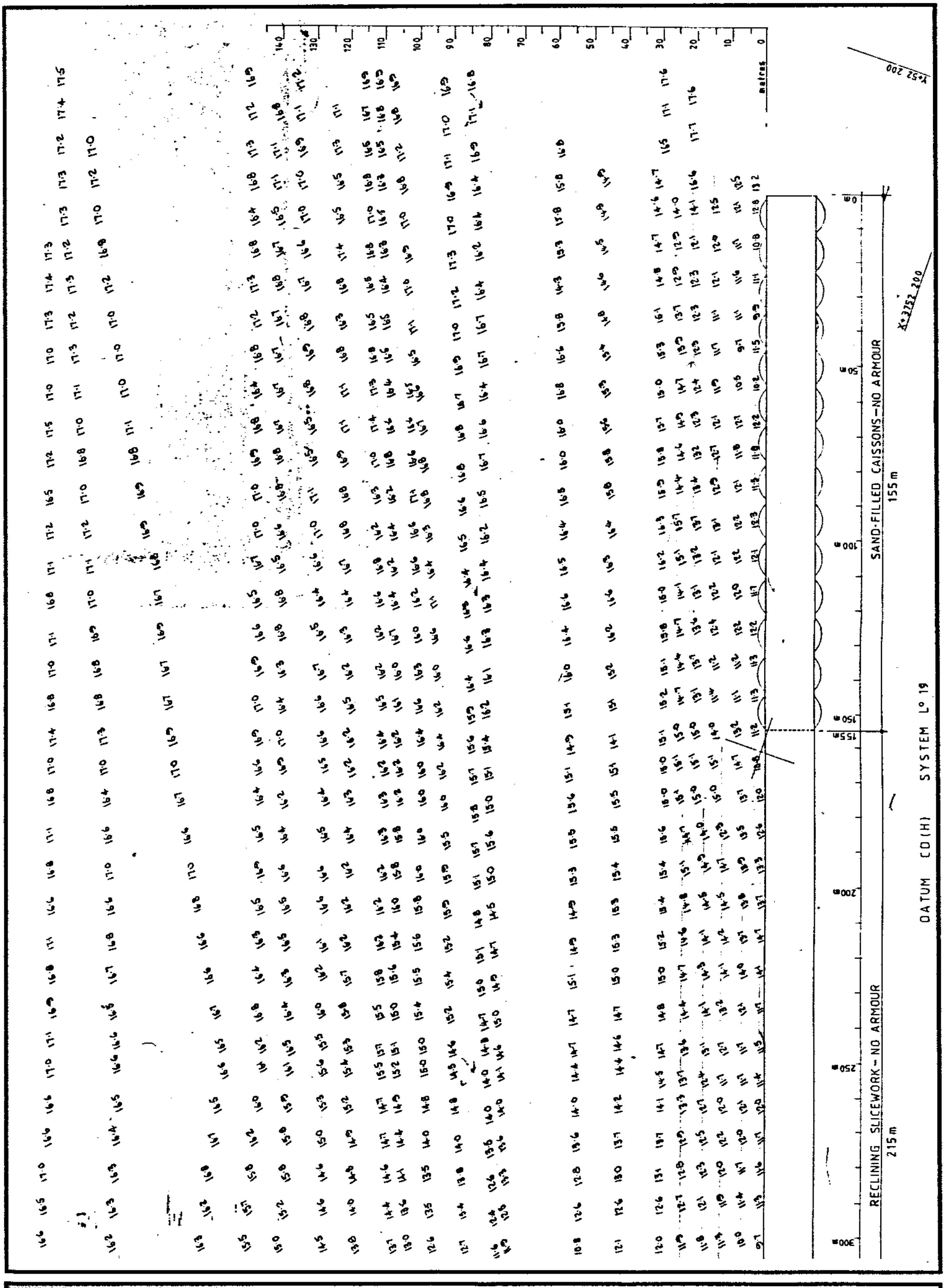
PORT OF CAPE TOWN

SOUNDINGS AT MAIN BREAKWATER

DATE: 24 FEBRUARY 1994

SCALE 1:750

CHART DATUM HARBOURS



Soundings at the Main Breakwater - Feb 1987

Fig 2.4

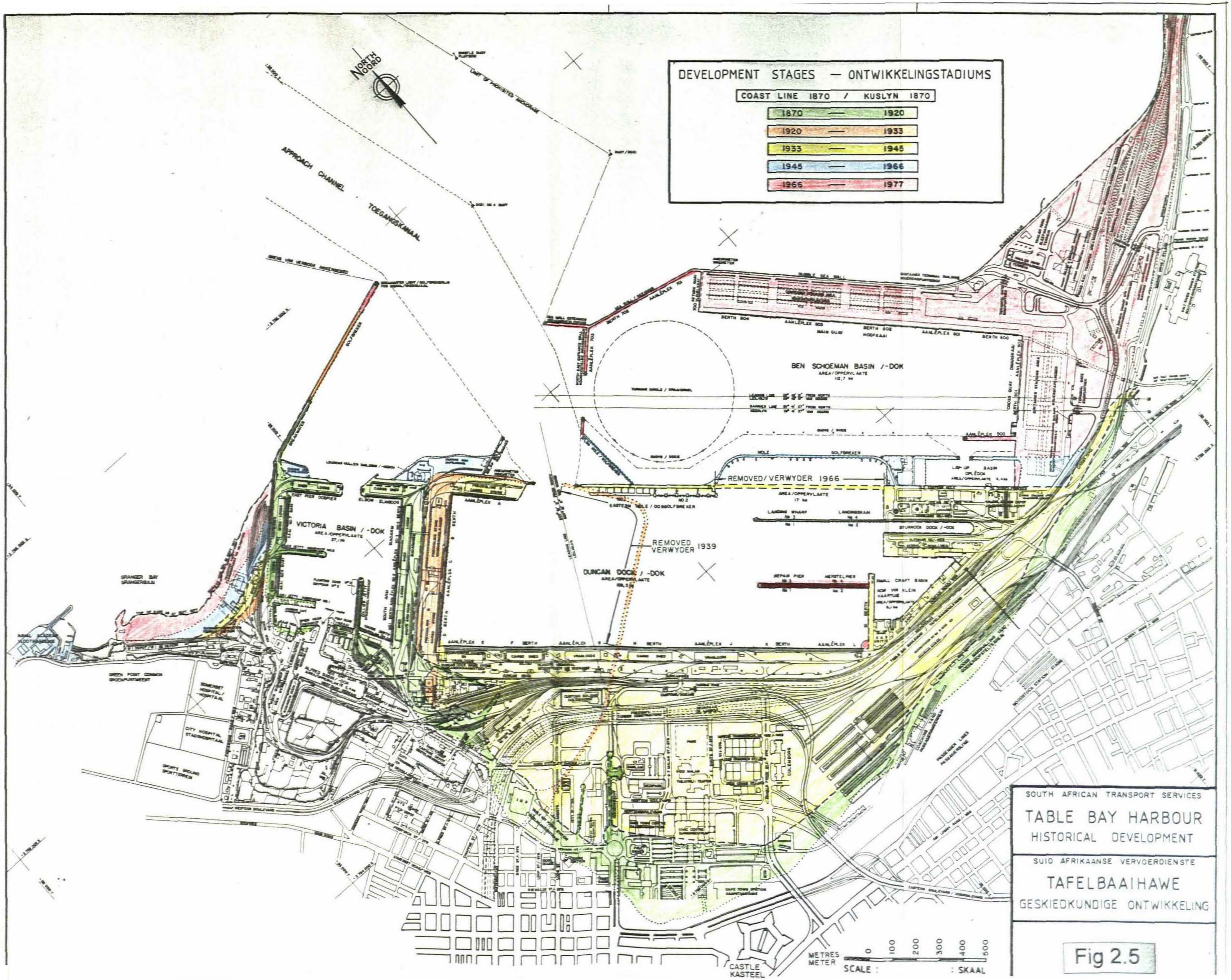

DEVELOPMENT STAGES — ONTWIKKELINGSTADIUMS
COAST LINE 1870 / KUSLYN 1870
1870 — 1920
1920 — 1933
1933 — 1945
1945 — 1966
1966 — 1977
NORTH
NOORD
APPROACH CHANNEL
TOEGANGSKANAAL
BEN SCHOEMAN BASIN /-DOK
VICTORIA BASIN /-DOK
DUNCAN DOCK /-DOK
REMOVED/VERWYDER 1966
REMOVED
VERWYDER 1939
GRANGER BAY
GRANGERBAAI
CASTLE
KASTEEL
METRES
METER
0
100
200
300
400
500
SCALE : : SKAAL
SOUTH AFRICAN TRANSPORT SERVICES
TABLE BAY HARBOUR
HISTORICAL DEVELOPMENT
SUID AFRIKAANSE VERVOERDIENSTE
TAFELBAAIHAWE
GESKIEDKUNDIGE ONTWIKKELING

Fig 2.5

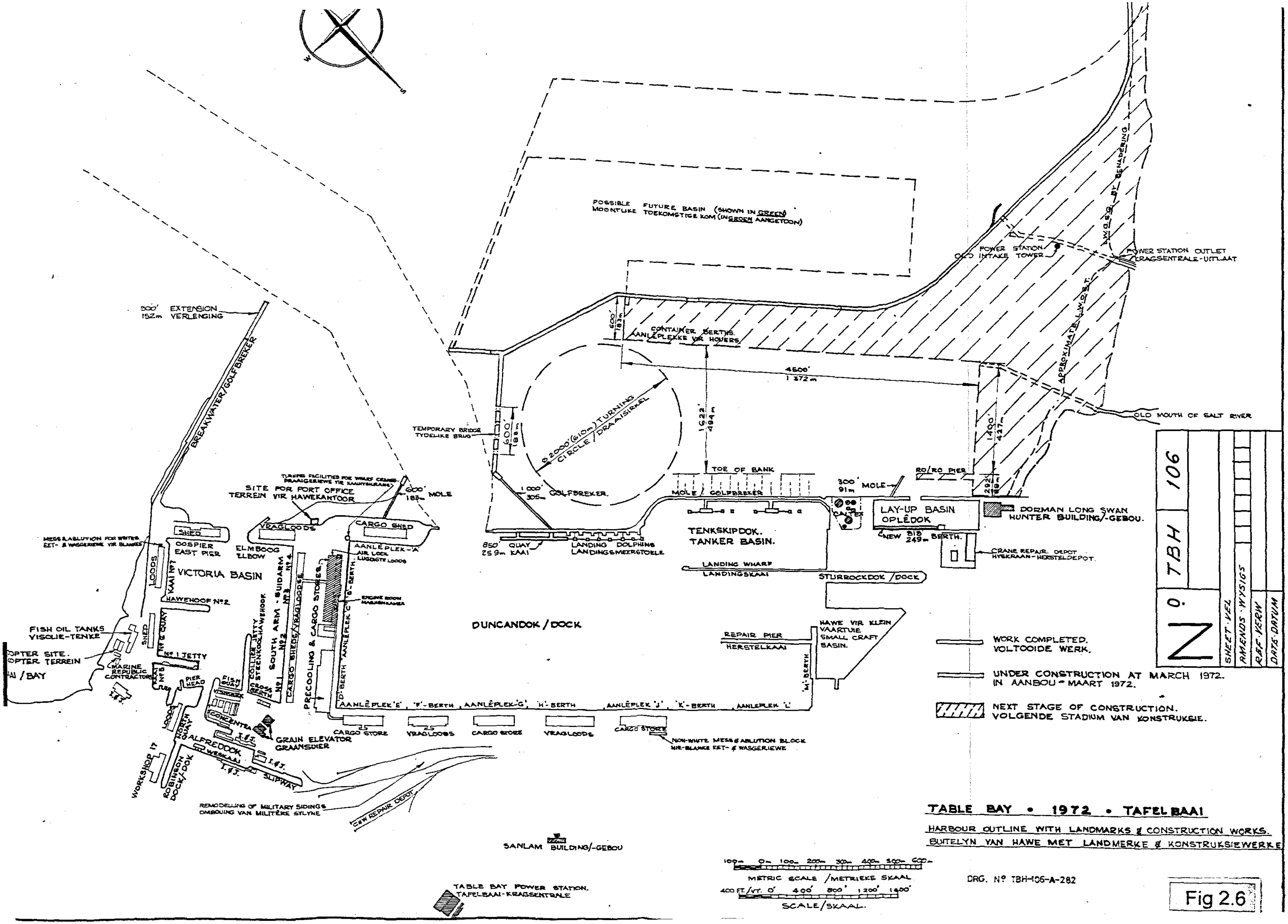
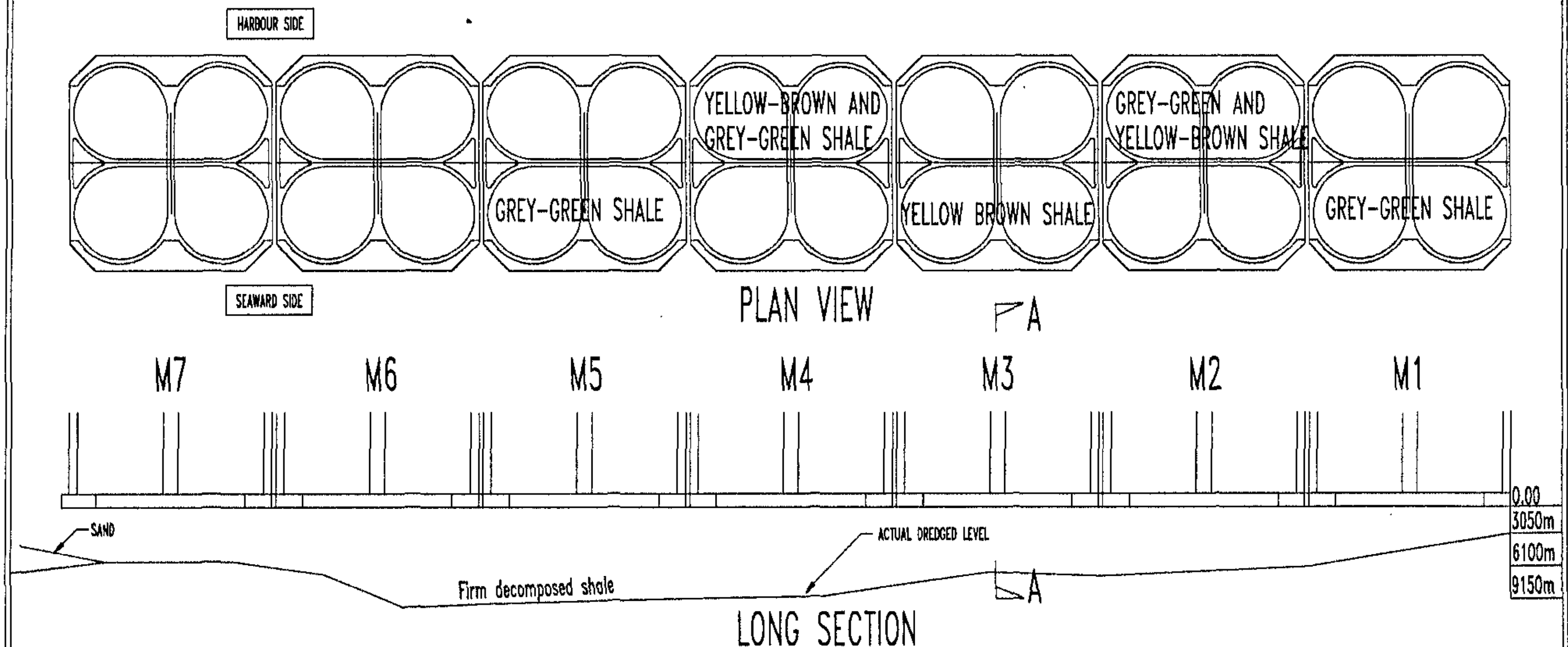
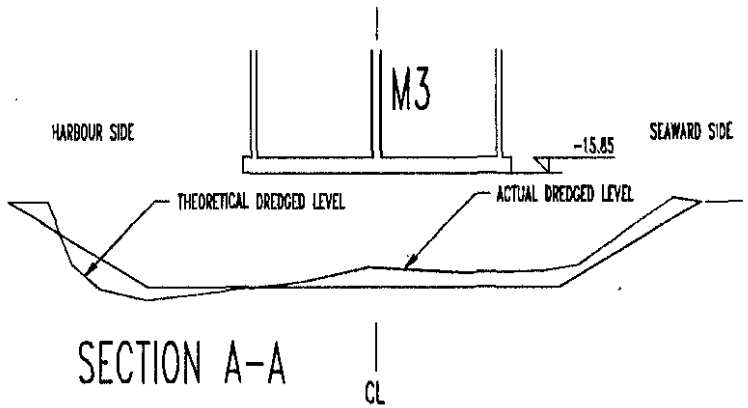

POSSIBLE FUTURE BASIN (SHOWN IN GREEN)
MOONTLIKE TOEKOMSTIGE KOM (IN GROEN AANGETOON)
POWER STATION OLD INTAKE TOWER
POWER STATION OUTLET KRAGSENTRALE-UITLAAT
L.W.O.S.T. BY BENADERING
APPROXIMATE L.W.O.S.T.
OLD MOUTH OF SALT RIVER
500' EXTENSION 152m VERLENGING
BREAKWATER/GOLFBREKER
CONTAINER BERTHS. AANLEPLEKKE VIR HOUERS
600' 183m
4500' 1 372m
1622' 494m
1400' 427m
Ø 2000' (610m) TURNING CIRCLE / DRAAISIRKEL
TEMPORARY BRIDGE TYDELIKE BRUG
600' 183m
TOE OF BANK
MOLE / GOLFBREKER
300' 91m MOLE
RO/RO PIER
LAY-UP BASIN OPLÊDOK
NEW 818 249m BERTH.
DORMAN LONG SWAN HUNTER BUILDING/-GEBOU.
CRANE REPAIR DEPOT HYSKRAAN-HERSTELDEPOT.
TENKSKIPDOK. TANKER BASIN.
CALTEX
LANDING WHARF LANDINGSKAAI
STURROCKDOK / DOCK
1000' 305m GOLFBREKER
850' QUAY 259m KAAI
LANDING DOLPHINS LANDINGSMEERSTOELE
SITE FOR PORT OFFICE TERREIN VIR HAWEKANTOOR
600' 183m MOLE
VRAGLOODS
CARGO SHED
AANLEPLEK 'A'
AIR LOCK LUGDIGTE LOODS
ENGINE ROOM MASJIENKAMER
MESS & ABLUTION FOR WHITES. EET- & WASGERIEWE VIR BLANKES
SHED
COSPIER EAST PIER
ELMBOOG ELBOW
VICTORIA BASIN
LOODS
KAAI Nº7
HAWEHOOF Nº2
SOUTH ARM - SUIDARM
Nº4 Nº3 Nº2 Nº1
CARGO SHEDS/VRAGLOODSE
PRECOOLING & CARGO STORES
COLLIER JETTY STEENKOOLHAWEHOOF
Nº1 JETTY
Nº6 QUAY
FISH OIL TANKS VISOLIE-TENKE
COPTER SITE. COPTER TERREIN
BAAI / BAY
MARINE REPUBLIC CONTRACTORS
FISH QUAY
PIER HEAD
CROSS BERTH
CONCENTRA
NORTH QUAY
GRAIN ELEVATOR GRAANSUIER
ALFREDDOK
WESKAAI
SLIPWAY
ROBINSON DOCK/DOK
WORKSHOP 17
DUNCANDOK / DOCK
REPAIR PIER HERSTELKAAI
HAWE VIR KLEIN VAARTUIE SMALL CRAFT BASIN.
'M' BERTH
'D' BERTH
AANLEPLEK 'C'
'B' BERTH
AANLÊPLEK 'E'
'F' BERTH
AANLÊPLEK 'G'
'H' BERTH
AANLEPLEK 'J'
'K' BERTH
AANLEPLEK 'L'
CARGO STORE
VRAGLOODS
CARGO STORE
VRAGLOODS
CARGO STORE
NON-WHITE MESS & ABLUTION BLOCK NIE-BLANKE EET- & WASGERIEWE
REMODELLING OF MILITARY SIDINGS OMBOUING VAN MILITÊRE SYLYNE
C&W REPAIR DEPOT
SANLAM BUILDING/-GEBOU
TABLE BAY POWER STATION. TAFELBAAI-KRAGSENTRALE
WORK COMPLETED. VOLTOOIDE WERK.
UNDER CONSTRUCTION AT MARCH 1972. IN AANBOU - MAART 1972.
NEXT STAGE OF CONSTRUCTION. VOLGENDE STADIUM VAN KONSTRUKSIE.
TABLE BAY • 1972 • TAFELBAAI
HARBOUR OUTLINE WITH LANDMARKS & CONSTRUCTION WORKS.
BUITELYN VAN HAWE MET LANDMERKE & KONSTRUKSIEWERKE
METRIC SCALE / METRIEKE SKAAL
SCALE/SKAAL.
DRG. Nº TBH-106-A-282
Nº TBH 106
SHEET VEL
AMENDS WYSIGS
REF. VERW
DATE DATUM

Fig 2.6





CAISSON	PENETRATION TEST			FOUNDING MATERIAL
	SEA	CEN	HARB	
M1	102mm	076mm	051mm	Decomposed shale
M2	102mm	102mm	076mm	Decomposed shale
M3	051mm	051mm	025mm	Decomposed shale
M4	051mm	076mm	051mm	Decomposed shale
M5	076mm	051mm	076mm	Decomposed shale
M6	076mm	076mm	076mm	Decomposed shale
M7	076mm	127mm	076mm	Decomposed shale

NOTE:

Penetration obtained by hammering 19mm rod into the seabed

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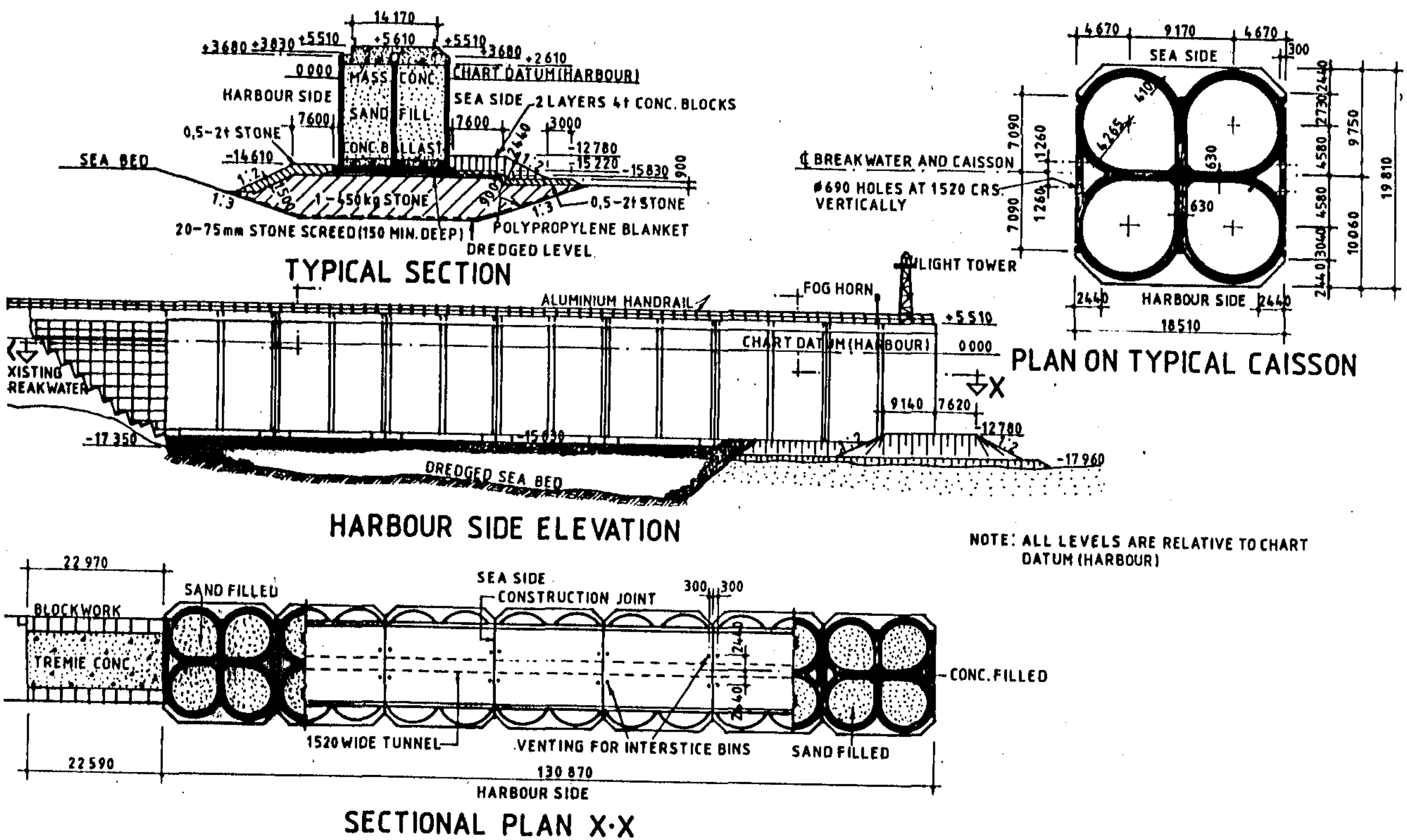
MAIN BREAKWATER CAISSON
M1-M7 DREDGING

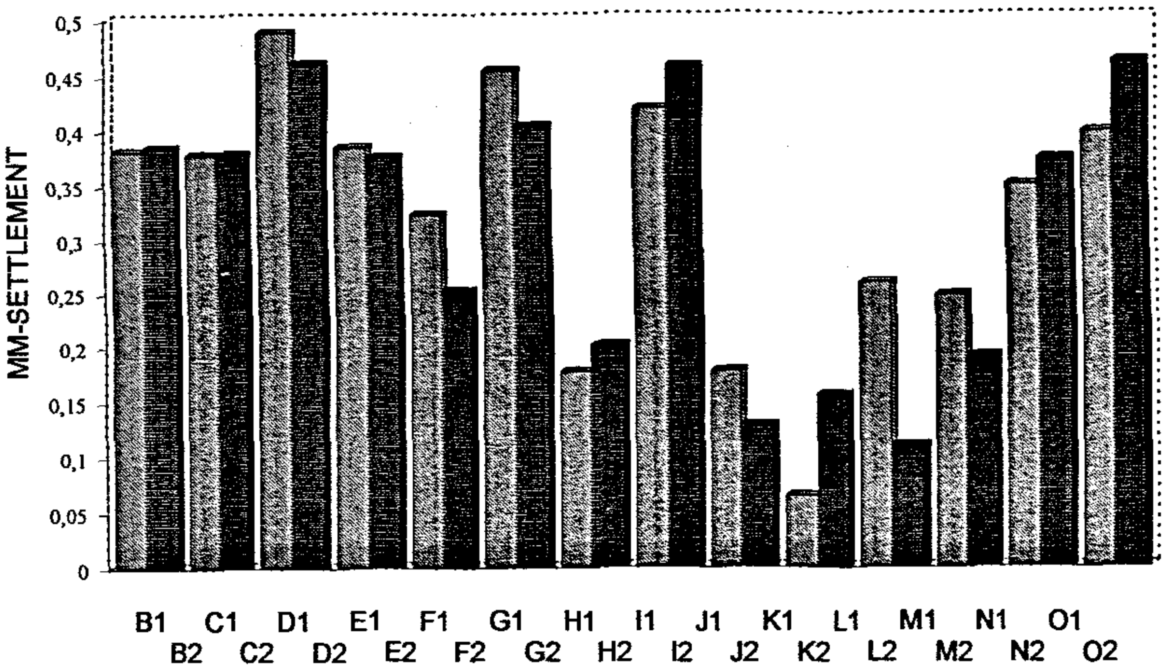
Main Breakwater-Caisson M1-M7 (dredging materials)

Fig 2.7

Main Breakwater - General arrangement

Fig 2.8



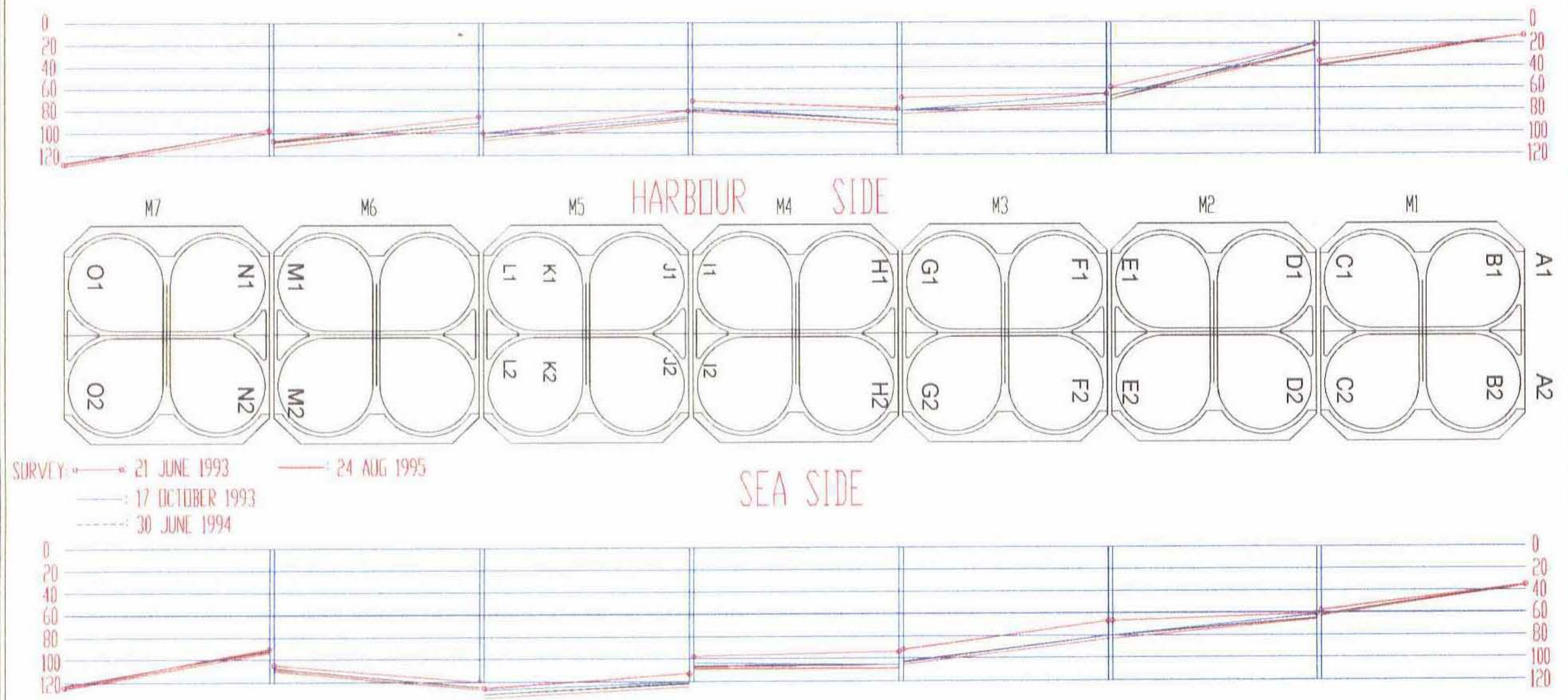


CAISSON BRASS STUD LEVELS
JUNE 1971

Caisson construction levels - 1971

Fig 2.9

Main Breakwater - Settlement surveys



HARBOUR SIDE		SEA SIDE		21 JUNE 1993
0	A1	-25	A2	
-13	B1	-35	B2	
-35	C1	-58	C2	
-20	D1	-61	D2	
-59	E1	-67	E2	
-65	F1	-67	F2	
-68	G1	-92	G2	
-78	H1	-94	H2	
-71	I1	-98	I2	
-80	J1	-113	J2	
-100	K1	-126	K2	
-85	L1	-120	L2	
-107	M1	-105	M2	
-97	N1	-90	N2	
-128	O1	-125	O2	

HARBOUR SIDE		SEA SIDE		17 OCTOBER 1993
0	A1	-26	A2	
-12	B1	-36	B2	
-35	C1	-58	C2	
-21	D1	-62	D2	
-70	E1	-80	E2	
-65	F1	-80	F2	
-80	G1	-104	G2	
-89	H1	-106	H2	
77	I1	-104	I2	
-85	J1	-120	J2	
100	K1	-128	K2	
87	L1	121	L2	
-107	M1	-106	M2	
-97	N1	-90	N2	
128	O1	124	O2	

HARBOUR SIDE		SEA SIDE		30 JUNE 1994
0	A1	-25	A2	
-12	B1	-36	B2	
-39	C1	-62	C2	
-25	D1	-65	D2	
-67	E1	-80	E2	
-73	F1	-80	F2	
-80	G1	-103	G2	
-89	H1	-106	H2	
-79	I1	-107	I2	
-87	J1	-122	J2	
-104	K1	-132	K2	
-91	L1	-125	L2	
-108	M1	-108	M2	
-97	N1	-92	N2	
127	O1	-123	O2	

HARBOUR SIDE		SEA SIDE		24 AUG 1995
0	A1	25	A2	
-13	B1	-36	B2	
-40	C1	-66	C2	
-26	D1	-66	D2	
-70	E1	-83	E2	
-75	F1	-83	F2	
-83	G1	-106	G2	
-93	H1	-109	H2	
-81	I1	-109	I2	
-89	J1	125	J2	
-107	K1	-135	K2	
-94	L1	-127	L2	
-112	M1	-110	M2	
-100	N1	-94	N2	
-130	O1	-126	O2	

NOTE:
ALL LEVELS WERE TAKEN ON BRASS STUDS INSTALLED AFTER CONSTRUCTION OF THE CAISSON STRUCTURE.

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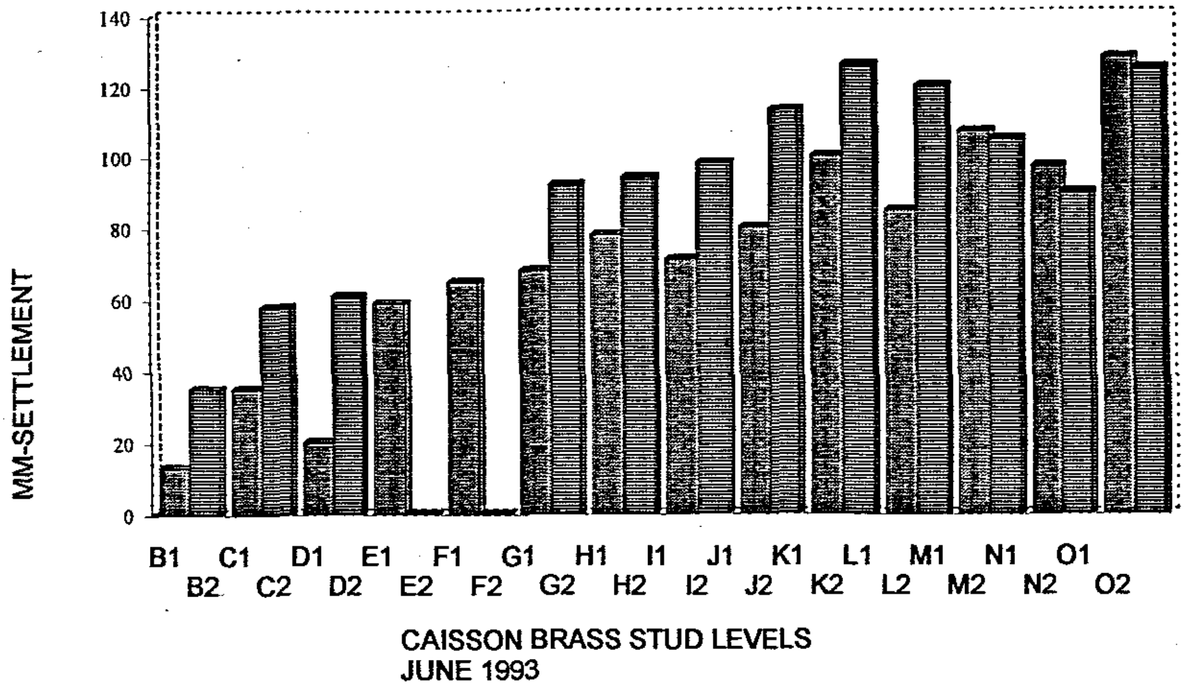
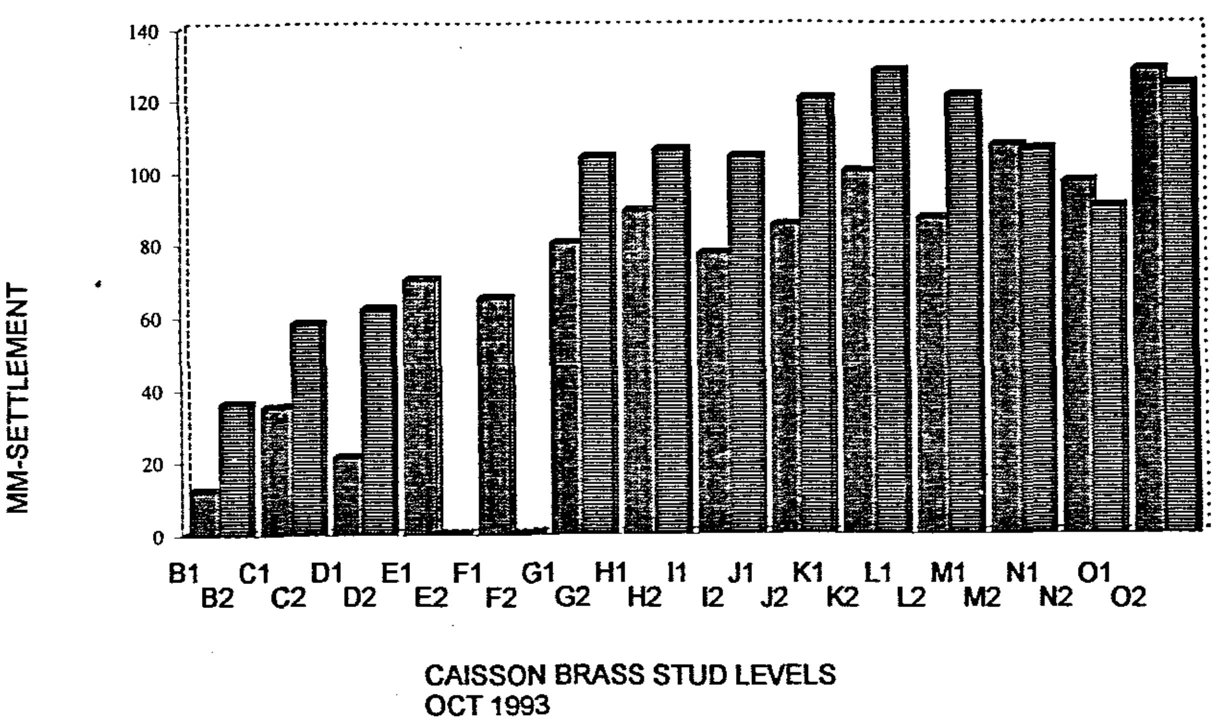
MAIN BREAKWATER CAISSON LEVEL SURVEY

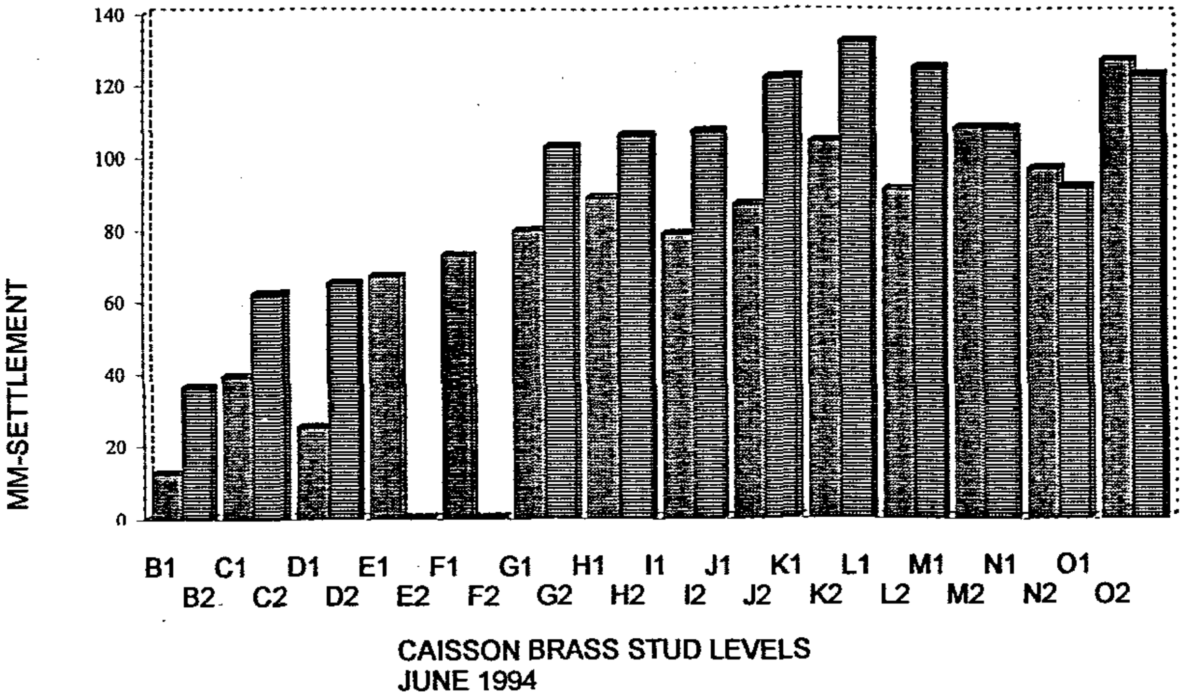
Fig 2.10

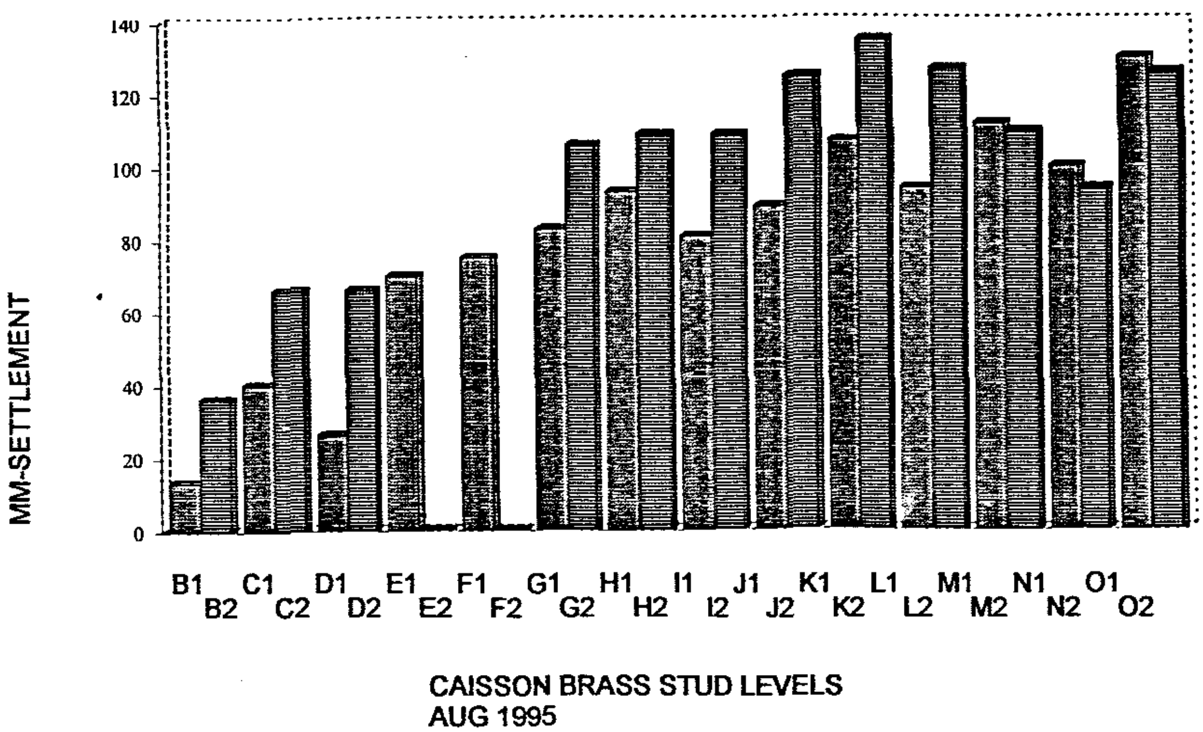




Caisson Settlement-June 1993 & Oct 1993

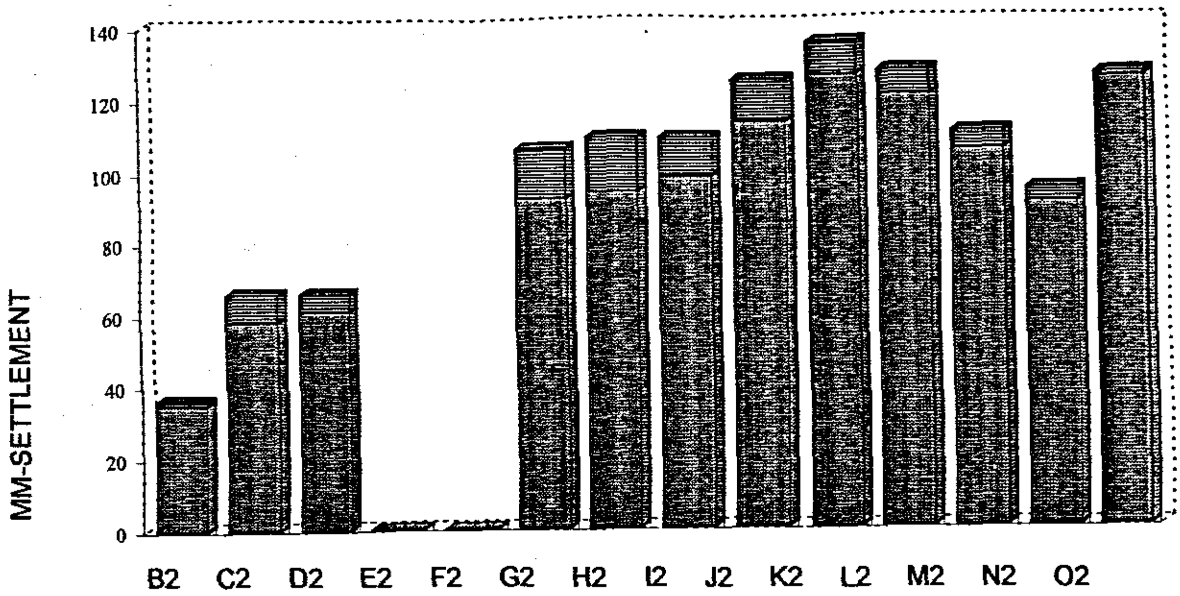
Fig 2.11



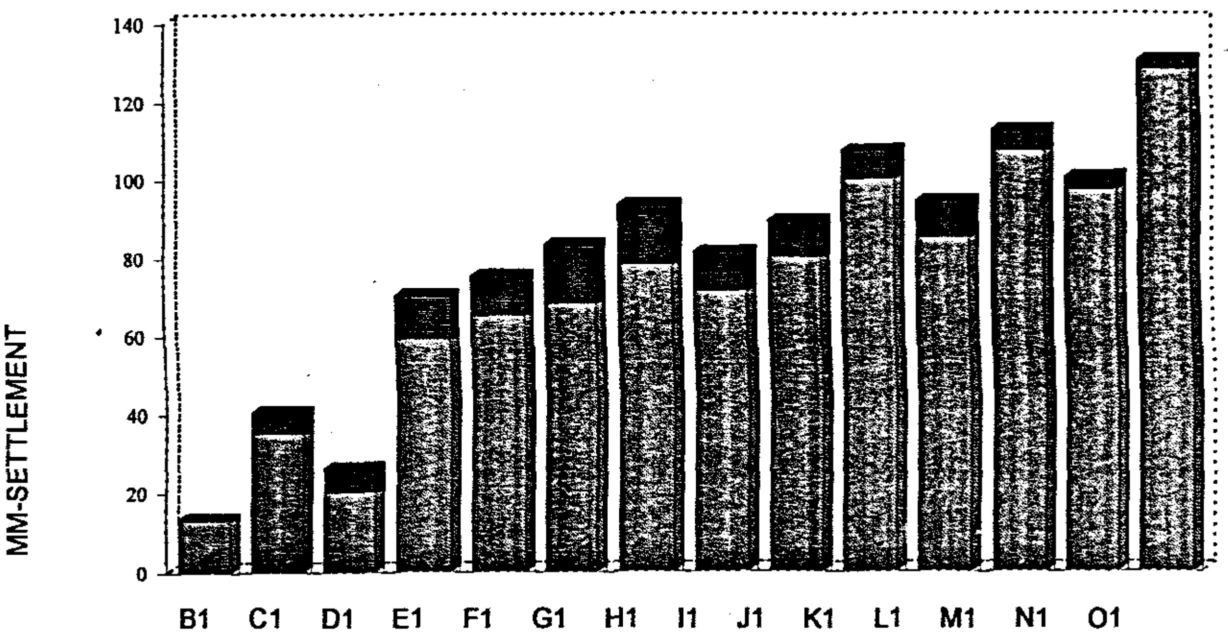


Caisson Settlement-June 1994 & Aug 1995

Fig 2.12

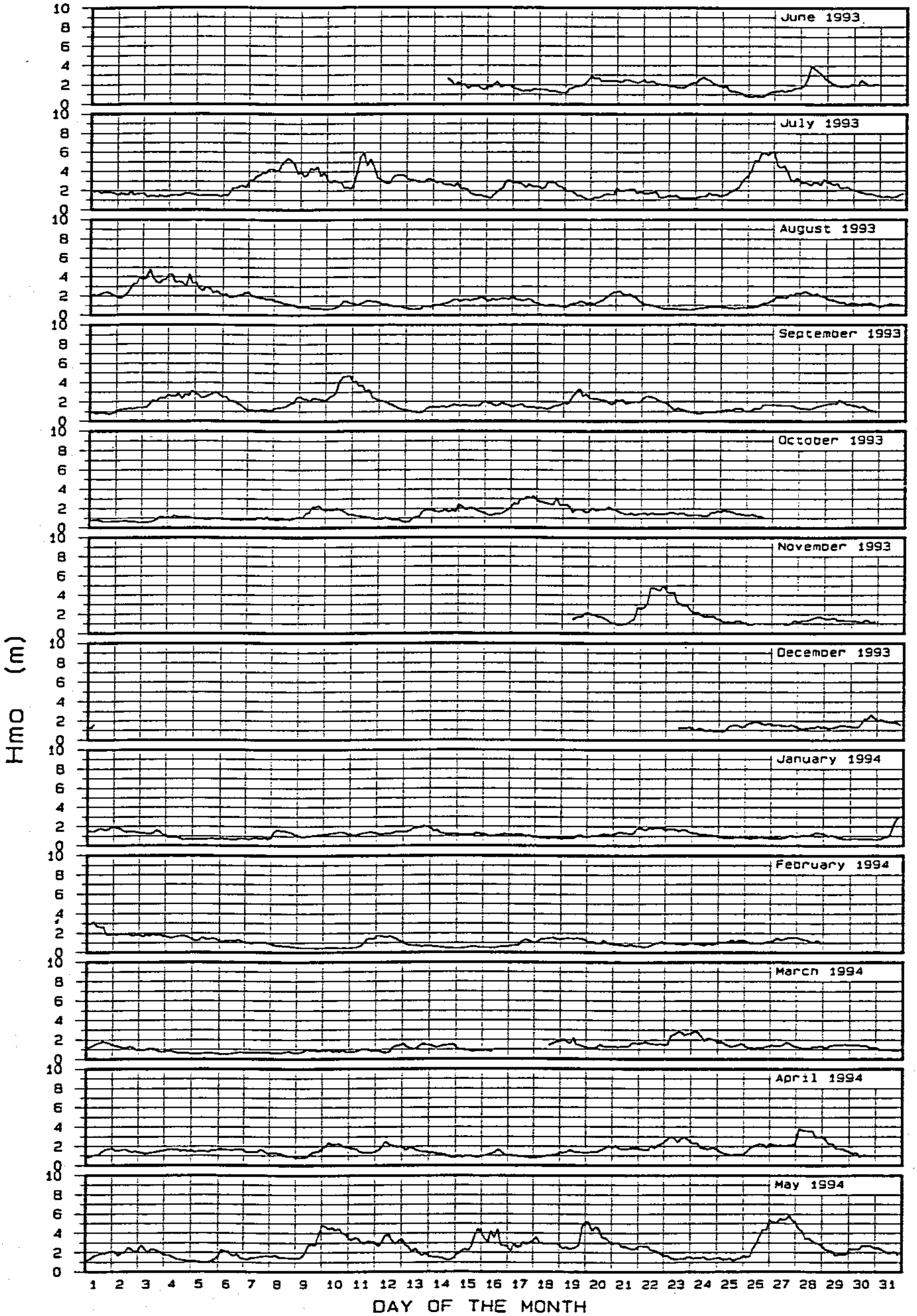


CAISSON BRASS STUD LEVELS-(sea side)
JUNE 1993 TO AUGUST 1995



CAISSON BRASS STUD LEVELS-(harbour side)
JUNE 1993 TO AUGUST 1995

Caisson Settlement increase-June 93 until Aug 95 | Fig 2.13



CSIR

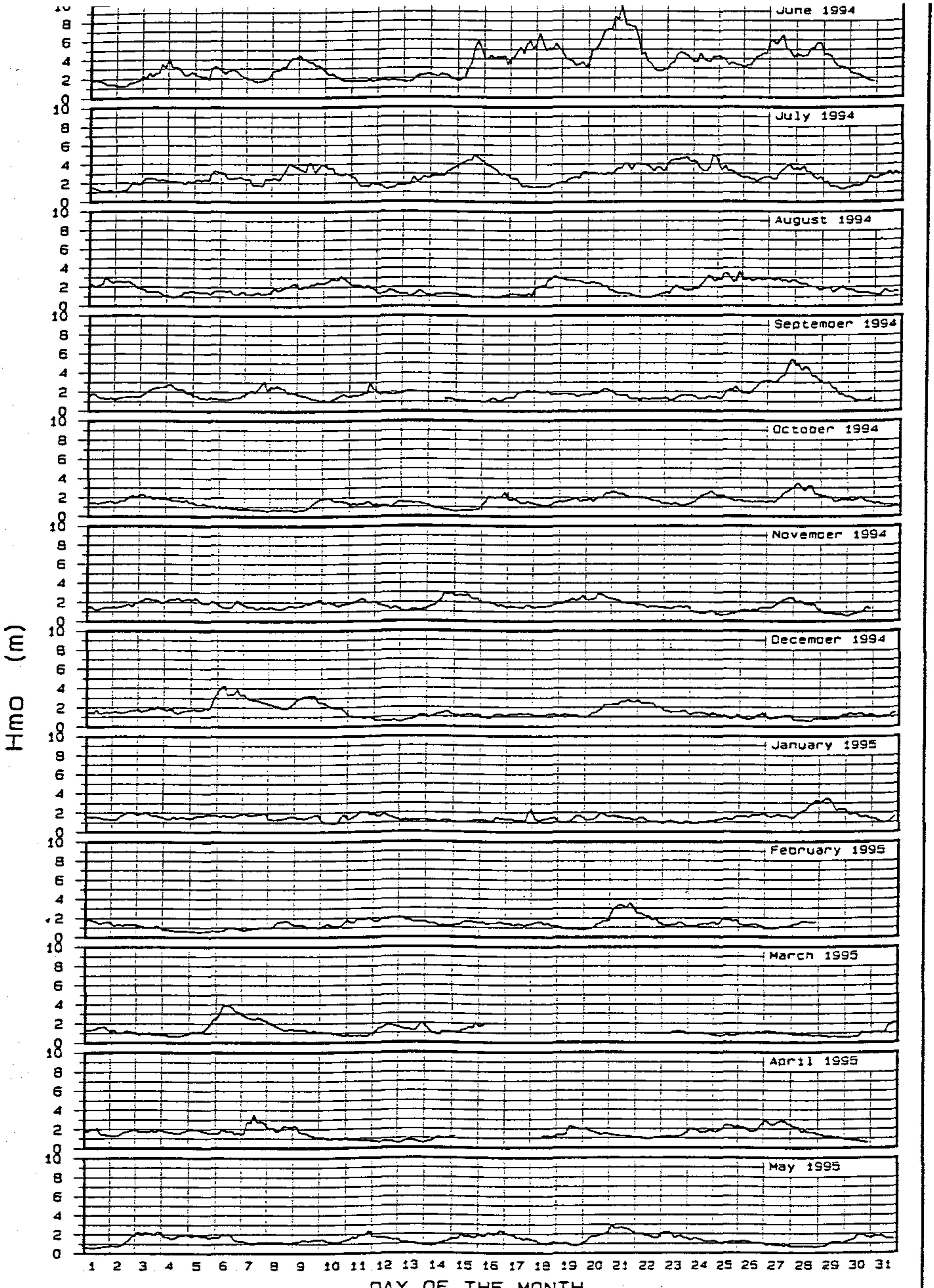
TABLE BAY/KOEBERG

TIME SERIES FOR Hmo

1993-06-01 to 1994-05-31

FIGURE 2.14

WAV_TSY 1996-29-03 08:36:17



CSIR

TABLE BAY/KOEBERG

TIME SERIES FOR Hmo

1994-06-01 to 1995-05-31

FIGURE 2.15

WAV_TSY 1996-29-03 08:37:35

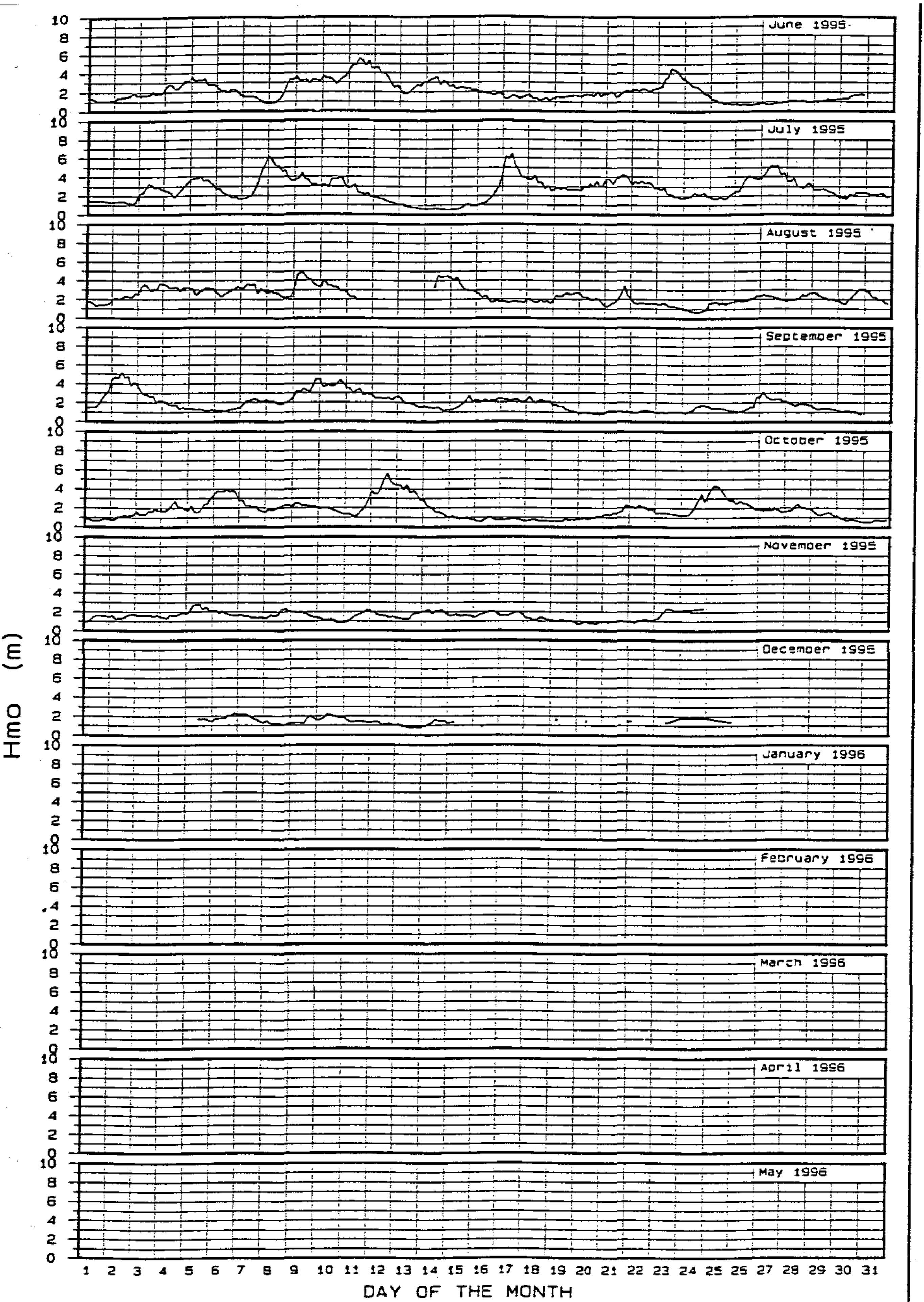


TABLE BAY/KOEBERG

TIME SERIES FOR Hmo

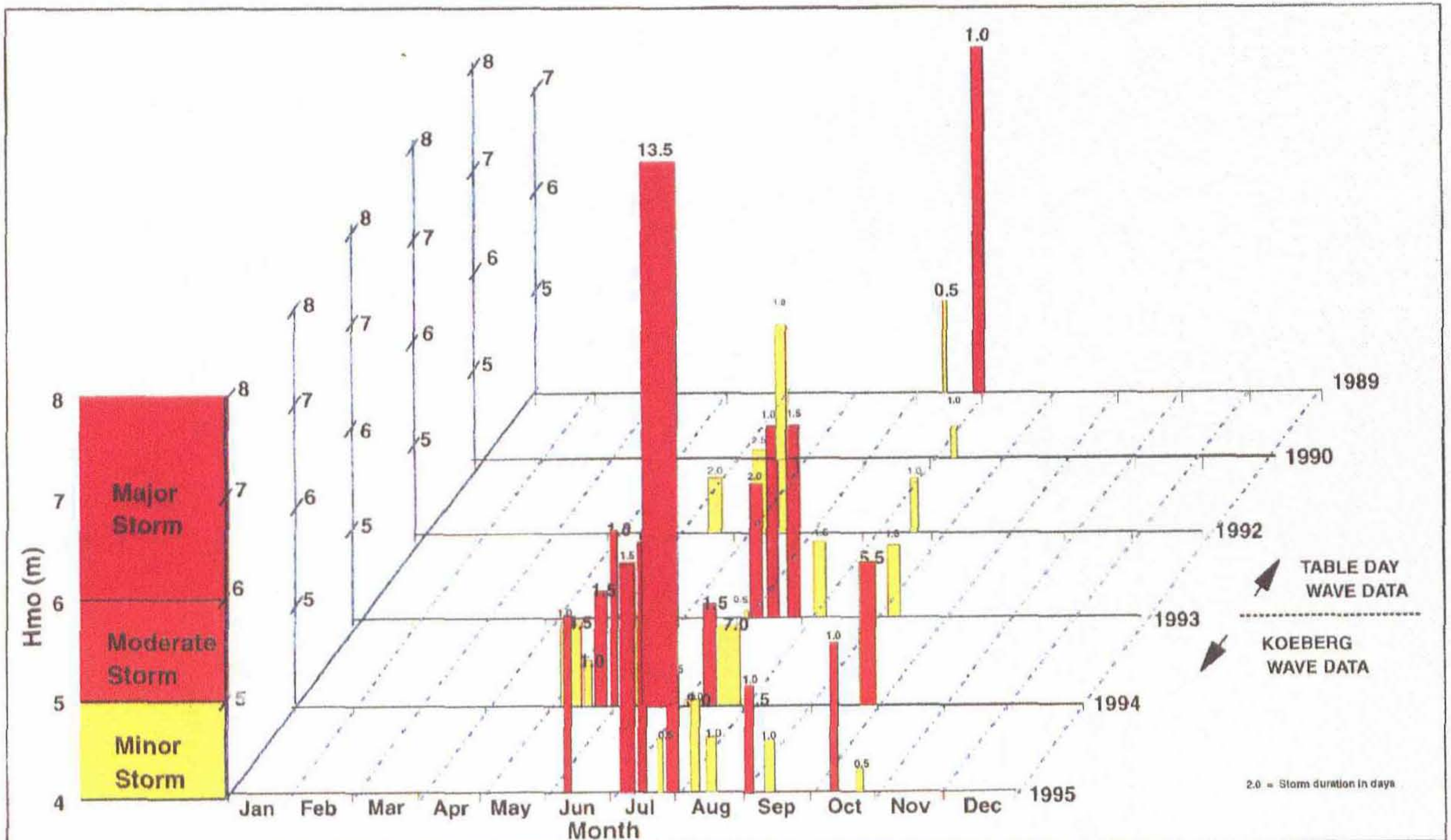
1995-06-01 to 1996-05-31

CSIR

FIGURE 2.16

WAV_TSY 1996-29-03 08:38:58

Storms recorded from 1989 to 1995



Earth, Marine and Atmospheric Science and Technology

CSIR

PORT OF CAPE TOWN

STORMS RECORDED FROM 1989 TO 1995

FIGURE

Fig 2.17

3. IMPLICATIONS OF CAISSON MOVEMENT

3.A. FAILURE OF THE CAISSON STRUCTURE

3.A.1 Failure

A breakwater may be defined as a structure constructed for the purpose of providing a water area protected from the effects of the sea so as to afford safe accommodation for shipping. (PORTNET 1993, Port Engineers Handbook) When does a breakwater structure fail? When the structure is no longer able to effectively provide safe accommodation for shipping.

In the case of the caisson breakwater there could be several stages of failure.

Least severe failure would be excessive movement of an individual caisson causing a disturbance of the monolithic ability of the caisson structure. Repairs to the caisson structure could still be carried out during this stage of failure.

Severe failure would be when a caisson is completely out of position when compared to the rest of the structure. This will definitely occur when a caisson slides or overturns. The structure would most probably experience movement of more than one caisson. Repairs would probably still be possible, but at great cost.

Complete failure would occur when one or more caissons moves out of position, causing a breach in the breakwater and allowing waves to penetrate the harbour basin.

3.A.1.1 Modes of failure

Sliding and overturning of caissons

Warning signs would normally be expected to precede the above-mentioned failures, which are caused by foundation deterioration.

The warning signs could include:

(a) Deterioration of grout socks.

(b) Visible settlement.

(c) Foundation erosion.

3.A.1.1.1 Sliding

Sliding may take place between the bedding material and the caisson, by shear of the bedding layer itself or sub foundation shear failure. (See fig 3.1)

To improve resistance to sliding failure the following should be done:

(a) Caissons should be connected with robust grout keys over the full height of the caisson. (fig 1)

(b) Frogged joints should be placed in the mass capping. (fig 1)

(c) Undersides of the caissons should be roughened to resist sliding.

3.A.1.1.2 Overturning

Overturning can be divided into in two categories:

(a) Overturning where the caisson might rotate around the inner heel of the caisson.

(b) Overturning due to a development of a slip circle in the foundation material. This is caused by wave forces exceeding the structure designed safety factors, consequently affecting the resistance moments in the foundation materials.

3.A.2 Factors influencing structural damage

The forces of wave impact must always be accommodated in breakwater design. These forces are however largely affected by circumstances present in nature or the changing environment around the breakwater.

The following can contribute to an increase in wave amplification or a decrease in wave force impacts around a breakwater structure.

(a) The topography around the structure.
(b) The shape of the structure.
(c) Wave steepness.
(d) Wave period and height.
(PORTNET 1994, Port Engineers Handbook)

What wave forces are exerted on breakwater structures?

The three most common breaker types (fig 3.2) that exert forces on the breakwater are:

(a) Plunging breaker with a large entrapped air pocket.
(b) Plunging breaker with a small entrapped air pocket.
(c) "Flip through" This is a wave that slides up against face of the structure without plunging against it.

A vertical caisson type breakwater is designed to reflect wave forces encountered. Plunging waves breaking against vertical caisson structures should be avoided at all times.

A plunging wave occurs when the crest of the wave curls over the air pocket. Breaking is usually with a crash.

In a large scale model investigation the following conclusions were made with regards to plunging breakers. (ICCE 1992, Chapter 113) The intensity present in impact loads are largely governed by the shape of the wave breaking on the structure. The most critical loads are present in double-peaked impact forces. They are present in plunging breakers with large entrapped air pockets.

This type of plunging breakers is not encountered at the Table bay breakwater in moderate wave conditions. There is however a more "flip through" wave condition with over topping. (See photograph group D)

Severe overtopping took place during a storm encountered on 19 June 1994. The wave heights were estimated by the CSIR to be as for a one in fifty year storm. Wave conditions with a significant wave height (Hmo) of up to 10 metres were experienced.

The following formula determines the relation between wave height and water depth in front of the structure and whether a plunging breaker will occur at certain wave conditions.

Hb/d	=	**0,78**
Hb	=	height of breaker
d	=	depth in front of structure
0,78	=	ratio

(CSIR 1985, report 561)

Example:
If the water in front of a caisson has a depth of -17 metres (depending on tidal variations) the wave height needs to be 13,28 metres before any plunging breakers occur.

3.A.3 Types of caissons

There are two geometrical types of caissons (See fig 3.3)

(a) Square type

(b) Cylindrical type (See fig 3.3)

The reason for the different types is to reduce wave forces and wave overtopping.

<u>The reasons for reducing impact forces:</u>

(a) To reduce impact pressures on the reinforced caisson wall.

(b) To minimise foundation loads and which may influence the stability of the structure.

<u>Why reduce wave overtopping?</u>

(a) To limit waves generated inside the harbour by waves overtopping the caisson structure.

(b) To minimise damage to equipment present on the caisson structure.

3.A.4 Consequences of foundation failure

As concluded in Chapter 2, failure of the monolithic capabilities of the caisson breakwater can be expected to lead to failure of the breakwater.

In the CSIR 1985, C/SEA 8508 (Table Bay breakwater stability calculations) report, the following was concluded:

"The vulnerability of vertical/composite type breakwaters to wave forces is often as a result of scour and undermining of the toe, or overloading and shear failure of the foundation soil."

The above are present at the Table bay caisson breakwater. Scouring has already undermined a section of foundation stone between caisson M6 and M7. (Refer to damage report annexure C).

The other factor related to damage caused by impact waves is the deterioration of the grout socks. (Refer to damage report annexure A and B).

The settlement present at the caisson breakwater may be due to the following two factors:

(a) Overall settlement of the structure - settlement or shear of the foundation soil (shale).

(b) Local deformations of the stone foundation - Movement or rocking of individual caissons due to wave action and scour.

The 1985 stability calculations performed by the CSIR had the following conclusions:

(a) The recommended safety factors were not obtained as specified by Permanent International Association of Navigation Congress (PIANC) (see fig 3.4). The recommended factors of safety against sliding failure is 1.5 and that against overturning failure is 2.0

(b) This section of the breakwater could be more susceptible to damage (The CSIR suggested a model test before any remedial action is considered)

MODEL TEST ON VERTICAL BREAKWATER SECTION PERFORMED BY THE CSIR (CSIR 1988 EMA - C 8893)

In the case of Table Bay breakwater the occurrence of extreme impact forces on the caisson extension does not exist. This was concluded in the findings as explained in the model test on the caissons, performed by the CSIR.(CSIR 1988 EMA - C 8893) When wave heights exceed about 6m, the wave energy is dissipated due to overtopping.

<u>Test results of the CSIR report</u>

During the test under certain wave conditions a standing wave developed in front of the caisson. This causes the phenomenon called clapotis. Overpressure and under pressures on the vertical face of the caisson vary between minimum and maximum clapotis levels in phase with the wave period. During this phenomena, no impact from the waves breaking against the structure occurs. When wave breaking did occur close to the structure, severe overtopping took place.

This can be seen on the series photo's of the wave conditions at the breakwater on 20 June 1996. (See photograph group D)

3.A.4.1 Storm damage statistics

Steven Bentley (1996), Technical Manager (Civil and Marine), Victoria and Alfred Waterfront (Previously employed by Portnet), observed the breakwater structure and noted changes in its condition. He commented in a personnel interview that the breakwater deteriorated after 1985. Prior to this date little or no deterioration was observed in the Rubber joints (fig 1) on top of the structure and the grout socks.

During 1984 and 1985 severe winter storms battered the Cape causing damage to the dolos protection of the breakwater.(See 25 ton Dolos, fig 3.6)

S Bentley also mentioned that the damage to the grout socks and joints of the Breakwater occurred during the course of one year, between 1984 and 1985. He continued to monitor the caisson section until he was transferred to another division of Portnet. Unfortunately the records of these surveys were somehow misplaced and are not available.

Taking Bentley's report into account, it can be assumed that the breakwater caissons were damaged in severe storms during 1984/5. It can therefore be assumed that the deterioration of the caisson structure took place from the date of those severe storms up to present.

3.A.5 Prediction of the remaining useful life

With the information obtained from Bentleys report it can confidently assumed that the following holds true:

(a) Each caisson capping was constructed 'level' relative to each other.

(b) Deterioration of the caisson structure and subsequent settlement took place after 1984.

(c) It can be assumed that no movement took place on the blockwork section of the breakwater, where the caissons connect with the remainder of the structure.

Using the levels taken from 1993 to 1995 a average cumulative settlement between all seven caissons was calculated. This figure average to approximately 4 mm per year. With this information an attempt will be made to predict the remaining useful life of the caissons, if the current rates of settlement of the structure continues. This will vary slightly from year to year depending on the winter conditions experienced.

The settlement of the caissons would most probably increase gradually as the condition of the structure deteriorates. As the structure deteriorates the likelihood of a increase in movement might be more apparent. This could damage the foundation and cause a even greater increase in settlement. In addition to this increase in settlement, the monolithic ability (performed by the grout socks) will also have an influence. Any future predictions on the settlement and the remaining design life of the structure, will require the following:

(a) A limit at which the settlement will adversely affect the monolithic ability of the structure.

(b) An assumption that settlement will continue to take place at the same annual rate as presently experienced.

(c) A decrease in the sliding and overturning factors of safety as the settlement increases and the structure becomes less stable.

All the above factors will influence the remaining useful life of the caisson structure. (See fig 3.5)

In figure 3.5 the average accumulative settlement increase over the monitoring period has been plotted against time. All the levels were taken on the brass pegs.

In 1993 the average settlement over all the levels taken on the brass pegs was, 67 mm on the sea side and 85 mm on the harbour side. Could the settlement continue to increase and reach a figure of 75 percent more than the present figure by the year 2010 (35 years after construction)? How much will a 75 percent increase affect the structure? If it does affect the structure, a reduction in the useful lifetime of the structure and a increase in structure maintenance will take place. To predict exact figures is at present only speculation.

The following conclusions were drawn from the CSIR report with regards to the situations experienced: (CSIR 1985, C/SEA 8508)

(a) The breakwater was constructed in sufficiently deep water to prevent any plunging breakers occurring. The primary attack against the breakwater is from non-breaking reflective waves and clapotis formation(See definitions). The forces on the breakwater are preliminary quasi-static and are caused by the standing wave effect.

(b) The overturning moment does not increase with wave height to the extent predicted. Severe overtopping for higher wave heights also contributes to the reduced rate of increase in wave forces.

(c) The caisson breakwater structure is safe against sliding and overturning in 1:100 year design wave conditions. (Hs = 7.5m) The factor of safety against overturning is 1,6 and against sliding 1,2. This was calculated by the CSIR (CSIR C/SEA 8508).

The PIANC-recommended values are 2,0 and 1,5 against overturning and sliding. The caisson breakwater safety factors does not comply with the recommended theoretical values of PIANC. The CSIR felt however that greater confidence could be placed in model tests results than theoretical values.

3.A.6 Summary

The design of the caisson breakwater was done taking into account all the important factors present in breakwater design. There are however problem areas developing especially with regards to the foundation of the caisson extension.

It can be assumed that the damage pattern present in the structure originated as a result of the bad storms during 1984/85.

Storms exceeding the design profile of the structure have not been able to cause serious damage to the structure. The CSIR findings, regarding to the design safety factors, can therefore be accepted as factual.

The biggest concern is the continuous settlement of the entire structure. This factor will most probably shorten the expected useful life of the caisson extension.

To predict an exact period of time by which the useful life of the structure would be reduced, would be speculative. It is however conceivable that the structure will require major reconstructive repairs before the end of the design lifetime is reached. It can only be estimated what the total predicted settlement of the structure will be if the present rate of settlement continuous. (see fig 3.5)

3.B REPAIRS TO THE CAISSON STRUCTURE

3.B.1 Maintenance repairs

Possible repairs to the caisson structure can be divided into the following categories.

(a) Restriction of caisson movement.
(b) Repairs to caisson grout socks.
(c) Repairs to damaged concrete.

It is recommended that the above-mentioned repairs should be performed to extend the useful life of the structure.

3.B.2 Restriction of caisson settlement

To limit settlement of the structure is only possible if the cause of settlement is due to erosion around the structure.

If settlement is due to failure of the sub soil layers very little preventative measures will be possible.

The most likely cause of caisson settlement is settlement of stone bed layers or sub soil layers. (see fig 2.8)

3.B.3 Repairs to caisson grout socks

The importance of the grout socks has been shown to be critical in the Table Bay breakwater situation. Water surging between caissons due to the absence of grout socks could be responsible for an increase in stone bed disturbance eventually causing settlement damage.

Repairs to grout socks might be very difficult. Marine growth is present and would need to be removed before any repair work can be undertaken.

3.B.4 Repairs to damaged concrete

The seaward face of the breakwater is important as the impact of wave forces has severe effects on this part of the structure.

The concrete capping has cracked due to alkali aggregate reaction and will probably eventually crumble as a result of impact forces on the structure.

3.C POSSIBLE SOLUTIONS TO THE PROBLEMS ENCOUNTERED

3.C.1. Repairs to the rubber membranes

It is recommended that the rubber membranes need to be reinstated to their original condition so as to serve the function they were designed for, to distribute wave forces from one caisson the next.(See fig 1) Due to the deformation of the membrane slots caused by settlement of the caisson capping, these will need to be reinstated. New slots will have to be cut into the concrete to obtain a tight fit for the membranes.

3.C.2 Repairs to grout socks

Not all the grout socks need repairs. The marine growth actually prevents further damage by forming a protective layer around the concrete sock. Immediate repairs must be undertaken where grout sock damage is close to the foundation. Important repairs are those to the damaged grout sock between caissons M7 and M6.
(See Annexure C)

Regular inspections should be carried out to check for grout sock damage.

3.C.3 Repairs to stone foundation

Surface repairs to the stone foundation can be undertaken with relative ease. Any shear failure of foundation and sub-foundation material is very difficult, if not impractical, to repair. It is not known if any foundation shear is taking place, but there is definite settlement of the foundation.

Until the cause of settlement has been clearly established, the prevention of foundation settlement is not practical.

3.D FINANCIAL IMPLICATIONS

3.D.1 Actual costs for maintenance repairs

The estimated cost to do maintenance repairs to the caisson breakwater are as follows. This will not include complete rehabilitation of the grout socks. These complete repairs to the grout socks might be required at a later stage as increased deterioration affects the ability of the socks to maintain the monolithic form of the structure.

D.1.1 Repairs to rubber seals in concrete capping
R 10 000

D.1.2 Repairs to grout socks (10 metres of grout sock repair)

(a) Cleaning of area for repairs
R 10 000

(b) Grout sock repairs
R 20 000

D.1.3 Concrete repairs
R 8 000

Total estimated material cost for maintenance repairs performed departmentally is R 48 000 (1997)

3.E Conclusion

(a) Sliding and overturning are not the major factors influencing the failure of the caisson breakwater. The caisson section has proven itself in wave conditions exceeding the design wave heights.

(b) According to Bentley, damage to the caisson extension occurred during the storms of 1984/85. Visible settlement of the caissons has been taking place since then. The estimated cumulative settlement is approximately 4 mm per year.

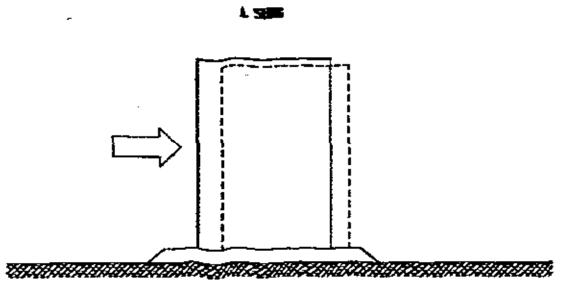
(c) The current settlement experienced can only be due to top foundation layer settlement. To identify the type of foundation layer settlement is not yet possible. No signs of foundation settlement are visible under water, except for localised foundation failure due to grout sock failure.

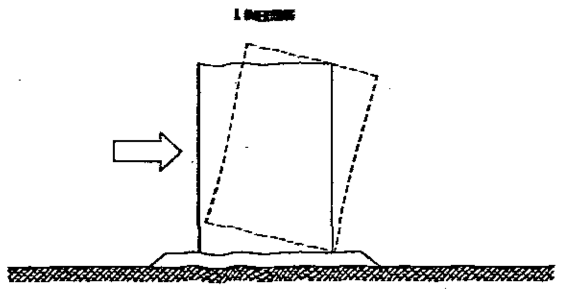
(d) If the present rate of settlement continues, the useful life of the caisson breakwater could be shortened due to failure of the structure. Regular inspection is therefore essential to maintain the structure in good condition.

An approaching wave,
South West direction,
overtopping at
caisson M3.

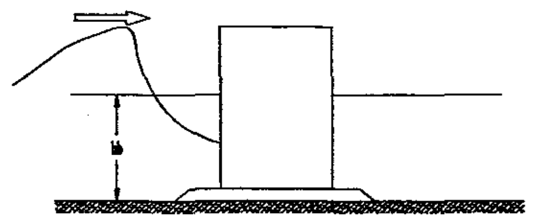
The wave overtopping continues
to caisson M7. (20 June 1994)

Photograph group - D

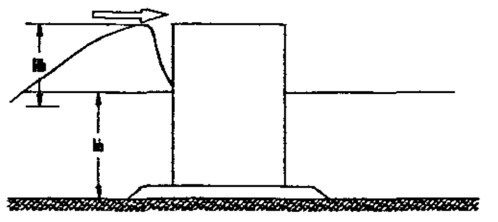




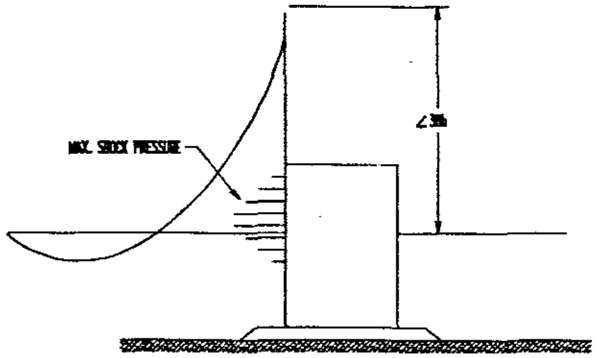
MODES OF MAJOR FAILURE OF VERTICAL BREAWATERS (GODA,1985)



PHASE 1:
Wave approaching the wall



PHASE 2:
Wave before hitting the wall

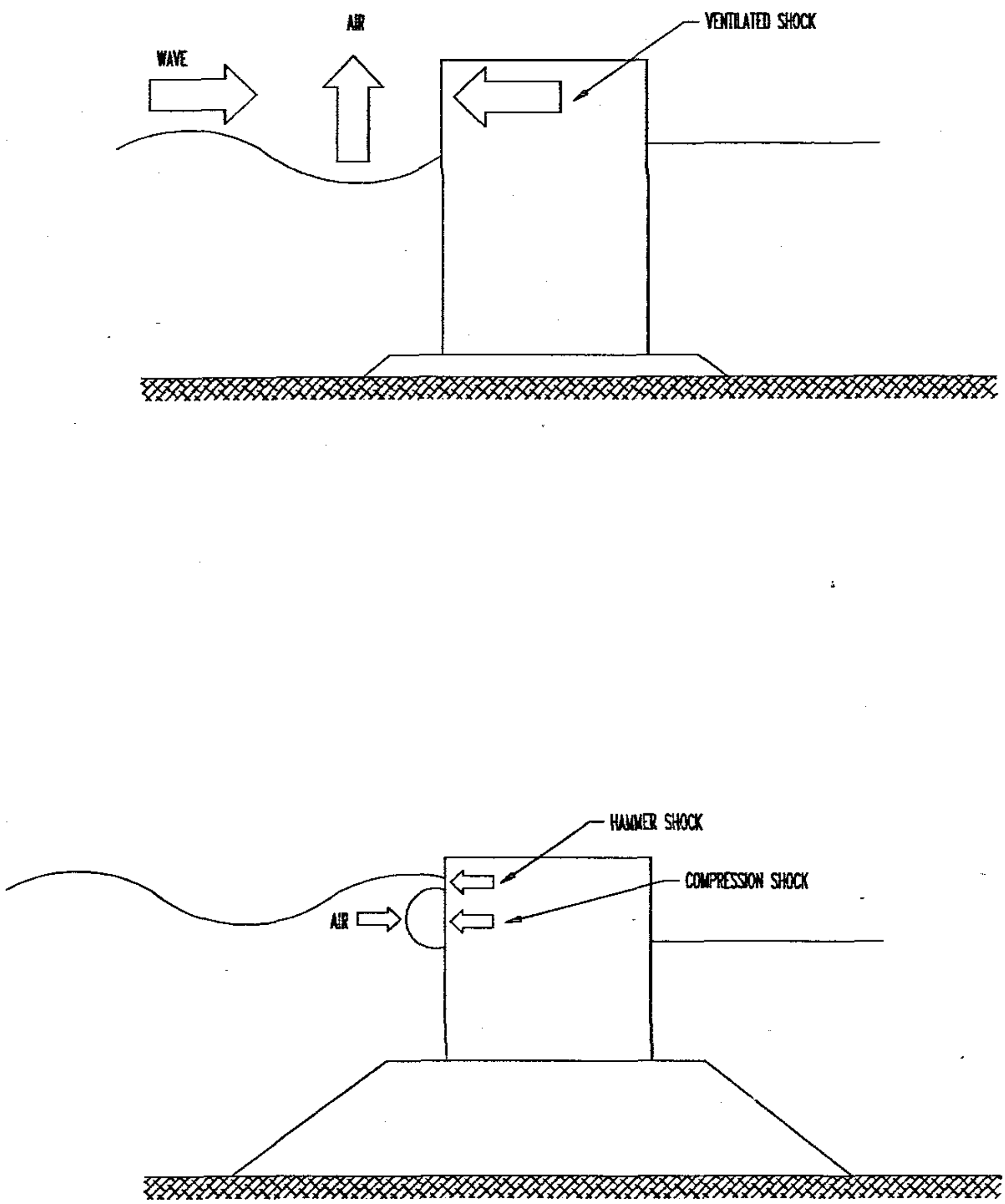


PHASE 3:
Maximum wave impact on the wall

TYPICAL SITUATION FOR HIGH IMPACT LOADING

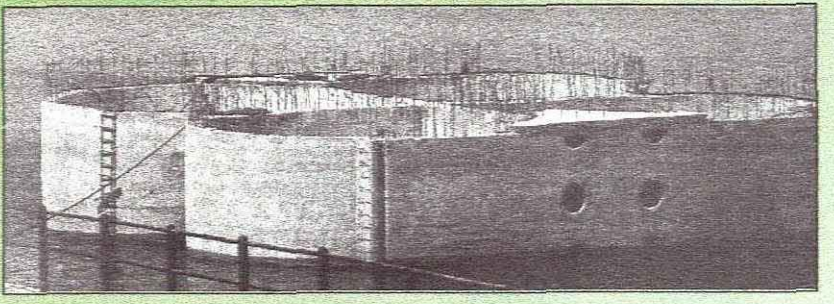
Modes of failure - Vertical Breakwaters

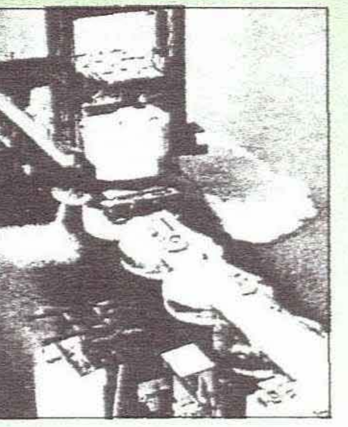
Fig 3.1



Types of shock forces

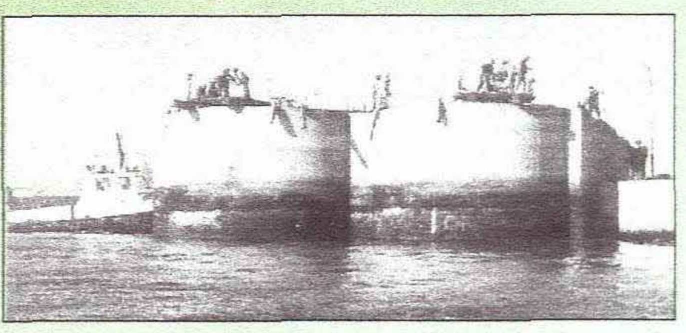
Fig 3.2



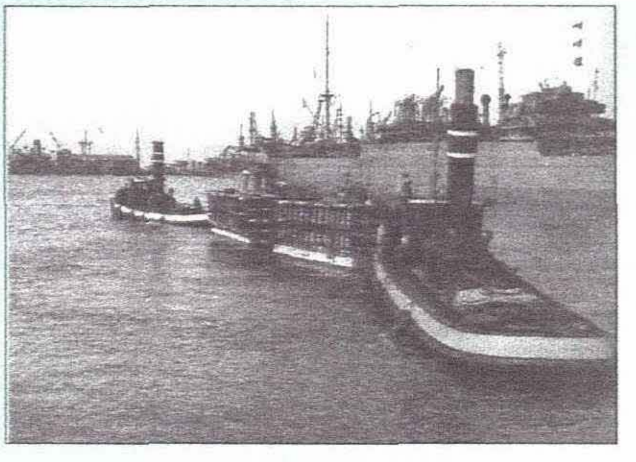


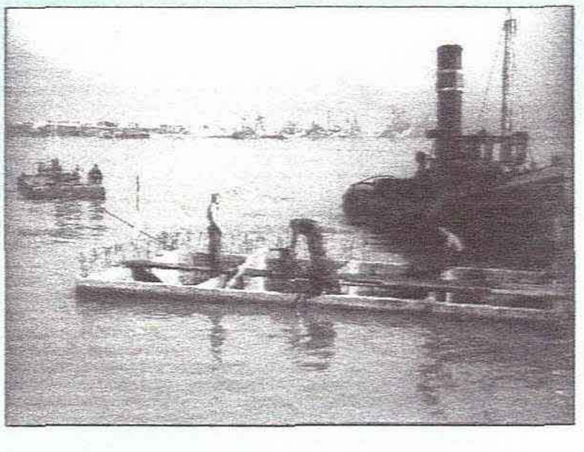
CYLINDRICAL CAISSONS

The above insert is of circular reinforced concrete caissons. (Hanstolm, Denmark) (Port Engineering - Breakwater, Jetties and Piers)



Circular segmented caissons where used for the construction of the Main Breakwater extension. These caissons have a square appearance from a side elevation but are a combination of four circular shaped sections. The purpose of the circular shape is to reduce the effects of wave impact on the structure.



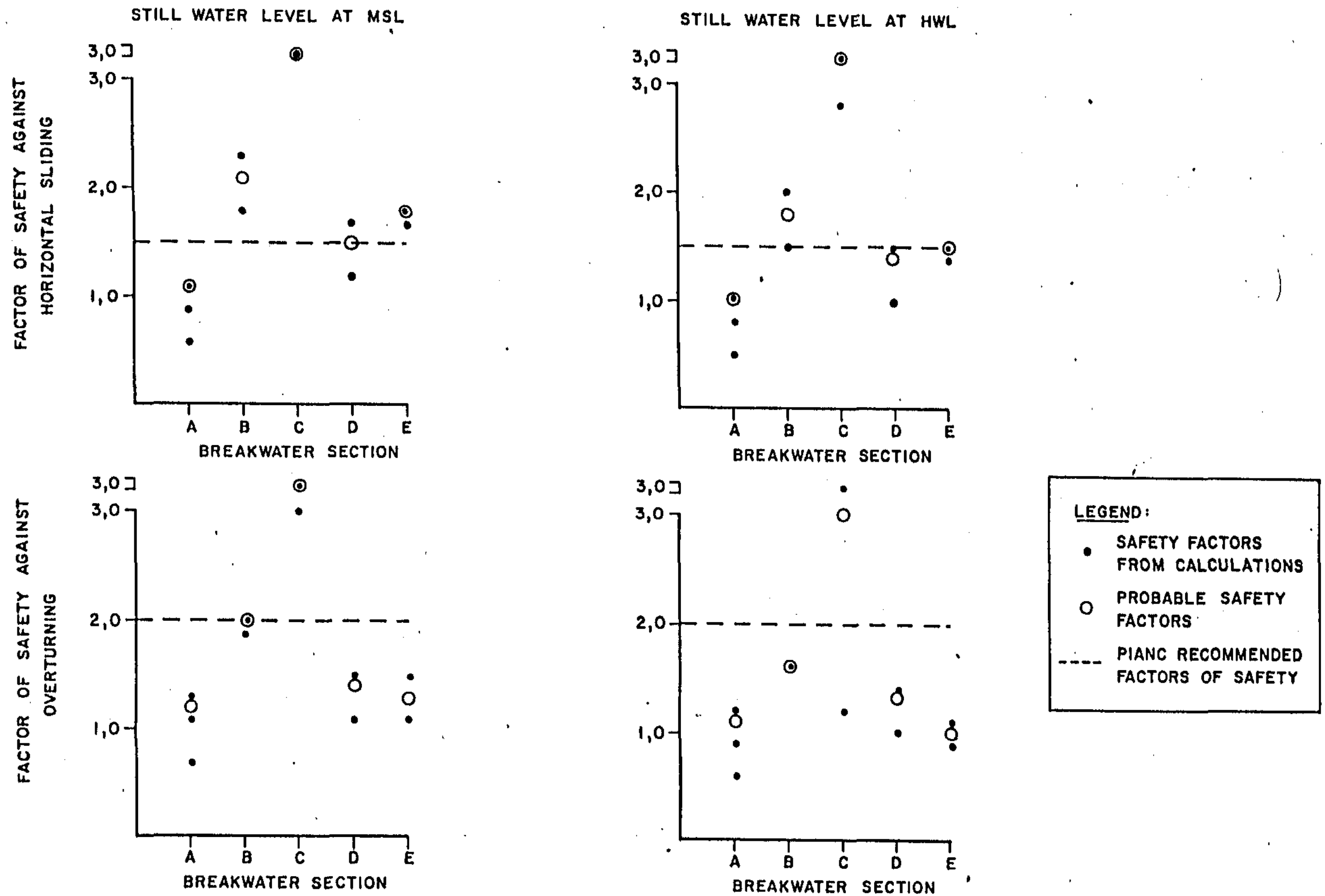


SQUARE CAISSONS

Square caissons were used for the majority of jetties in the Port of Cape Town.

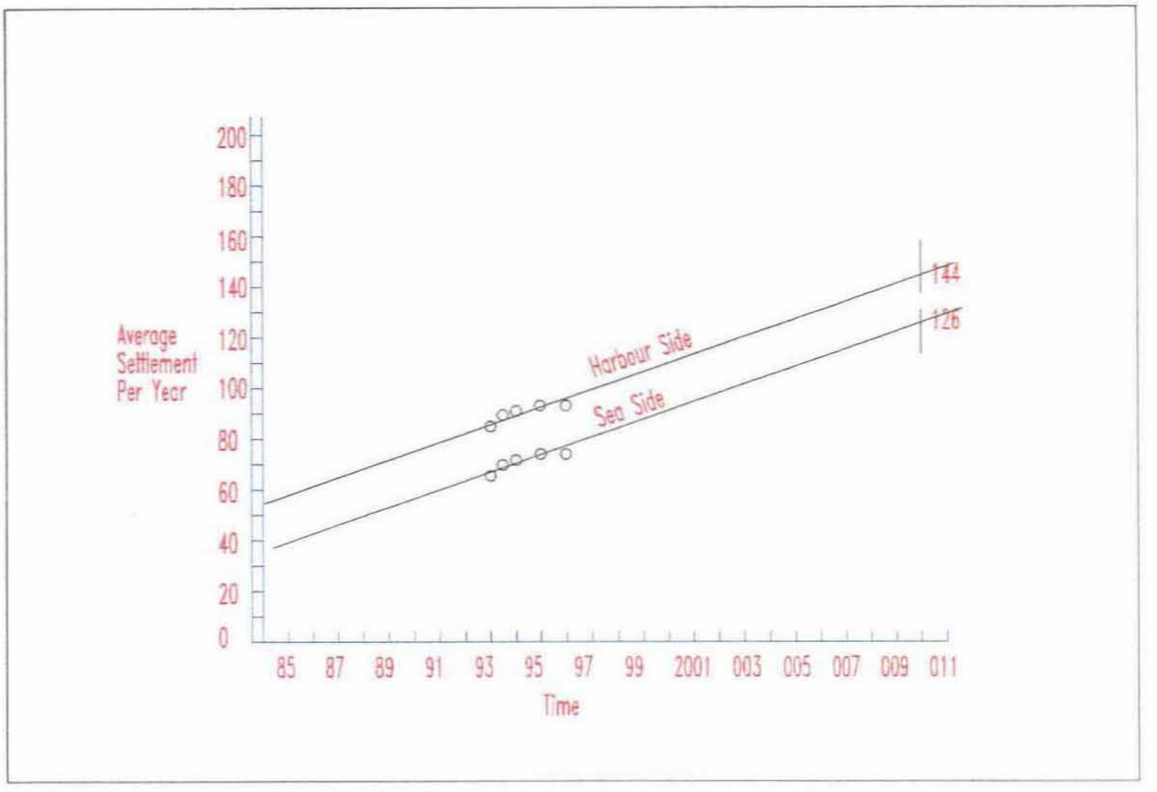
Types of Caisson construction

Fig 3.3

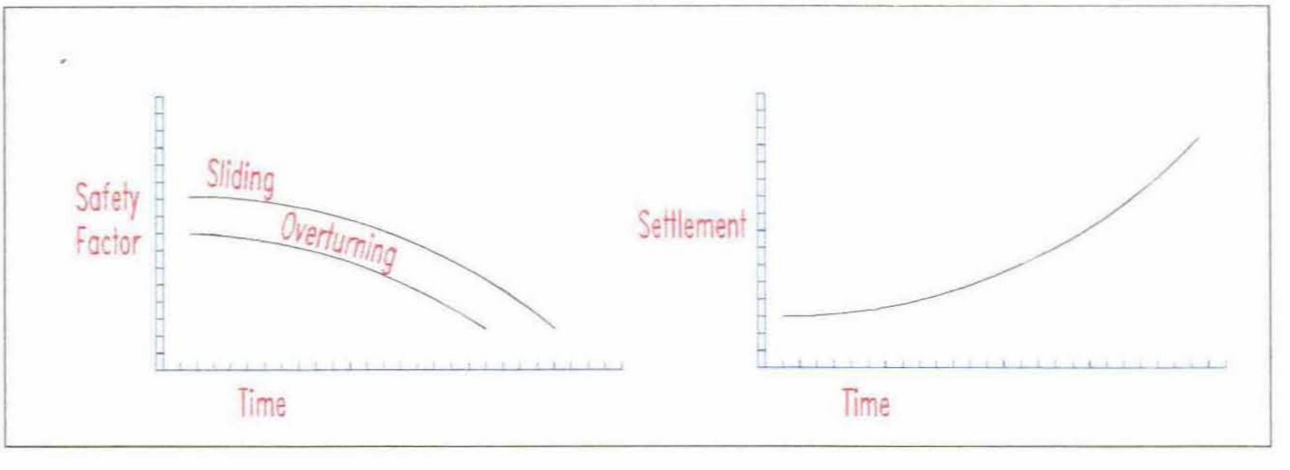


CSIR Report-Summary of obtained factors of safety

Fig 3.4



Possible predicted accumulated level settlement on the caisson corners, over the design lifetime of the structure.



Possible predicted safety factor reductions trends, due to caisson structure deterioration.

Safety Factor Reductions & accumulative Settlement.

Fig 3.5

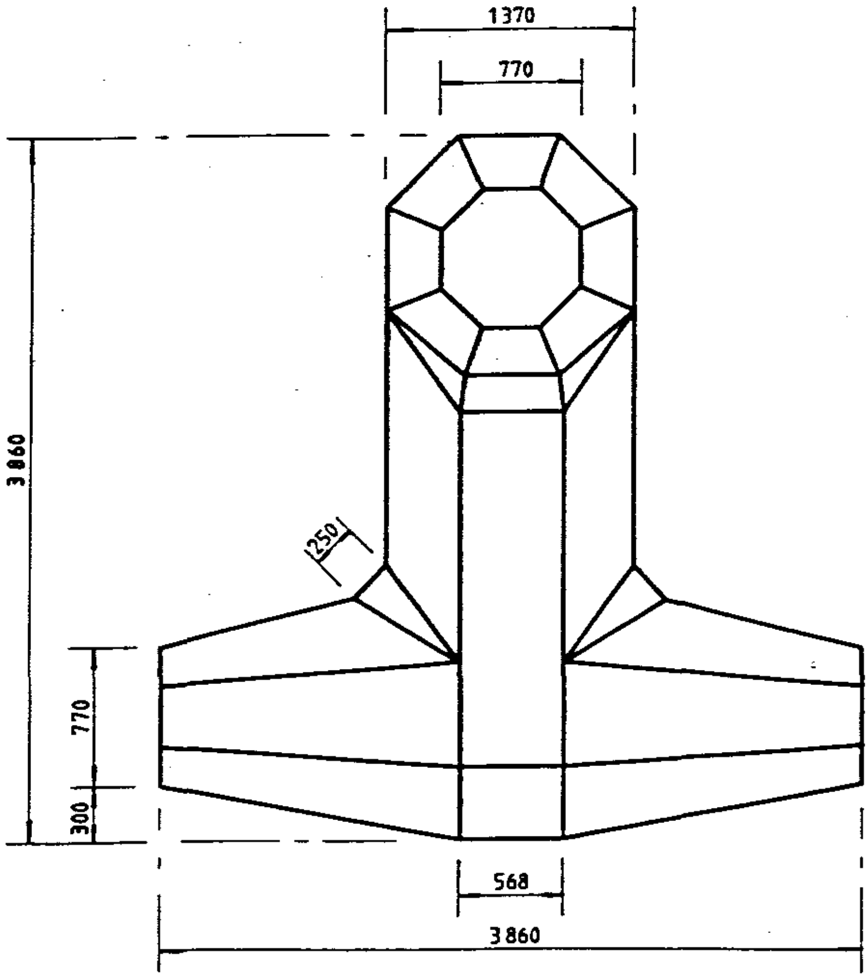


FIG. 7.22 - TYPICAL 25t DOLOS USED AT TABLE BAY HARBOUR BREAKWATER.

HOO. CT.

25 ton Dolos

Fig 3.6

4. TOTAL FAILURE SYNDROME

4.A FAILURE OF THE STRUCTURE

4.A.1 Introduction

For the purpose of this investigation, the following caisson failure scenarios were simulated to establish the effects on the Ben Schoeman Dock (protected by the breakwater , see fig 2.5 and fig 4.1) and the consequences thereof.

The Ben Schoeman Dock is used primarily for handling container vessels. De Beers marine, a diamond exploring company, also occupy a small area in the Ben Schoeman Dock. Facilities are available for the loading and off loading of containers, including areas for storage of containers.
(See photograph group E)

There are four standard size container cranes and two larger Panamax type cranes. The Panamax cranes were developed to handle the new generation, larger container ships.
The Dock water area covers 112,7 hectares.
(See fig 4.1)

4.A.2 Possible failure modes

In 1995 a small investigation was requested from the CSIR to determine the wave diffraction behind the breakwater and probable wave scenarios, if the breakwater should fail. This investigation was conducted by Mr H.Moes of the CSIR. (CSIR 1996, EMAS-C 96016)

The Main Breakwater extends 30 degrees north east into Table Bay. The water depth at the structure is approximate 15 meters. The wave periods encountered vary between 8 and 18 seconds. A wave period is the time lapse between successive crests passing a given point.

The wave range periods which are important to coastal engineering, are the gravity waves, with periods between 5 and 18 seconds. (U.S Army 1997, Shore Protection Manual)

Table: Wave lengths in different water depths with different wave periods.

Wave period (s)	Water Depth 15m	Water Depth 17m
8	82 m	85 m
10	109 m	114 m
12	135 m	143 m
14	161 m	170 m
16	186 m	197 m
18	212 m	224 m

The following factors influence the wave penetration into the Ben Schoeman Dock:

(a) Refraction
(b) Diffraction
(c) Reflection

(PORTNET 1994, Port Engineers Handbook)

These wave forms need to be evaluated in order to obtain correct information on wave penetration.

4.A.2.1 Wave refraction

The first factor which affects the direction of wave approach towards the breakwater, is the refraction of waves as they approach the coastline.

The Shore protection Manual (U.S Army 1977) explains wave refraction as follows:

"Wave celerity depends on the depth of water in which the wave propagates. If the Wave celerity decreases with depth, wavelength must decrease proportionally. Variation in wave velocity occurs along the crest of a wave moving at an angle to underwater contours because that part of the wave in deeper water is moving faster than the part in the shallower water. This variation causes the wave crest to bend towards the alignment of the contours."

This bending effect is known as wave refraction. (Fig 4.2)

Wave refraction tables for Table Bay were obtained from CSIR Report 561 "Deep-sea and near shore wave conditions for 15/16 May and 1/2 July 1984 storms". (CSIR 1985, Report 561)

Different refraction tables are available for different wave periods. The most significant wave periods for Table bay vary between 10 and 16 seconds. For each wave period there will be different wave directions which will affect refraction of the waves. (CSIR 1985, Report 561)

The following assumptions are made with reference to refraction. Shore Protection Manual (U.S Army 1977).

(a) Wave energy between the orthogonal remains constant.

(b) Direction of wave advance is perpendicular to the wave crest.

(c) Speed of a wave of a given period at a particular location depends only on the depth at that location.

(d) Changes in bottom topography are gradual.

(e) Waves are long-crested, constant-period, small-amplitude, and monochromatic.

(f) Effects of current, winds, and reflections from beaches, as well as underwater topographic variations, are considered negligible.

4.A.2.2 Wave diffraction

Wave diffraction takes place when waves encounter partial obstruction / breakwater. Subsequent wave bending effects around the obstructions are encountered. (See fig 4.4)

Diffraction of waves is the most important factor affecting wave heights in the Port.

In the Shore Protection Manual (1977) diffraction tables are available for different wave directions.

The following assumptions are made in the development of diffraction theories:

(a) Water is an ideal fluid.

(b) Waves are of small amplitude and can be described by linear wave theory.

(c) Flow is irrational and conforms to a potential function which satisfies the Laplace equation (Shore Protection Manual 1977).

(d) Depth shorewards side of the breakwater is constant.

4.A.2.3 Wave reflection

Wave reflection is the phenomenon whereby waves can be partially or completely reflected from man-made or natural barriers.

4.A.3 Nearshore wave conditions at Table Bay breakwater

Major storm conditions were experienced during the months of May and July 1984. The CSIR compiled a report describing the wave conditions experienced. (CSIR 1985, Report 561)

The figures in this section are based on the results obtained from the CSIR report. We assume that the wave height had the following characteristics:
Hmo = 10 meters
Wave Period (Tp = 16 seconds) out of a West ,South West direction. (See fig 4.4, Refraction diagram)

From the CSIR tables the following information was gathered for a WSW'ly direction wave approach:

Mean refraction Coefficient = 0.41
Mean wave front inclination = 31 degrees, relative to the breakwater axis. The shoaling coefficients are derived by using a contour plot of the area.
(Fig 4.3) The shoaling coefficient for a wave period of 15.5 seconds in 15 metres of water is 1.07
The coefficient will have a minor impact on this coefficient if the wave

period is 16 seconds. The deep-sea maximum wave height ,

M max = 1.95 x H mo

(CSIR 1985, report 561)

H mo = 10 metres

H max = 1.95 x 10 metres

= 19.5 metres

Taking the refraction coefficient into account the maximum wave height.

(H max)at the breakwater will be :

H max = (0.41) x (1.07) x (19.5)

= 8.6 metres

4.A.4. Wave penetration as a result of caisson failure

The Ben Schoeman Dock is protected by the caisson extension to limit wave penetrations into the Harbour basin. What then, would be the impact in the case of complete failure of the caisson extension? Let us consider this question.

The first step is to calculate the present wave penetration into the port with the caisson extension still in place.

(See fig 4.5)

Example:

The wave length of a wave with a period of 16 seconds is 186 metres in a water depth of 15 metres.

The wave height is 8.6 metres as calculated from the refraction tables.

The distance from the tip of the breakwater to the entrance of Ben Schoeman Dock is 1000 metres at an angle of 92 degrees relative to the breakwater. One wave length is 186 metres long. The angle of wave incidence of the tip of the breakwater is between 142 degrees (SW) and 105 degrees (NW) relative to the breakwater. The wave approach for the WSW'ly storms will be approximately 135 degrees to the breakwater. Using the diffraction diagrams (U.S Army 1977) we will determine the wave height at the Ben Schoeman Dock, Duncan Dock and the Alfred Basin entrance for the 8.6 meter waves outside the breakwater.(See fig 4.5 and 4.6)

The 135 degree diffraction diagram can be seen at fig 4.5. Each arch from figures 1 to 10 represent a wave length. The distance from the breakwater to the BSD entrance is 1000 metres. The wave length of a 16 second period wave is 186 metres. (See previous table). That represents 5.4 wavelengths on the diffraction diagram. Using the direction and the scale of the harbour plan, can we determine the k' factor.

Diffraction diagram:

The direction towards the BS Dock is 92 degrees.
The number of wave lengths 5.6.
Therefore the k' factor = 0.125

The formula for diffracted wave height: H = k'. Hi (U.S. Army, 1977)

H = k'. Hi
= 0.125 x 8.6 m
= 1.075 metres at the Ben Schoeman Dock entrance

This represents a 87% reduction of wave height.

H = 0.10 x 8.6 m

= 0.86 metres at the Duncan Dock entrance

H = 0.075 x 8.6 m

= 0.645 metres at the Alfred Basin Entrance

If complete failure of the caisson extension should take place the wave heights will be as follows (See fig 4.6):

H = k'. Hi

= 0.15 x 8.6 m

= 1.29 metres at the Ben Schoeman Dock entrance

H = 0.12 x 8.6 m

= 1.032 metres at the Duncan Dock entrance

H = 0.075 x 8.6 m

= 0.645 metres at the Alfred Basin Entrance

The Ben Schoeman Dock entrance will also diffract the reduced wave height so that a even smaller wave will be experienced at the quays of the dock. To calculate the diffraction at the dock entrance the following tables in fig 4.7will be used. (U.S Army, 1977).
These diffraction diagrams use the wave approach angle, wave length and the entrance width to determine the specific k ' factors.

The k ' value and calculated wave height at the quays for the situation where the caisson extension are still in tack are:

H = 0.1 x 1.075

= 0.1075 metres

The k ' value and calculated wave height at the quays for the situation where the caisson extension had complete failure are:

$$H = 0.1 \times 1.29$$
$$= 0.129 \text{ metres}$$

The increase in wave height due to caisson failure is very small.

The worst scenario would however be a wave approach from a North Westerly direction.
Lets assume the same wave height as previously calculated (8.6 metres) but from a North Westerly approach. (diffraction diagram 105 degrees)
At the Ben Schoeman entrance the wave height will be:

$$H = 0.4 \times 8.6 \text{ m}$$
$$= 3.44 \text{ metres}$$

At the quays:

$$H = 0.1 \times 3.44$$
$$= 0.344 \text{ metres}$$

The wave height effects in the Ben Schoeman Dock showed very little increase due to the failure of the caisson extension.

4.B FINANCIAL IMPLICATIONS TO THE PORT

4.B.1 Operation downtime as a result of wave effects

In chapter 4A it was established what the percentage wave increase would be in the event of complete failure of the caisson extension. We have seen that the wave increase would be very small inside the Ben Schoeman Dock.

The following predictions have been made using the information available. Furthermore, the effects that the new wave conditions will have on the container vessels loading and off-loading efficiency, will furthermore be considered. These will exclude the effect of long period waves, resonance and refraction.

4.B.1.1 Container handling downtime (short period waves)

The following forces are of importance to a moored vessel.

- Current
- Wind
- Astronomical tide
- Passing ships
- Loading and unloading operations
- Waves
- Resonance

(PORTNET, 1994)

The movement of a vessel at a berth can be either horizontal or vertical. Horizontal ship movements are known as surge, sway and yaw. Vertical ship motions are roll, pitch and heave. (See fig 4.11)

Horizontal movements of a vessel are dependent on the mooring configuration of a vessel whereas vertical movements are almost independent of the mooring system.

A factor investigated in this study is the influence of waves on container operations. The present effects of wind on the container operations will also be looked at.

Container cranes are unable to operate when wind speeds exceed 40 knots. The danger of operating a container crane in such conditions, is the inability to brake the crane due to wind forces. The cranes have high wind resistance factors and can be pushed along the crane rails without the ability to brake itself. In addition to this danger, the movement of the ship and the containers also affects operational efficiency.

See figure 4.8 to 4.10 for attached graphs of losses to the container operations due to wind.

The attached graph (fig 4.9 and 4.10, percentage weather delays) represents delays due to high winds causing operations to be halted, expressed in percentage crane hours.

The Vertical motions of a vessel are dependant on wave motions.
If an excessive amount of movement is present when loading procedures are in progress, the efficiency of the procedure will be affected.

In addition to the effects of waves on ship operations, efficiency of loading operations are also controlled by the skill of the container crane operators.

The following table represents the allowable movement of a container vessel and the percentage efficiency of container operations as specified in a PIANC report. (PIANC, 1995)

	SURGE (M)	SWAY (M)	HEAVE (M)
100% EFF.	1.0 M	0.6 M	0.8 M
50% EFF.	2.0 M	1.2 M	1.2 M
	YAW (DEG/S)	PITCH(DEG/S)	ROLL(DEG/S)
100% EFF.	1.0 DEG	1.0 DEG	3 DEG
50% EFF.	1.5 DEG	2.0 DEG	6 DEG

With the assistance of Mr H Moes from the CSIR, it has been established that the critical wave height for container vessels in the BSD is 0.8 metres. This implies that a wave height of 0.8 metres inside the Ben Schoeman Dock will not effect the efficiency of container cranes due to ship movement of large container vessels. When a wave height larger than 0.8 metres is experienced, the efficiency of the container cranes will be affected by 50 percent. That implies that ships will experience delays due to wave motions having a affect on crane operation efficiency. When a wave height of 1.75 metres are exceeded inside the BSD, container crane operations will have to be stopped.

This is a very simplified way of explaining the effects that waves have on ship operations. The effects that waves have on moored vessels is a study in itself. The assumptions where made therefore made keeping this fact in mind.

4.B.1.2 Container handling downtime (long period waves)

We assume that short wave periods are unlikely to have any major effects on the ship motions inside the BS Dock. There are however down time losses in container crane handling due to ship movements.

These movements are recorded by the crane drivers when difficulty is experienced in locking onto a container aboard a ship. The total amount of down time was 30 minutes for the entire year of 1996.(See Annexure "D"). Would this figure increase if the caisson breakwater were severely damaged?

The CSIR was consulted to furnish their comments with regard to possible ranging effects as a result of caisson failure.

In a personnel interview with Mr D Phelp of the CSIR, who has been involved with the majority of wave studies done on Table Bay harbour, the following information was obtained:

"The main direction of the very long wave period is from the south west. These waves are mostly caused by slow moving low pressures (cold fronts), which have a very long unbroken fetch.

Although these long period waves will refract in deeper water and will therefore be more perpendicular to the breakwater than the shorter period swell, their direction is not as critical in determining the amount of wave energy (which will excite the range action inside the harbour). Thus the increase in the range action resulting from a particular length of the breakwater extension (caissons) which may fail, is independent of the long wave action.

The increased width of the harbour entrance, resulting from breakwater failure, will allow more wave energy through to the mouth of the BS dock, which may result in the ranging inside the dock felt sooner. This could result in a slightly longer downtime. Range action which effects container ships has a wave period longer than 50 seconds and this is more affected by the shape, size and depth of the dock, than by the height of the swell at the entrance to the dock."

It must be noted that the downtime experienced due to range action was only during loading and off-loading of vessels. This figure will increase as container vessel traffic increases.

D Phelp further said that; " The effect of breakwater failure on the range action inside the BS Dock will therefore be small, and mostly in the quicker reaching of the wave resonance, rather than a higher amplitude wave or worse ranging. However, ship motions due to swell waves in the BS Dock may increase by about 15% which could result in more downtime for container ships "

H Moes conducted a study (Wave conditions to the entrance channel to the Port of Cape Town, CSIR,1996) in which it was concluded that an increase in the breakwater length by 130 metres would cause a 15% reduction in swell wave heights at the entrance to the BS dock. Therefore, the estimated increase of wave action inside the BS dock due to caisson breakwater failure would be approximately 15%.

4.B.2 Financial implications

In the event of complete failure the only financial losses would be the repair or replacement of the caisson structure. The container terminal would experience very little inconvenience due to the possible failure of the caisson breakwater.

Ship operations will be affected with regards to marine operations. That is the steering of vessels and rendering of tug assistance due to the reduced length of the breakwater.

4.B.3. Replacement of the caisson structure

The construction cost of the caisson structure in 1974 was approximately R2 000 000. This figure was an estimate obtained from engineers who were involved with the construction of the breakwater at that time. The exact costs were not available. The replacement value of the structure in 1997 would be approximately R50 000 000.

4.B.4 Summary

It has been concluded that the possibility of waves affecting operations in the BSD, even if the caisson extension never existed, is minimal for short period waves. That is if the wave height of 0.8 metres is used as a cut off height where waves will have an effect on container handling operations.

For long period waves, the effects of range inside the BSD are very small. If the breakwater extension should fail there may be a 15% increase in the possible occurrence of range effects in the BSD. It can therefore be assumed that there will be little or no financial implications to the container terminal due to excessive wave heights affecting container handling operations. Comparison to wind delays in the Port, the delays due to waves will be negligible.

4.C EFFECTS ON THE ENVIRONMENT

4.C.1 Environment

What effects will the failure of the caisson structure have on the environment?

These effects will depend largely on what the caisson extension to the breakwater was designed to protect. This was primarily the protection of the Ben Schoeman Dock against excessive wave action and to allow for easy manoeuvring of vessels in the shadow of the breakwater.

The effect of total failure of the caisson extension will depend on the type and number of ships in the Ben Shoeman Dock at the time of the failure.

4.C.2 Possible scenarios

Breakwaters most often fail during adverse weather conditions. The effects of resonance and strong wind conditions could have a effect on shipping.

These conditions increase the possibility of ship moorings breaking. If visibility is limited and strong winds prevail, a critical situation could develop. Ships breaking their moorings in bad weather could cause serious damage to other vessels and to quay walls. (See fig 4.11)

4.C.3 Summary

There could be some minor environmental threats to the Port itself. If the caisson structure should fail completely, vessels could also be damaged resulting in possible oil spills. Considering the predicted wave height in the BSD in the event of caisson failure, the chances of any major environmental threats is unlikely.

If strong winds and wave conditions are experienced simultaneously, the scenarios could be worse and shipping could be affected.

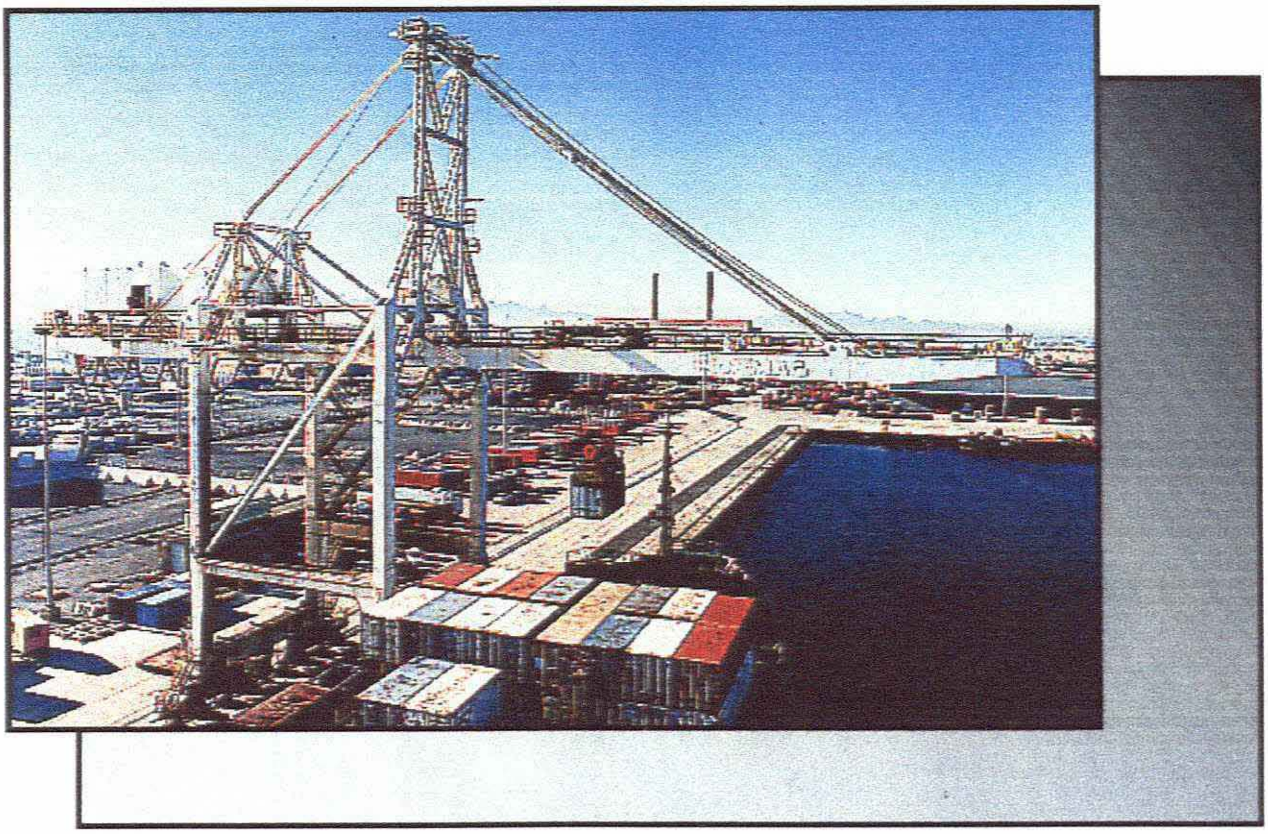
4.D CONCLUSION

Failure of the caisson extension may have the following effects on Port operations:

(a) The possibility of container handling operation delays due to short wave action inside the BSD, is small. However, if such a condition should occur, the port would most probably be closed due to the high winds which would be experienced.

(b) A 30 minute delay in shipping operations, is experienced annually due to long period wave action in the BSD. This delay is as a result of those ship movements which affect container handling operations. If the caisson extension should fail completely, the consequences would be minimal when compared with the delays due to wind experienced in the Port. There may be an increase in the duration of resonance. This figure is however difficult to obtain, and not critical due to the small influence thereof.

(c) If the caisson extension should fail completely, shipping might experience difficulty entering the BSD as a result of the increased wave height at the BSD entrance.



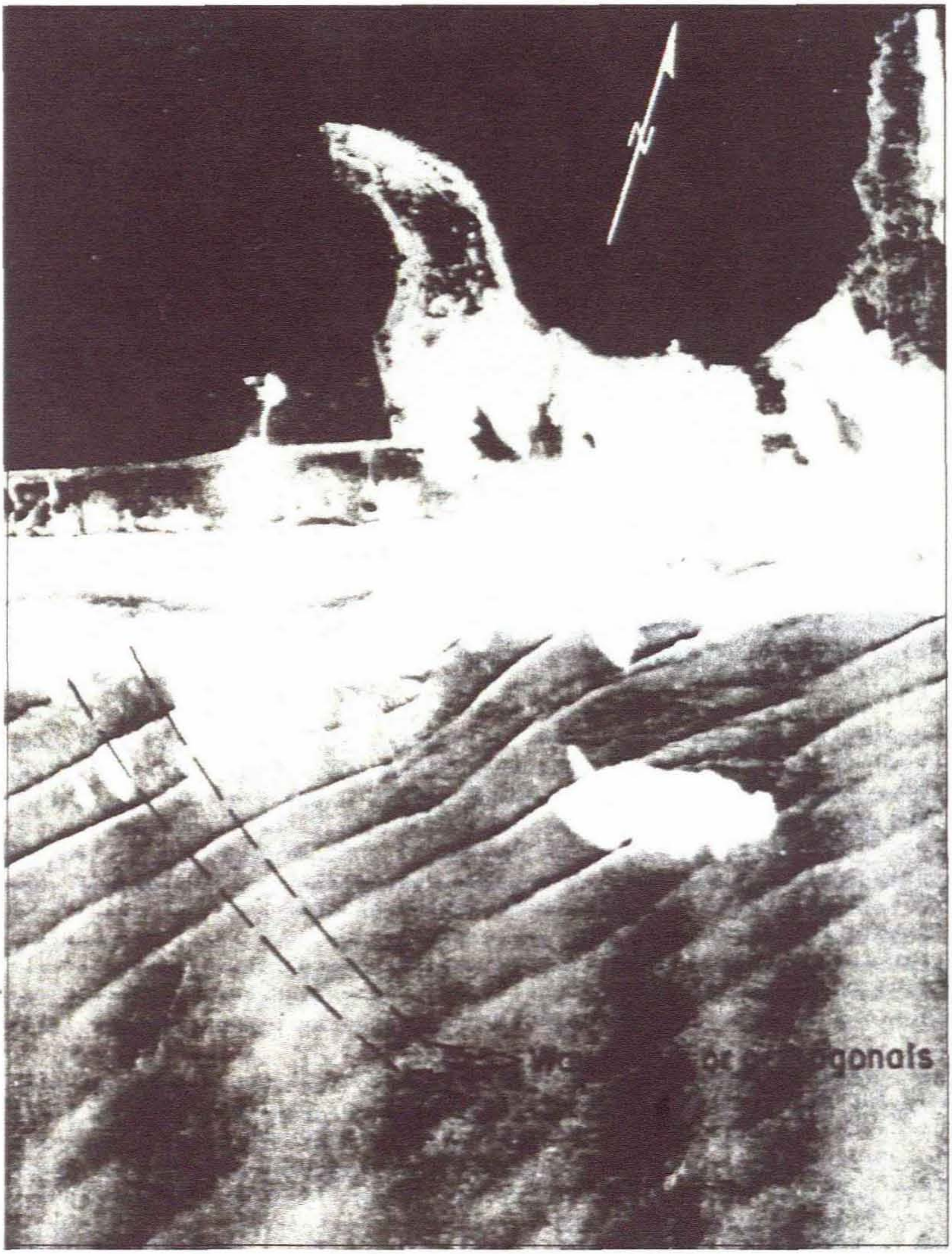
A container crane busy offloading containers

The crane, reaching for a container

Photograph group - E

Cape Town - 1994

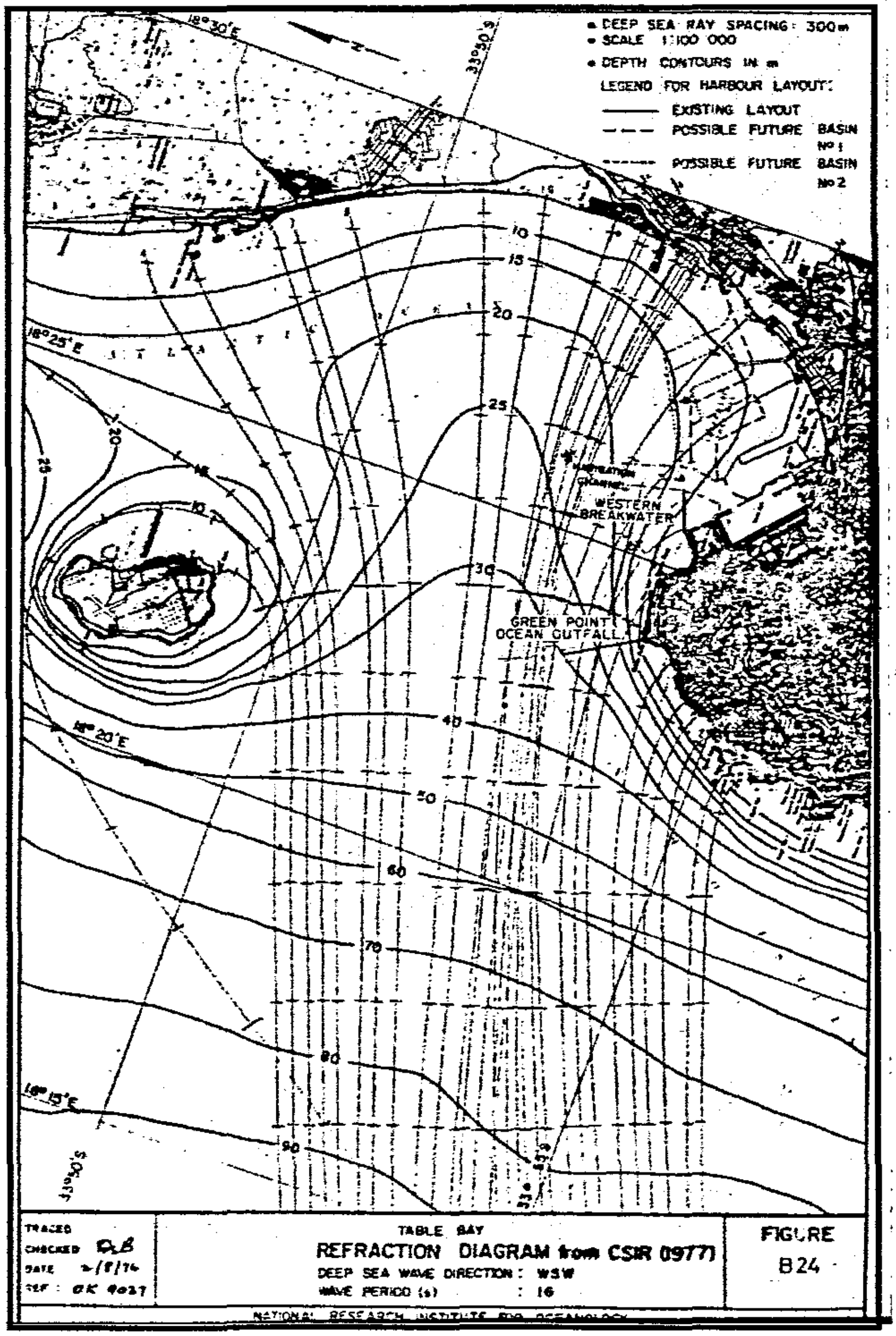
Fig 4.1



Wave refraction at Westhampton
Beach, Long Island, New York
(Shore Protection Manuel)

Wave Refraction

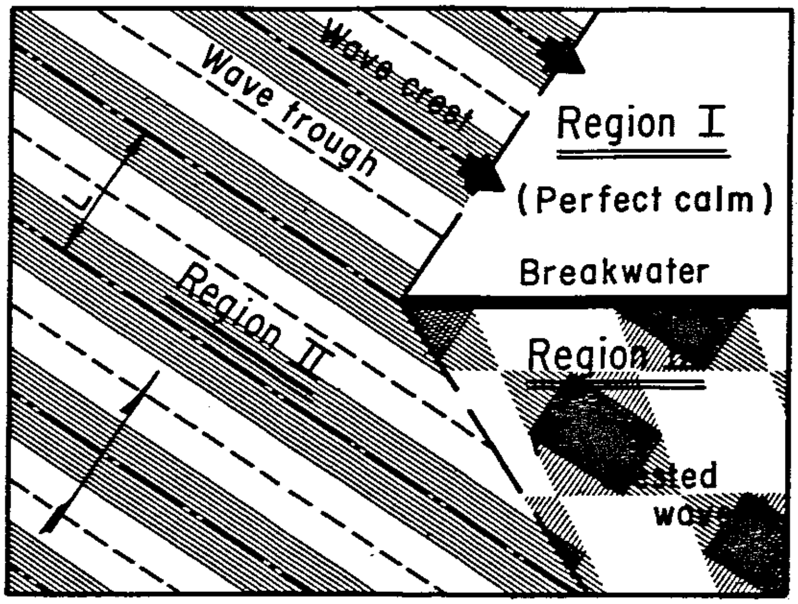
Fig 4.2



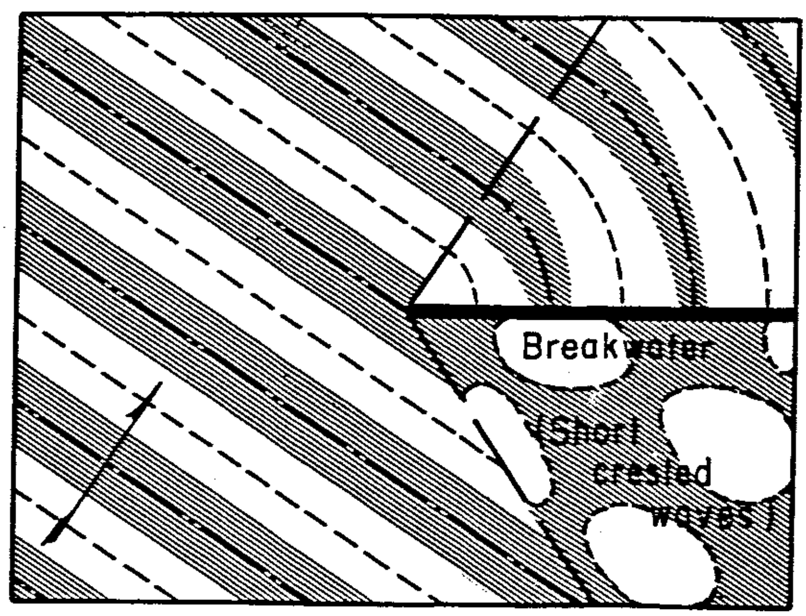
REFRACTION DIAGRAM from CSIR (1977)
Deep sea wave direction: WSW
Wave period (s) : 16

Wave Refraction Diagram

Fig 4.3



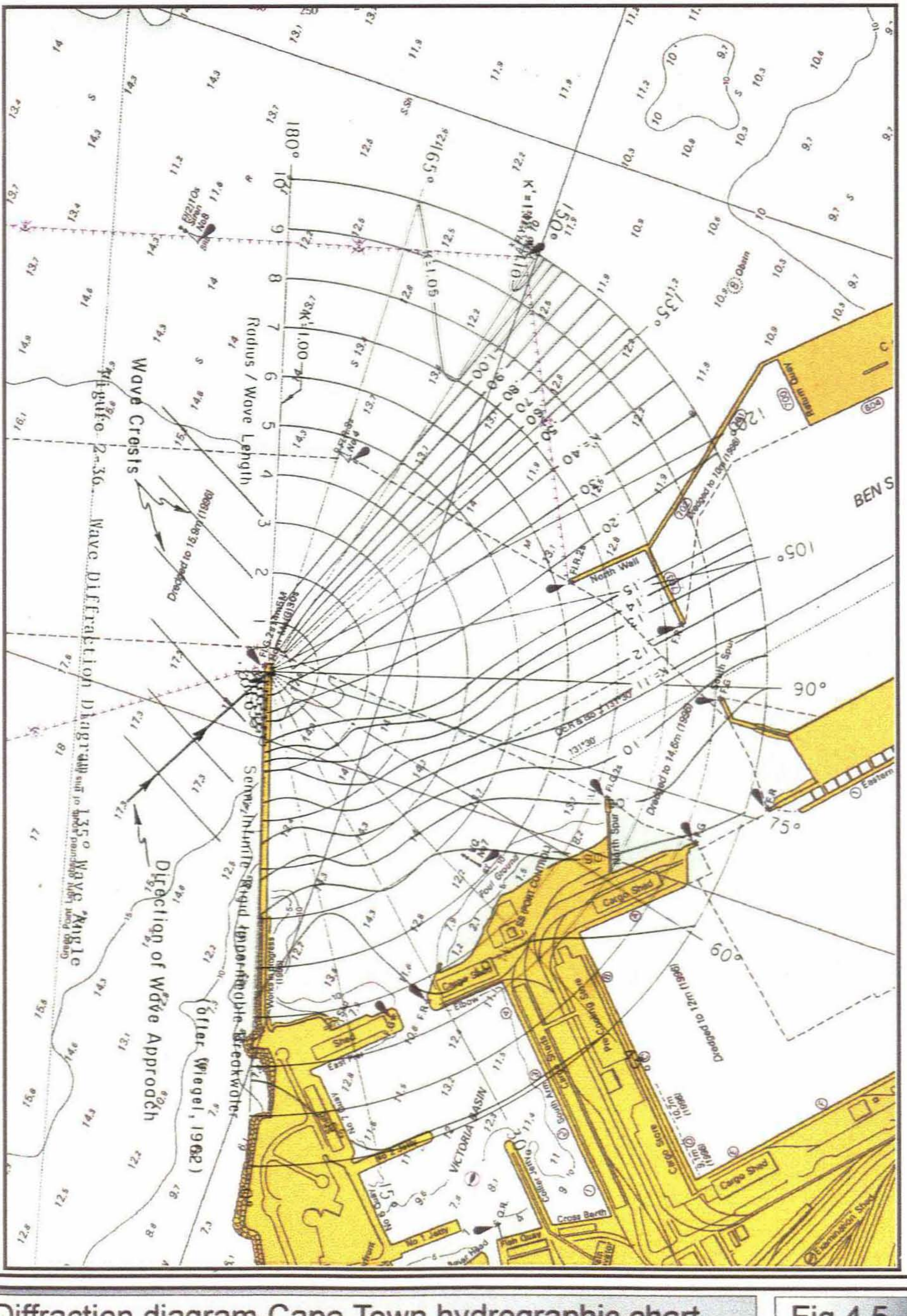
Wave Incident on a breakwater - No Diffraction



Wave Incident on a Breakwater - Diffraction Effects
(Shore Protection Manuel)

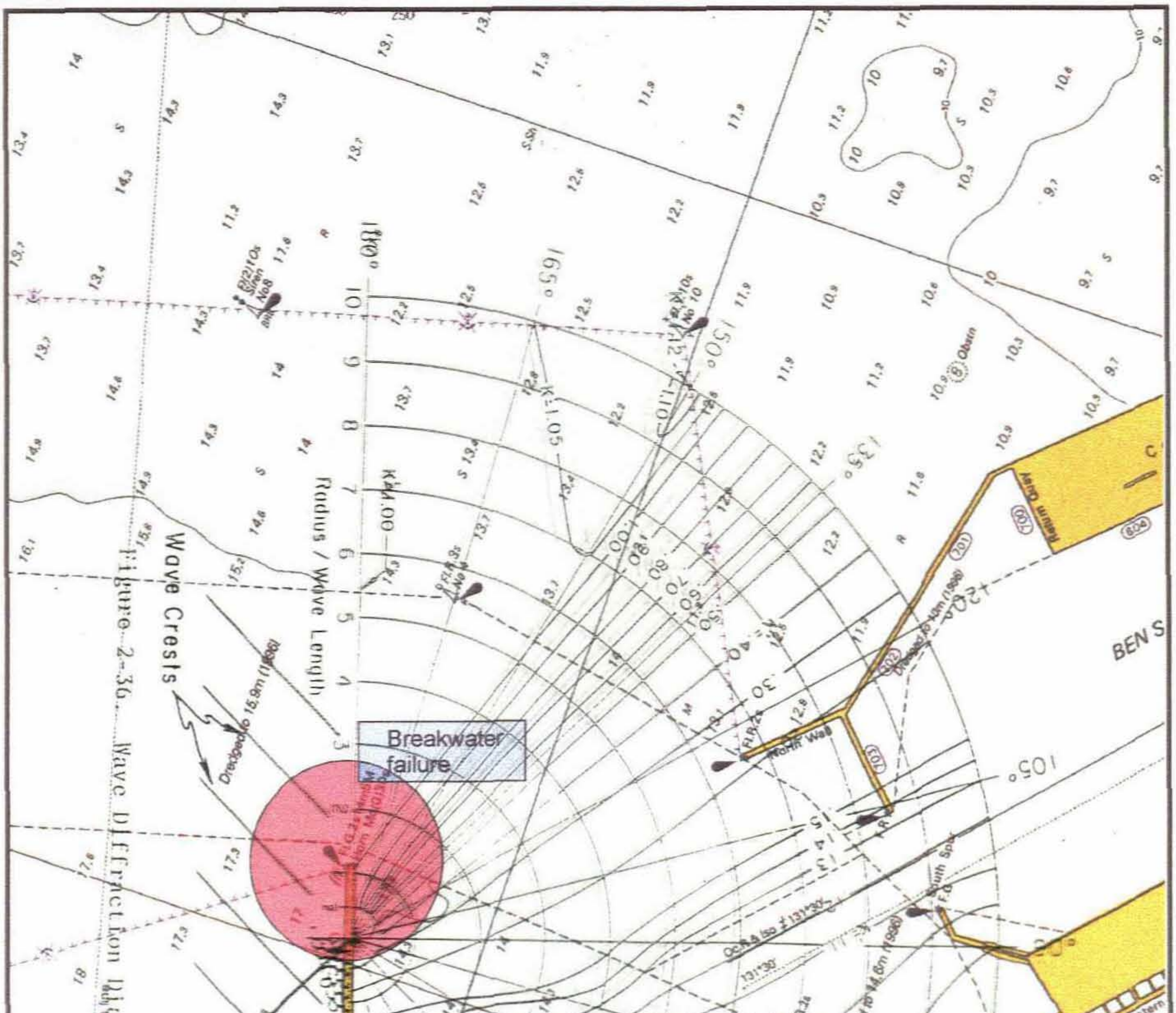
Wave Diffraction

Fig 4.4



Diffraction diagram-Cape Town hydrographic chart

Fig 4.5



Diffraction diagram-Cape Town hydrographic chart

Fig 4.6

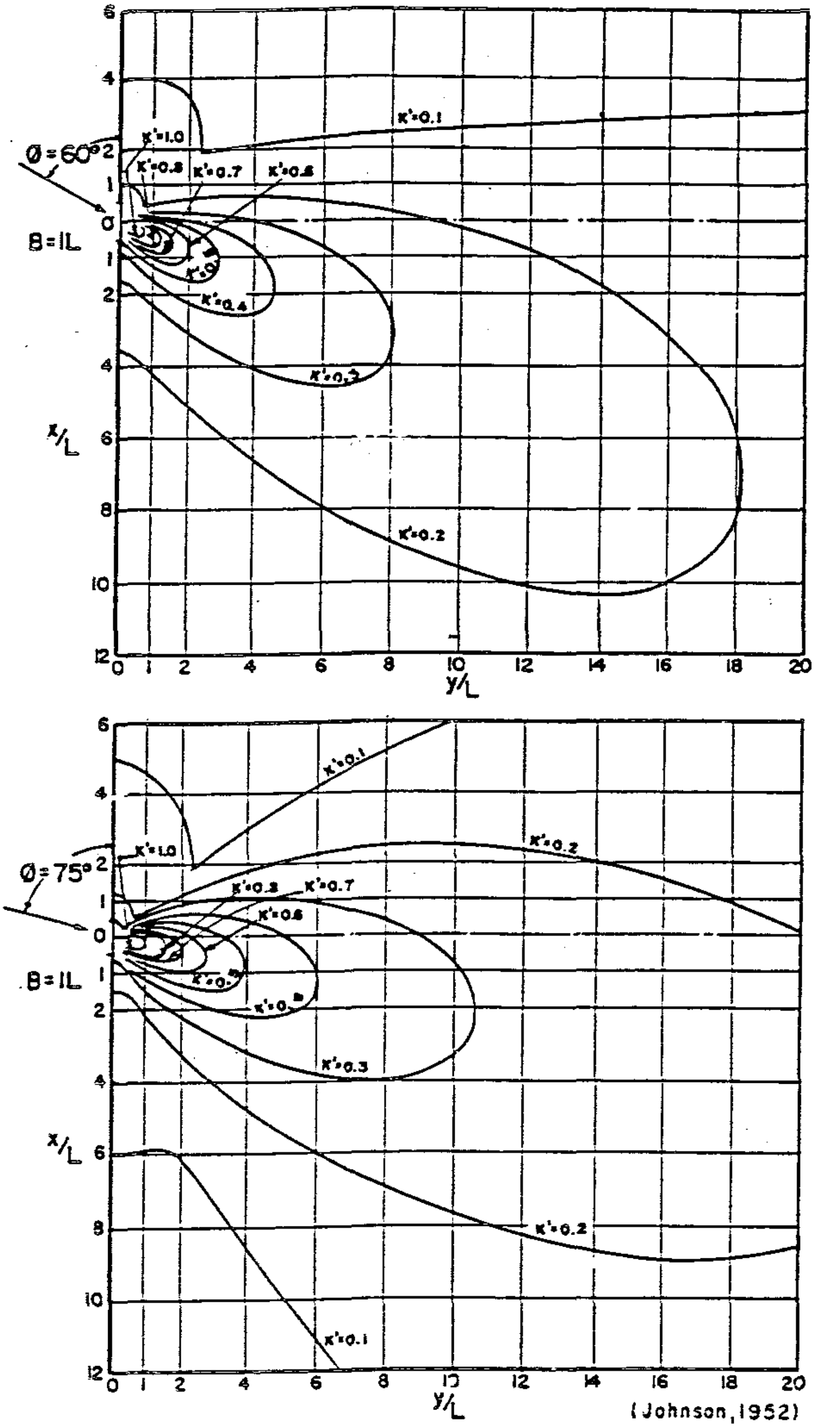


Figure 2-57. Diffraction for a Breakwater Gap of One Wavelength Width (ϕ = 60 and 75°)

Diffraction diagrams used for BSD entrance

Fig 4.7

MONTH	TOTAL HOUR DELAYED	NUMBER VESSELS DELAYED	NORM
Mar-94	893.84	79	11.31
Apr-94	742.44	81	9.17
May-94	1232.38	87	14.17
Jun-94	740.21	86	8.61
Jul-94	777.25	86	9.04
Aug-94	509.75	98	5.20
Sep-94	323.69	85	3.81
Oct-94	379.49	86	4.41
Nov-94	585.04	91	6.43
Dec-94	577.05	77	7.49
Jan-95	1171.59	81	14.46
Feb-95	1145.8	80	14.32
Mar-95	1762.82	69	25.55
Apr-95	2697.46	57	47.32
May-95	2760.91	53	52.09
Jun-95	720.88	79	9.13
Jul-95	674.27	82	8.87
Aug-95	885.56	78	11.35
Sep-95	388.93	88	7.86
Oct-95	591.26	88	7.25
Nov-95	1751.30	84	9.56
Dec-95	1812.77	71	11.56
Jan-96	2339.66	76	13.84
Feb-96	1879.03	87	13.84
Mar-96	1116.68	94	14.44
Apr-96	838.55	94	13.93
May-96	871.67	96	13.54
Jun-96	1667.57	93	13.91
Jul-96	461.47	90	13.24
Aug-96	642.72	90	12.81
Sep-96	255.56	85	12.2
Oct-96	362.4	94	11.66
Nov-96			
Dec-96			

Delays to the Container operations - Hours

Fig 4.8

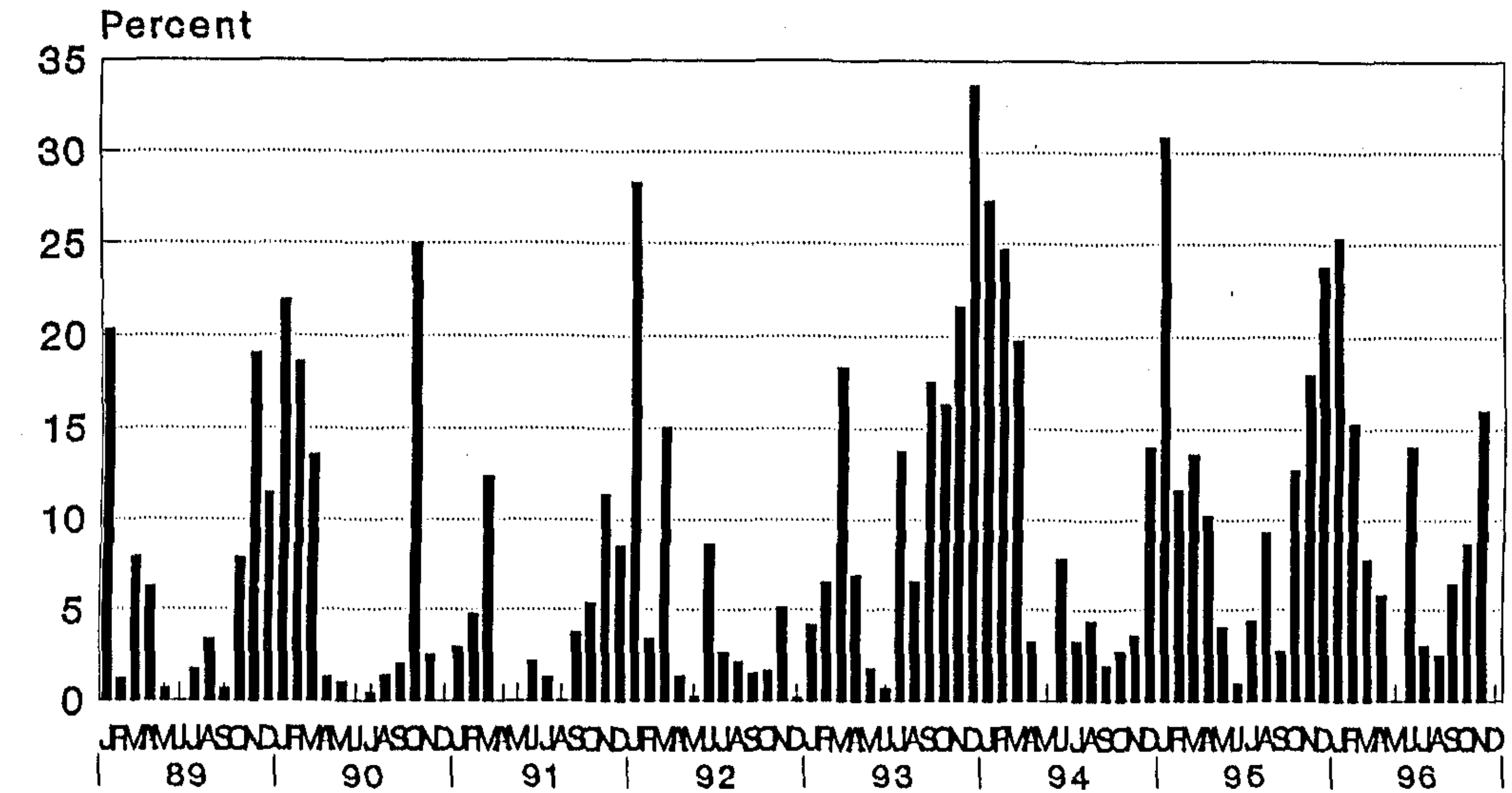
Container Terminal - Weather Delays

Fig 4.9

CONTAINER TERMINAL CAPE TOWN

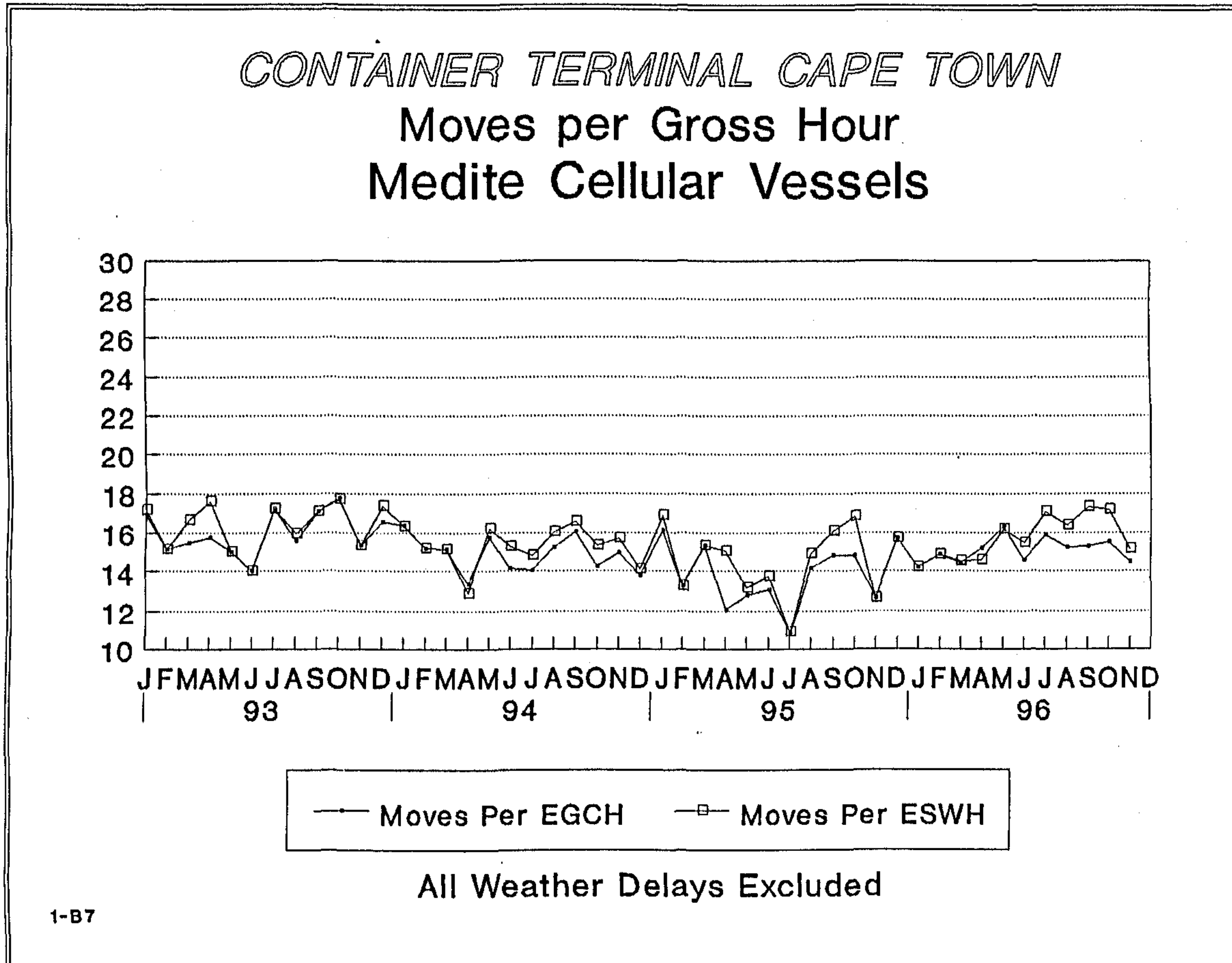
Weather Delays

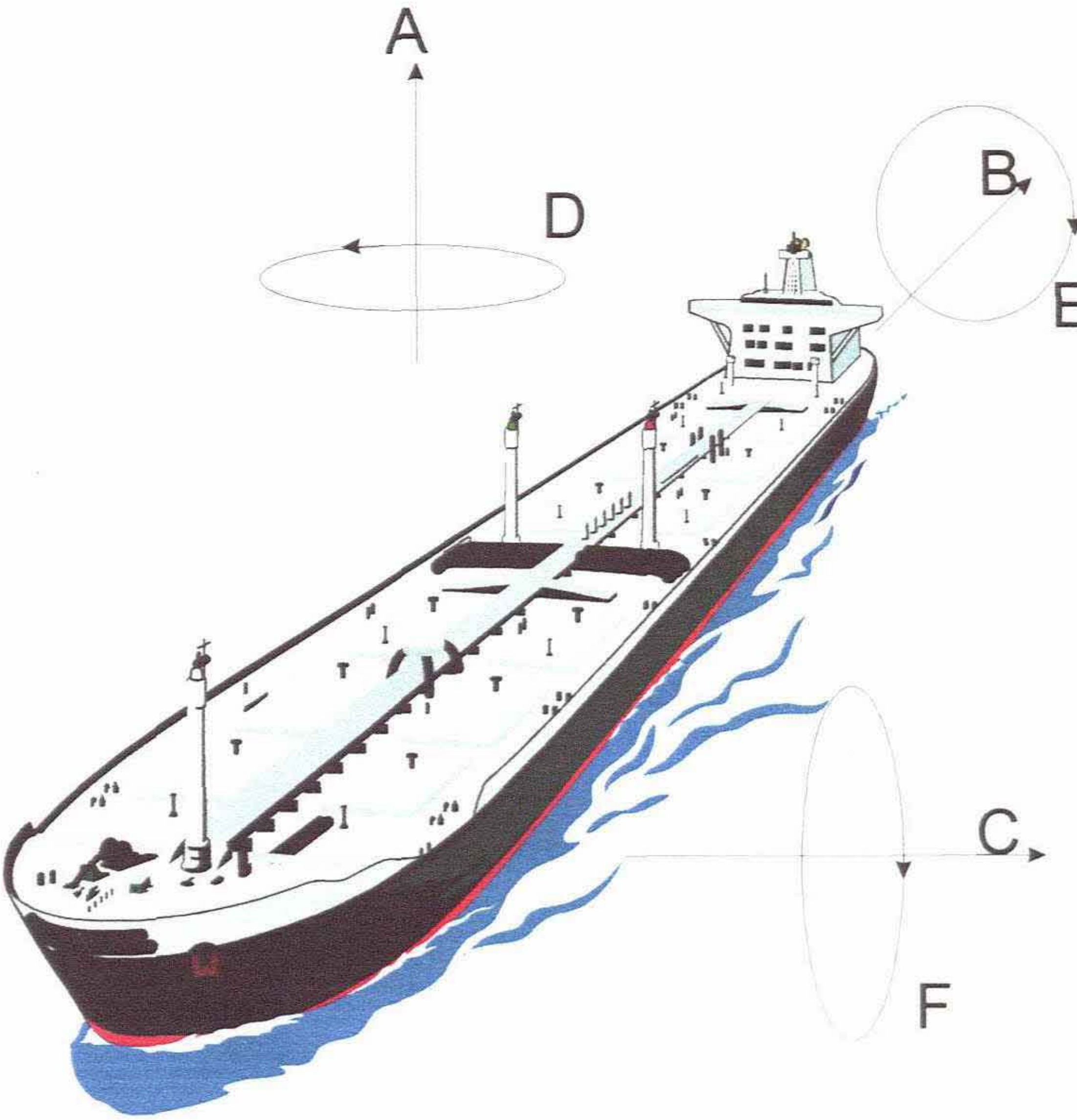
All Vessels



Percent Weather Delays/Gross Crane Hour

1-B8


CONTAINER TERMINAL CAPE TOWN
Moves per Gross Hour
Medite Cellular Vessels
30
28
26
24
22
20
18
16
14
12
10
J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
93
94
95
96
Moves Per EGCH
Moves Per ESWH
All Weather Delays Excluded
1-B7



A-Heave
B-Surge
C-Sway
D-Yaw
E-Roll
F-Pitch

Ship Motions

Fig 4.11

5. <u>REPAIRS TO THE STRUCTURE</u>

5.A ESTABLISHMENT OF MONITORING PROGRAMME

5.A.1 Monitoring and maintenance

It is commonly assumed that breakwater structures require little or no maintenance for the majority of their design life. This assumption is based on the idea that since large amounts of money have been invested in the structure, little or no maintenance is required for the larger part of its design life.

Monitoring and maintenance of a breakwater structure is very important to maximise its useful life. Due to the uncertainties involved in the design process it is even more important to keep the breakwater structure in good repair. Such monitoring and maintenance might at some stage be the critical factor determining whether or not the structure will withstand adverse weather conditions. The majority of breakwater designs are based on the prediction of possible future wave conditions which may occur at a certain time period in the future (design lifetime). The majority of structure failures have been due to the inability of the engineer to accurately predict future conditions. It is therefore very important to monitor structures, to be aware of their condition and to carry out repairs timeously as to avoid damage failure.

5.A.2 Caisson breakwater monitoring program

A lack of information exists between the time the caisson extension was constructed and the 1985 findings. There is no information available on diving inspections, level surveys or any information that would help to establish the condition of the caisson section during that time period. This creates the situation where predictions regarding the condition of the caisson extension will have to be made using information gathered from people and their observations 10 years ago. This is not a desirable situation and could affect accurate predictions concerning future maintenance of the structure.

5.B EFFECTS OF REPAIRS ON ENVIRONMENT

5.B.1 Effect on environment - general repairs

The general repairs will have little or no effect on the marine life present on the structure. The materials used will be cement based. The remaining repairs above water level will have no effect on the environment.

5.B.2 Effect on environment - Caisson failure reconstruction

In the event of caisson failure, the marine life on the structure will unfortunately be affected. The surrounding area of the seabed consists largely of a sand bottom with little significant sea life that can be disturbed. The caisson structure is a breeding ground for crayfish and small marine organisms. If the structure should fail or be replaced, the local surrounding marine life will be disturbed.

5.C TOURIST POTENTIAL

5.C.1 Tourist

The Victoria & Alfred Waterfront (V&AW) have various proposals for the area surrounding the breakwater.(See fig 6.1)

5.C.1.1 Planned ferry terminal

There were V&AW proposals to utilise the inside of the breakwater for yacht moorings. This proposal has however being replaced with the proposed ferry terminal.(see fig 6.2 & 6.3)

This area will be utilised for the Robben Island ferries, Heliport and a cultural centre.

5.C.1.2 The breakwater as a tourist attraction

The breakwater is the property of Portnet and the public is discouraged from using it for safety reasons. The hand railings have disappeared over the years, partly due to vandalism but mainly due to corrosion and adverse weather conditions. The breakwater is also a dangerous area when high swell conditions are experienced and overtopping occurs.

With the V&A Waterfront developing the inside of the Breakwater, many new opportunities will arise. Piers and jetties are always an attraction to people, especially the breakwater since it forms part of the Port and allows for a magnificent vantage point for viewing any marine activity. The view of the mountain from the tip of the breakwater makes it a wonderful vantage point for photographers and tourists.

Very little activity is present at night time due to safety factors. (no lights and no handrailings). The daytime and night-time views of the city and Table mountain are impressive.

The breakwater has the potential to be a tourist attraction. Can we afford not to investigate the viability of such a project?

5.D CONCLUSION

(a) Monitoring of the caisson section of the breakwater is important. Previous monitoring of the extension has not been adequate. This has left large gaps in the monitoring information since the construction of the structure.

Comprehensive condition surveys should be conducted after each winter and all damages to the caisson structure repaired.

(b) If minor caisson repairs are required it will have little effect on the environment.

6. RECOMMENDATIONS

6.A PRESENT REVIEW

6.A.1 Review

The breakwater has largely been utilised by sport fishermen, local workers in the port, and on a small-scale by the general public. The majority of people who visit the port are not even aware of the structure and that they are allowed entry to it. Warning signs discourage people from utilising the structure for recreational activities.

6.A.2 Present condition-general public

The structure cannot be utilised for safe recreational purposes. Hand railings are not present and there is no safety equipment available. The vertical sides of the breakwater make it impossible for anyone to climb, in the event of their falling into the water. There are no life buoys or rope ladders present to assist a person who has fallen into the water. Apart from the light at the end of the breakwater no electric lighting does exists on the breakwater.

6.B ENHANCING THE RESOURCE

6.B.1 Resource

To utilise the Breakwater as a safe place for tourism the following factors will need to be addressed.

(a) Structure safety.
(b) Weather conditions.
(c) Awareness of the structure.

(d) Enhancement of the structure.

(e) Property rights.

6.B.1.1 Structure safety

There are very few hand railings remaining on the Breakwater, due to theft and weather damage. With the development of the Ferry Terminal improved security will be present to the breakwater and the theft of handrailings (sold for scrap value) will be prevented.

The most costly exercise will be to provide new handrailings on the breakwater for public safety. Provision will have to be made handrailings strong enough to resist the wave forces encountered during the winter months.

6.B.1.2 Weather safety

Measures will have to be taken to warn people when it is unsafe to enter the Breakwater depending on wave conditions. (See photograph group D).

It is suggested that a light system with a notice informing people when it will be safe or unsafe to use the breakwater (See fig 6.4).

6.B.1.3 Awareness of the structure

The V&AW will have to make people aware of the facility available and tourism can promote it as a place to visit.

6.B.1.4 Enhancement of the structure

There are five items that would enhance the structure:
(See fig 6.4)

(a) Railings:

Strong and corrosion resistant handrailings need to be provided at places where people would be in danger of falling into the sea.

The railings should also be wave resistant to withstand wave forces.

(b) Lighting:

Good lighting will need to be provided for access to the structure at night. There is electricity available on the caisson extension

(c) Lookout:

A lookout can be provided to give a better vantage point for the individuals or for sporting and other events.

(d) Safety equipment:

Safety equipment needs to be provided in case of persons falling into the sea.

(e) Property rights:

The breakwater is an asset belonging to Portnet. It is suggested that Portnet retains the asset but allows the V&AW to develop it. This joint venture would be a tourist attraction and would benefit the V&AW as well as Portnet.

6.C CONCLUSION

To utilise the breakwater as a tourist facility will be expensive initially. However, the potential is there for it to be used in this regard.

Victoria & Alfred Waterfront - Future layout

Fig 6.1

V&AW proposed land use - Nov 1995

Fig 6.2

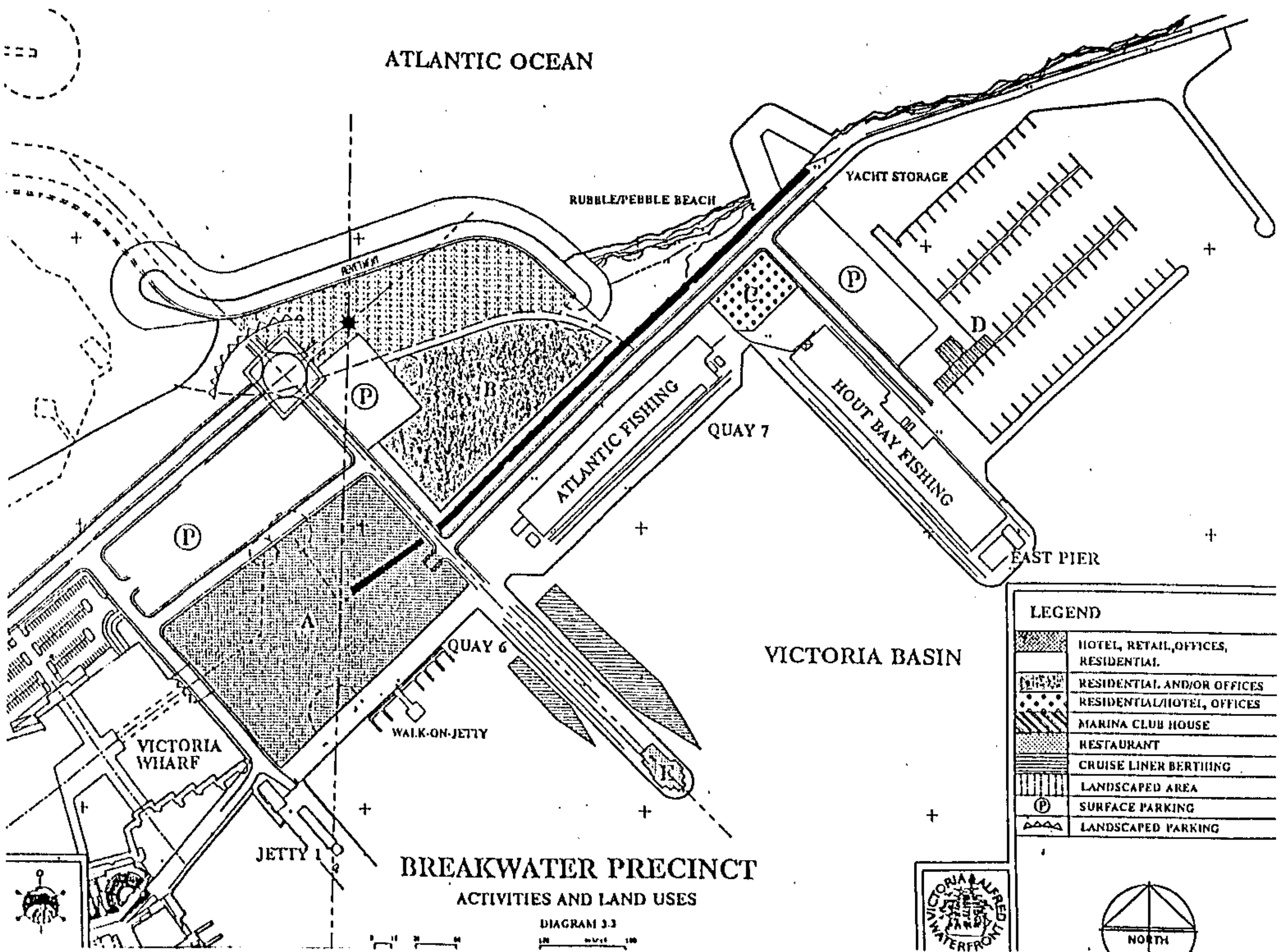
Design	GDR
Drawn	RCJ
Date	NOV95
Scale	

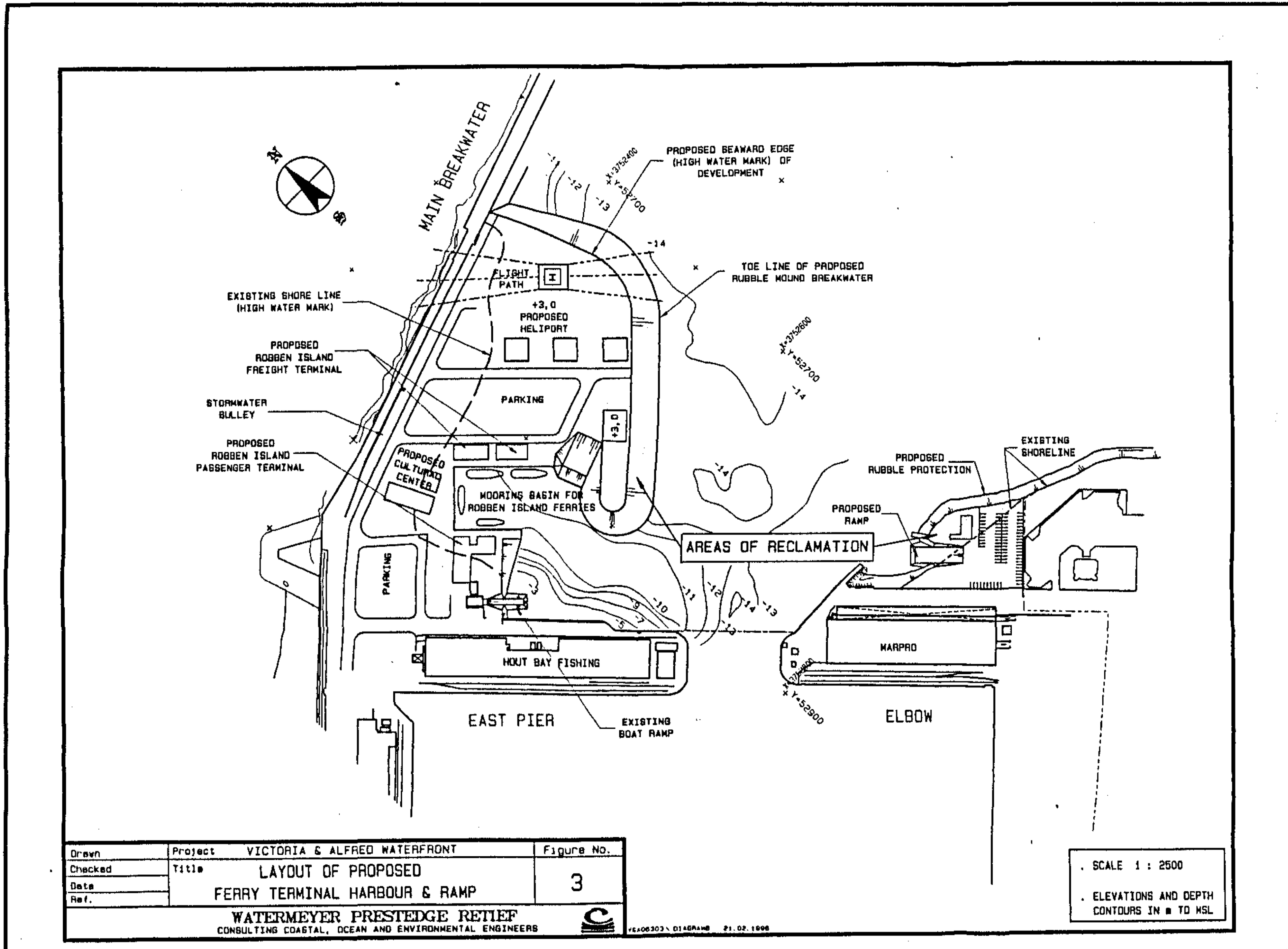
Project: V&A WATERFRONT

Title: BREAKWATER PRECINCT PROPOSED LAND USE

Figure No. 2

WATERMEYER PRESTEDGE RETIEF
CONSULTING COASTAL OCEAN AND ENVIRONMENTAL ENGINEERS





Proposed layout of Ferry Terminal and Harbour ramp

Fig 6.3

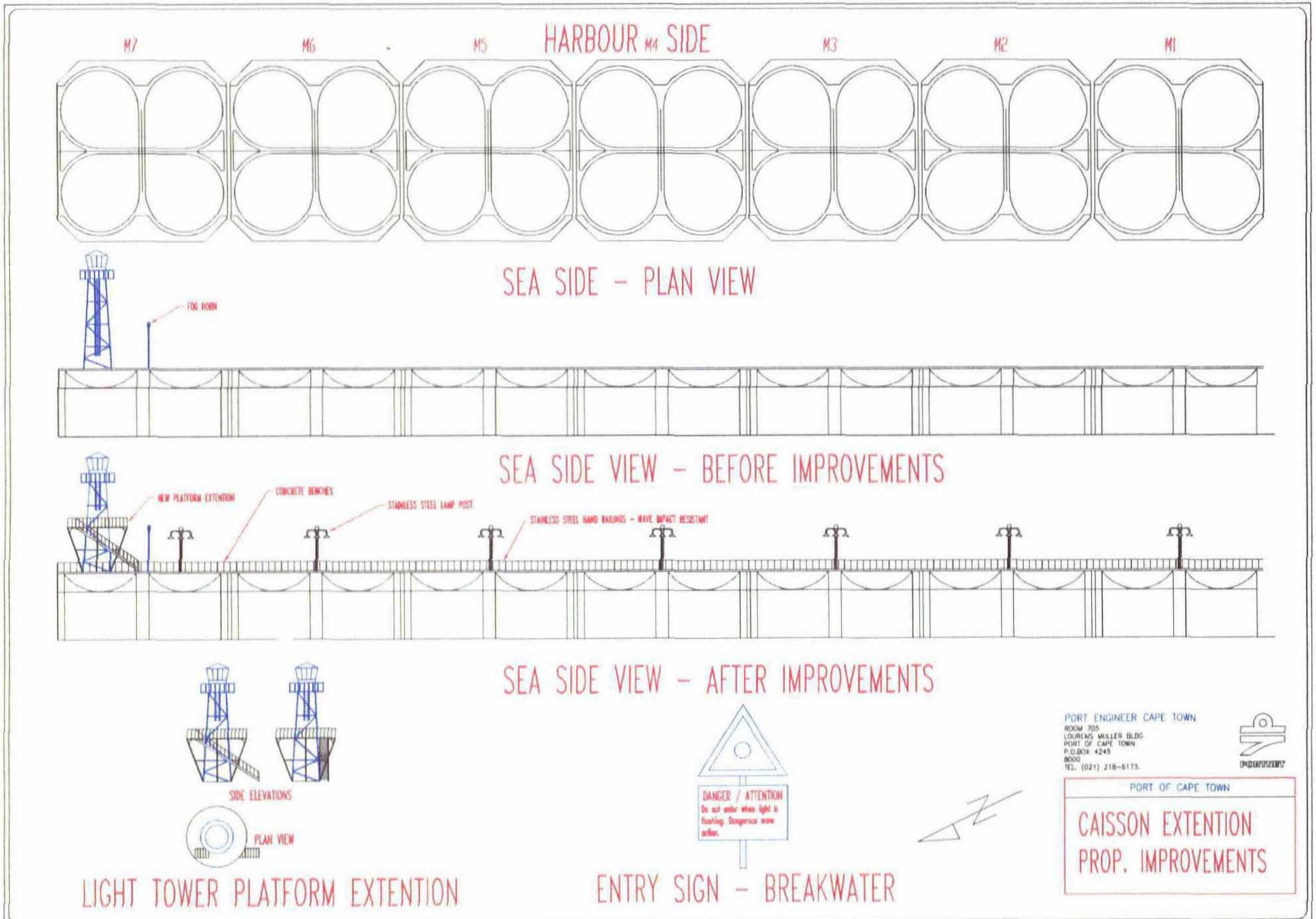

HARBOUR M4 SIDE
M7
M6
M5
M3
M2
M1
SEA SIDE - PLAN VIEW
FOG HORN
SEA SIDE VIEW - BEFORE IMPROVEMENTS
NEW PLATFORM EXTENTION
CONCRETE BENCHES
STAINLESS STEEL LAMP POST
STAINLESS STEEL HAND RAILINGS - WAVE IMPACT RESISTANT
SEA SIDE VIEW - AFTER IMPROVEMENTS
SIDE ELEVATIONS
PLAN VIEW
LIGHT TOWER PLATFORM EXTENTION
DANGER / ATTENTION
Do not enter when light is flashing. Dangerous wave action.
ENTRY SIGN - BREAKWATER
PORT ENGINEER CAPE TOWN
ROOM 705
LOURENS MULLER BLDG
PORT OF CAPE TOWN
P.O.BOX 4245
8000
TEL. (021) 218-6173.
PORT OF CAPE TOWN
CAISSON EXTENTION
PROP. IMPROVEMENTS

Fig 6.4

7. CONCLUSION

(a) There is settlement of the caissons extension to the Table Bay breakwater. The exact cause of settlement could be the result of various stages of foundation settlement. The exact cause of settlement at this stage remains unknown. It could be as a result of shear failure of the undisturbed shale layers or settlement of the stone layers.

Regular under-water inspections should be carried out to prevent any localized foundation damage due to grout sock failure. If localised foundation failure should proceed undetected, stability of the caisson extension will be affected. It is therefore very important to monitor the caisson extension for any damage.

(b) The major failure factors in caisson breakwater design are sliding and overturning. The Table Bay breakwater caisson extension has proven itself in wave conditions exceeding that for which it was designed.

A factor of greater concern is the condition of the grout socks. The deterioration of the grout socks can affect the condition of the stone foundation layers due to erosion. This in turn will reduce the sliding and overturning factors.

The condition of continuous settlement of the structure will place additional strain on the grout socks, adding to the risk of foundation erosion.

(c) The sediment movement along the Table Bay coastline is minimal. Localised sand movement is continuously taking place around the breakwater. This movement, however has little effect on the wave impact on the caisson breakwater section.

(d) In the event of complete caisson failure, the effects of diffraction, refraction and reflection will have little or no effects on container operations and are of minor concern when compared to the effects of wind delays on port operations.

This includes long period wave action which is responsible for resonance in the port.

Short period wave action will also have minor influence to port operations, even if the caisson extension has failed completely. During storm conditions the port is normally closed as a result of high wind conditions.

Long period wave effects will also be minimal, taking into account that the current delays to shipping due to ranging is 30 minutes per year. In the unfortunate event of caisson failure, the occurrence of resonance in the port will occur sooner, having a slightly longer effect on ship movements. This figure is minor compared to wind delays.

(e) In the event of complete caisson failure, shipping might experience difficulty entering the BSD. Ships require calmer waters to make the process of tug assistance possible.(attaching ropes to vessels entering the port) This factor will be an inconvenience and may cause delays to shipping.

(f) The financial implications to the port in the event of complete caisson failure, will largely be damages experienced on the structure itself. The environmental effects as a result of oil spills will be minor. Replacement of the caisson breakwater structure will be costly.

(g) Diving inspections of the caisson extension should be carried out annually after the winter months.

(h) With little trouble and some expense the breakwater can be utilised as a valuable asset to the city. The caisson breakwater extension was build to protect the BSD against wave conditions that might interfere with container operations.

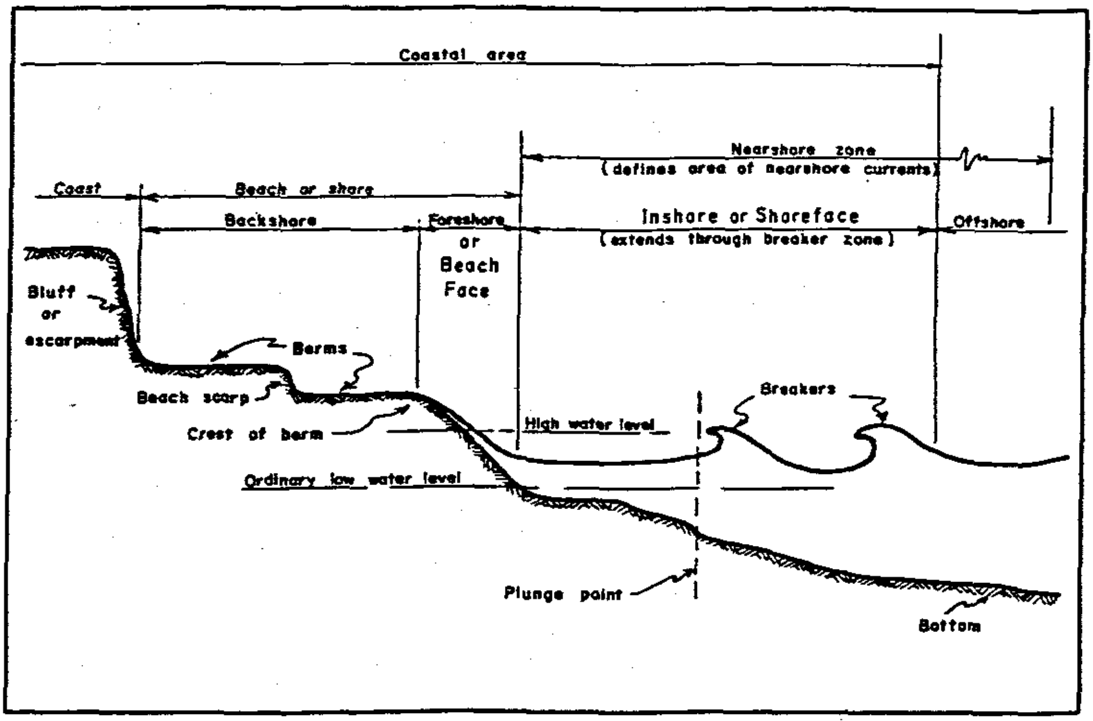
The question can be asked; "was the caisson extension really required considering the financial outlay of structure construction, compared to the possible risk of downtime to container operations due to wave effects?"

DEFINITIONS:

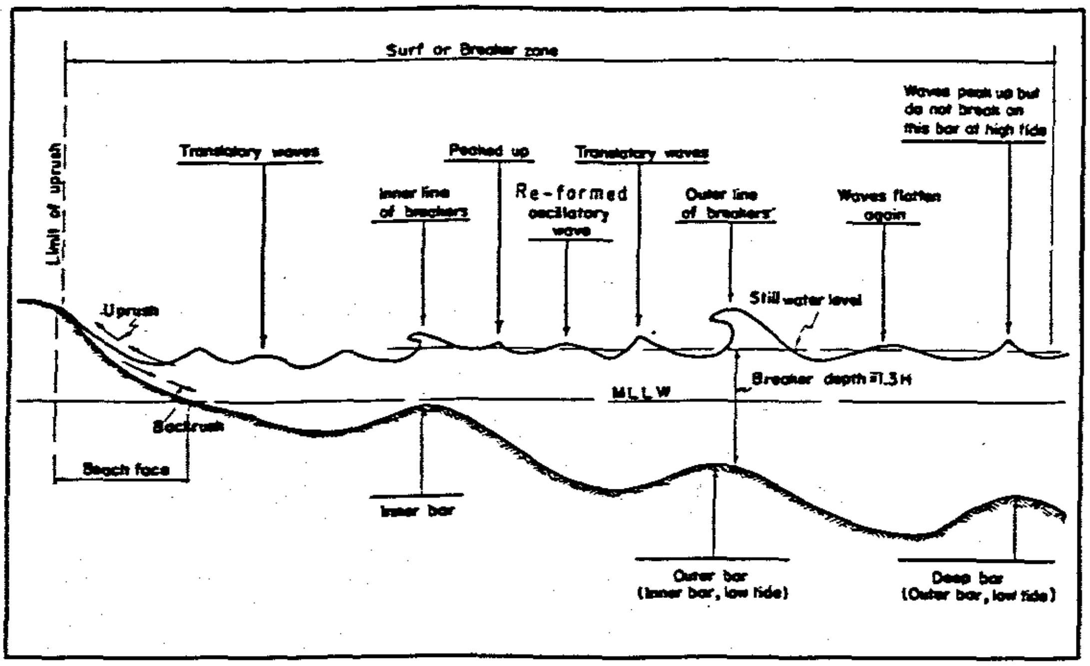
AMPLITUDE, WAVE:	The magnitude of the displacement of a wave from a mean value. An ocean wave has a amplitude equal to the vertical distance from still-water level to wave crest.
BEACH EROSION:	The carrying away of beach materials by wave action, tidal current, littoral current, or wind.
BREAKER:	A wave breaking on a shore, over a reef, etc. Breakers may be classified into four types.
Spilling:	Bubbles and turbulent water spill down front face of the wave. The upper 25 percent of the front face may become vertical before breaking. Breaking normally occur at quite a distance.
Plunging:	Crest curls over air pocket: breaking usually with a crash. Smooth splash-up usually follows.
Collapsing:	Breaking occurs over lower half of wave, with minimal air pocket and usually no splash-up. Bubbles and foam present.
Surging:	Wave peaks up, but bottom rushes forward from under the wave, and wave slides up beach face with little or no bubble production. Water surface remains almost plane except where ripples may be produced on the beachface during runback.
BREAKER DEPTH:	The still water depth at the point where a wave breaks.

CLAPOTIS:	Usually associated with the standing wave phenomenon caused by the reflection of a non breaking wave train from a structure with a face that is vertical or nearly vertical.
CURRENT, COASTAL:	One of the offshore current flowing generally parallel to the shoreline in the deeper water beyond the near surf zone; these are not related genetically to waves and resulting surf, but may be related to tides, winds, or distribution mass.
ECHO SOUNDER:	An electronic instrument used to determine the depth of water by measuring the time interval between emissions of a sonic or ultrasonic signal and the return of its echo from the bottom.
GROUND SWELL:	A long high ocean swell; also, this swell as it rises to prominent height in shallow water.
HARBOUR OSCILLATION (HARBOUR SURGING):	The nontidal vertical water movement in a harbour or bay. Usually the vertical motions are low; but when oscillations are exited by tsunami or storm surge, they may be quite large. Variable winds, air oscillations, or surf beat may also cause oscillations.
HYDROGRAPHY:	A configuration of an underwater surface including its relief, bottom materials, coastal structures, etc.
LENGTH OF WAVE:	The horizontal distance between similar points on two successive waves measured perpendicularly to the crest.

PROPAGATION OF WAVES:	The transmission of waves through water.
REFLECTED WAVE:	The part of an incident wave that is returned seaward when impinges on a steep beach, barrier, or other reflecting surface.
REFRACTION:	The process by which the direction of a wave moving in shallow water at an angle to the contours is changed: The part of the wave advancing in shallower water moves more slowly than the part still advancing in the deeper water, causing the wave crest to bend toward alignment with the underwater contours.
RESONANCE:	The phenomena of amplification of a free wave or oscillation of a system by a forced wave or oscillation of exactly equal period. The forced wave may arise from an impressed force upon the system or from a boundary condition.
SINUSOIDAL WAVE:	An oscillatory wave having the form of a sinusoid.
SWELL:	Wind generated waves that have travelled out of their generating area. Swell characteristics exhibits a more regular and longer period and has flatter crests than the waves within their fetch.
Hmo:	The significant wave height determined in the frequency domain as 4 Mo where Mo is the area under the spectrum curve S (f) between an upper and the lower cut-off frequency (m)

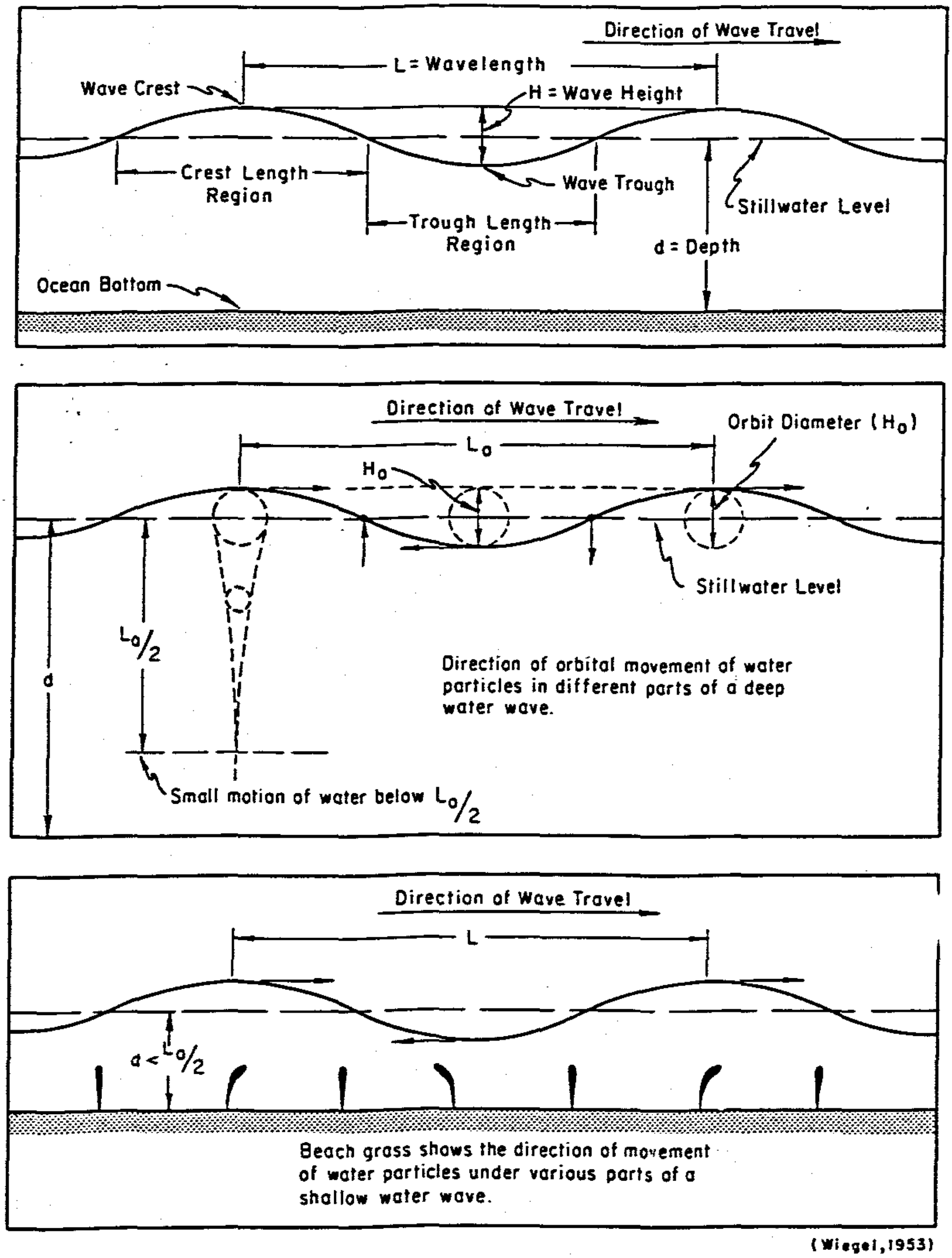


Beach Profile-Related Terms.



Schematic Diagram of Waves in the Breaker Zone.

(Shore Protection Manual)



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Annexure A

Report on the condition of the caisson joints of the Main Breakwater 1988:

Mr S Bently

CONTENTS

S.A. HARBOURS : TABLE BAY HARBOUR

REPORT ON THE CONDITION OF THE CAISSON JOINTS OF THE MAIN BREAKWATER (CHO TO CH131M) TABLE BAY HARBOUR

(a) TERMS OF REFERENCE

Further to the findings of the report "Conditions of the seaward face of the Main Breakwater" of September 1988, the acting Harbour Engineer (Civil) requested a detailed underwater investigation of the caisson joints.

(b) INTRODUCTION

Theoretical stability calculations, on this breakwater section, were carried out by the C.S.I.R. (DEMAST) at Stellenbosch. Based on these results it became necessary to investigate the caisson joints of the main breakwater.

(c) THE BREAKWATER (CHO TO CH131M)

This section of the wall consists of sand-filled caissons with no armour protection.

Each caisson is nominally 19,8m x 18,5m x 18,4m deep and divided into four circular chamber with a diameter of 8,5m each.

Each caisson is founded on a 20-75mm stone layer overlaying the 1-450 kg stone in fill of a dredged trench.

The caissons are linked by "grout socks" in 100mm recesses.

The gap tolerance between caissons was set at 75mm. The best that was achieved was gaps from 75mm to 400mm with an average of 200mm for the 7 caissons.

(d) <u>SURVEY METHOD AND RECORDING</u>

Diving was from the launch "Troupant" using airlines. Recording was done using an underwater slate and 30m tape measure secured from the breakwater deck, at each joint.

Joints are numbered to correspond with the information on drawing TBH.106.W2-1008 Sheet 1,.

The joints were inspected on the seaward side and the landward side.(See drawing 1)

(e) <u>DIVER SURVEY AND INSPECTIONS</u> (Seaward Side)

The initial inspection was reported in the report "Condition of the seaward face of the main breakwater" of September 1988.

During October 1988 the divers attempted to clear the marine growth from the joints to expose the entire grout socks for inspection. At joint M1-M2 on the seaward side a section 2,9m long was cleaned of marine growth in 2 hours of diving. The divers reported that the marine growth was extremely difficult to remove and had adhered to the entire grout sock recess and had "cemented" the entire recess closed.

To determine the strength of the marine growth, the divers tried to push a ø25mm reinforcing rod through the growth, without any success. The divers were instructed not to remove anymore marine growth as it appeared to have very good adhesion properties and was serving a useful function of blocking the recess where the grout sock was missing and protecting the grout sock where it was in place.

Damage report drawing 1

1. Grout sock in position
2. Area covered by marine growth
3. Grout sock missing

The divers also reported large water pressures at the positions where there was no grout sock. When a wave broke against the structure at the opposite side the water pressure 'jet force' at a 300mm hole was described as similar to being blasted by a fire hydrant hose. (700 kPa). - Refer joint M4/M5 on seaward side.

The marine growth extended in a wide band over a width of 7,5 metres from the water line.

(f) DIVER SURVEY AND INSPECTIONS (Harbour Side)

The grout socks are in a reasonably sound condition. At joint M1/M2 the rock is displayed in two places. Small sections of sock are missing at joints M3/M4 and M4/M5.

At joint M6/M7 the sock is missing at a position at the seabed. A section of the sock is missing over 1 metre and has resulted in the scour of the rock foundation underlayer to a depth of 400mm. This scour extends for a distance of 280mm from the structure. Visible movement of small particles of rock and sand was observed by the divers.

The marine growth extends in a wide band 6,5 metres wide from the water line.

(g) REPORT SUMMARY

From the divers observations it can be deducted that the failure of the grout sock is in the joint where the caisson gaps are wider than the designed required gap. The sock originally did not "expand" to completely fill the recesses allowed. The sock also shows, in places, as if the tremie concrete was not poured evenly and created pockets or "blobs" of concrete within the sock.

Seabed scour has occurred at joint M6/M7, as well as the loss of large sections of the grout sock.

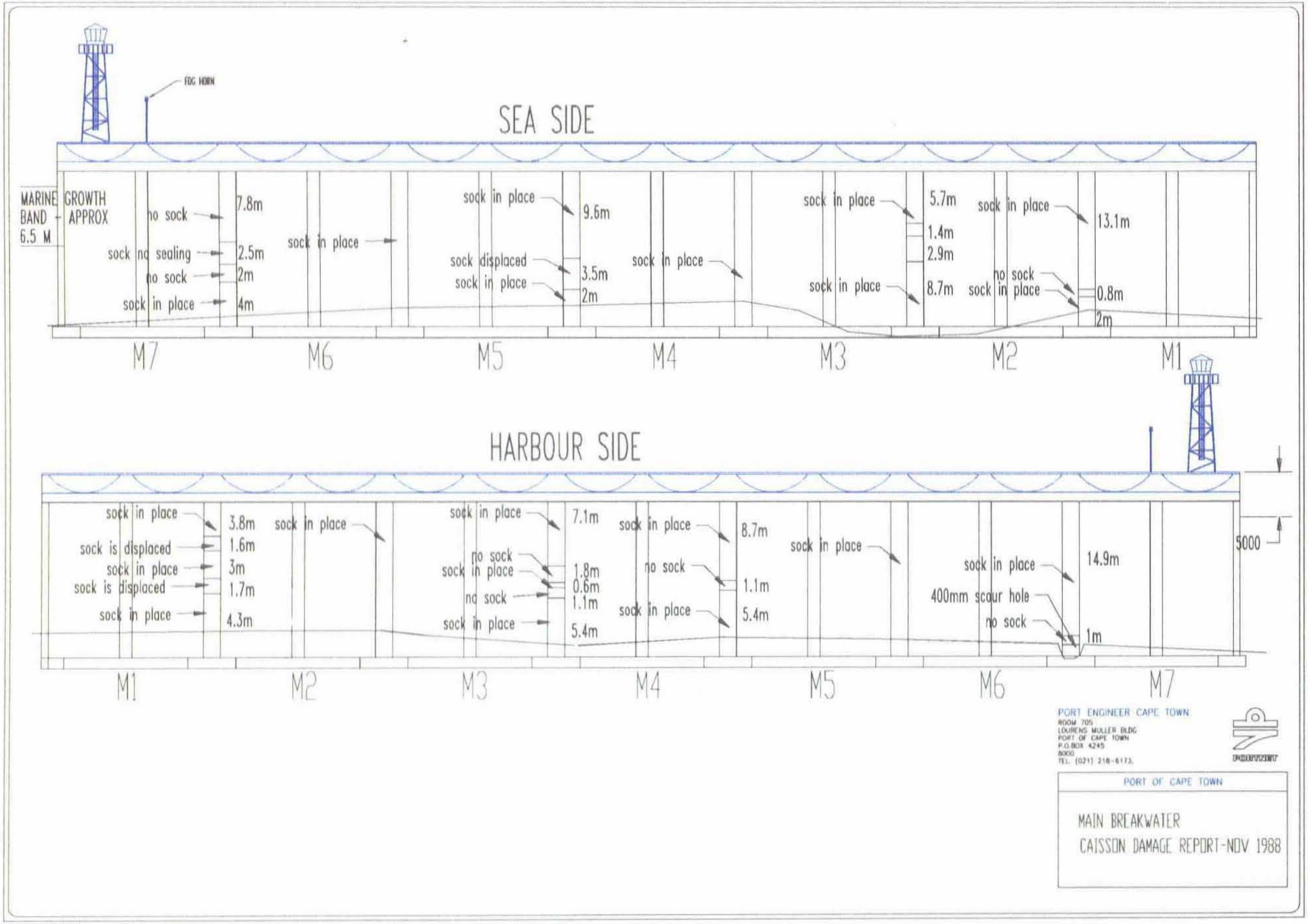
Based on the assumption that the severest wave attack occurs from the seaward side the order of repairs would be: (See Dwg 1)

1. Joints M6/M7
2. Joints M2/M3
3. Joints M4/M5
4. Joints M1/M2
5. Joints M3/M4

Joint M5/M6 - no repairs

Annexure A - Damage Report

DWG 1



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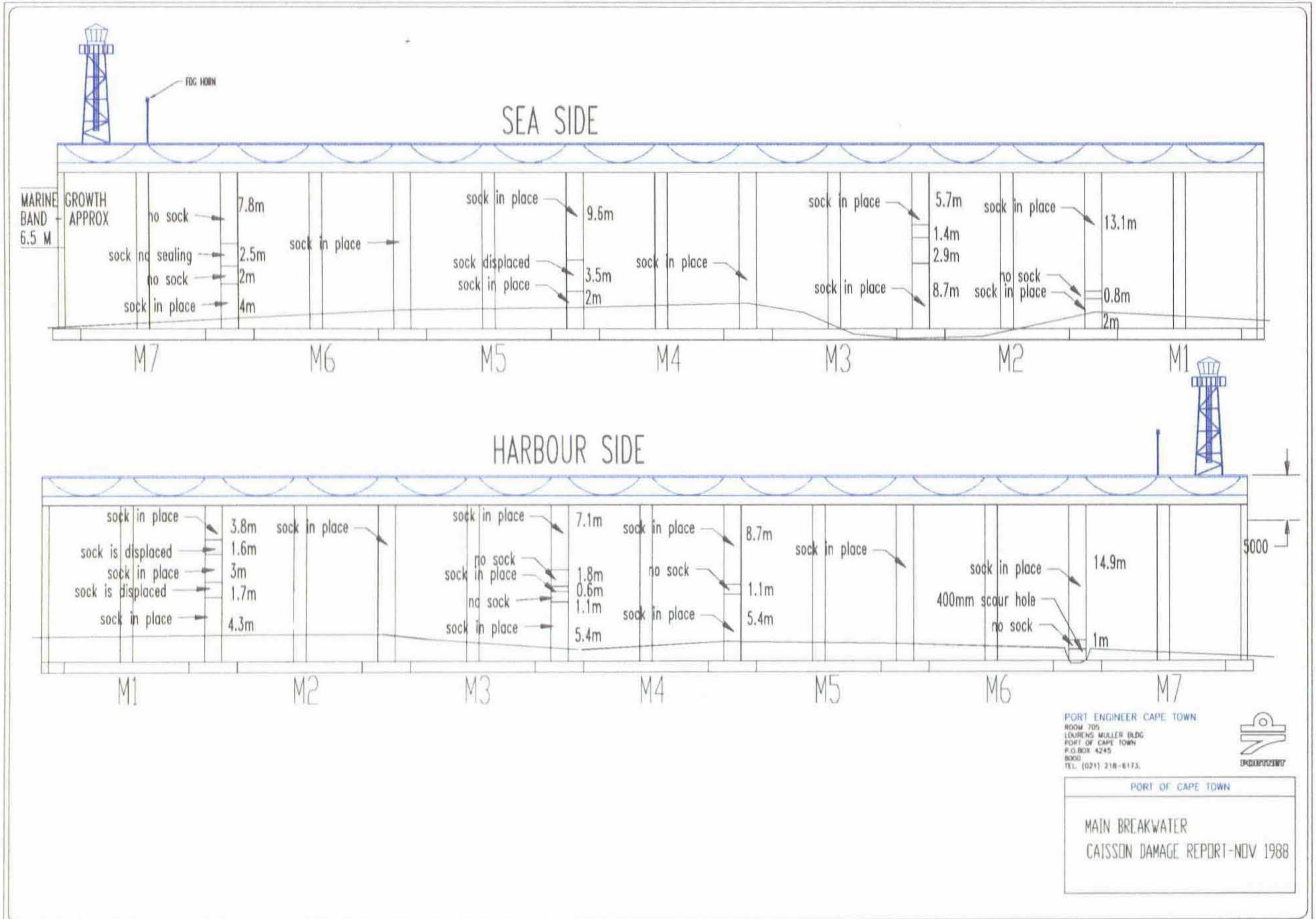
1. Joints M6/M7
2. Joints M2/M3
3. Joints M4/M5
4. Joints M1/M2
5. Joints M3/M4

Joint M5/M6 - no repairs

Annexure A - Damage Report



DWG 1

Annexure B

Report on the condition of the caisson joints of the Main Breakwater 1997:

Mr S Bently

MAIN BREAKWATER WALL INSPECTION, SURVEY

Joint M5 - M4: Grout sock in joint loose and moves inside the groove with the current.

Joint M7 - M6: No scour visible in foundation opening as this opening had been filled with ballast stone and 300mm stone. The stone fill is still in positions as placed during repairs and has sealed off the undermined area completely.

Joints M6 - M5, **M4 - M3,** *M3 - M2* and **M2 - M1** : Above joints were inspected and surveyed. No noticeable changes to the grout socks had been observed by the divers.

MAIN BREAK WALL SURVEY - SEASIDE

Joint M7 - M6: The scour opening at this joint had been filled and sealed with ballast stone and 300mm stone to approximately 1m above the foundation footing and is in a good condition.

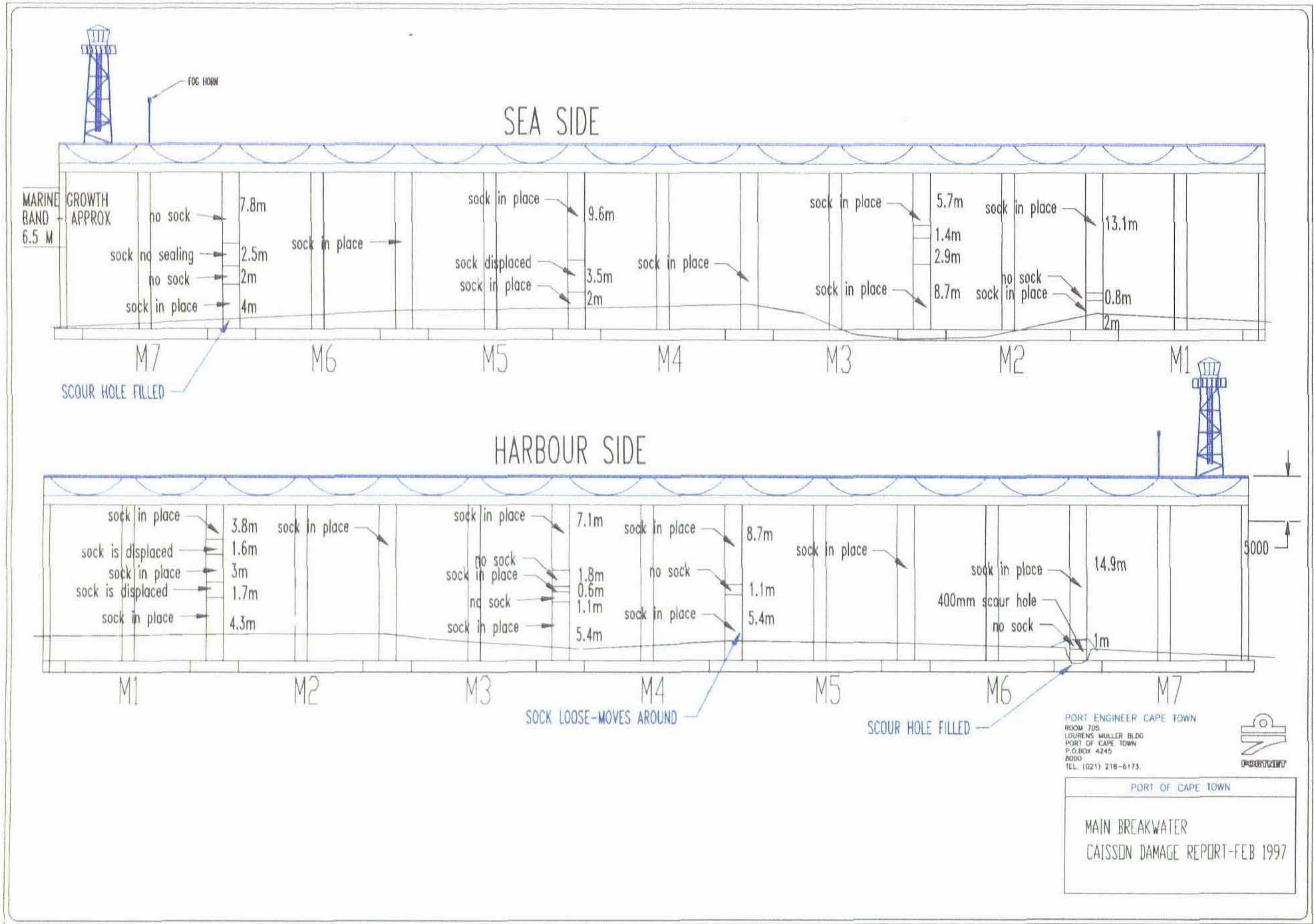
Joints M1 - M2, **M2 - M3,** *M3 - M4,* **M4 - M5** and *M5 - M6*: Above joints were surveyed and no noticeable changes had been observed by the divers.

Thank you.

Diver supervisor: Mr B van der POLL

Annexure B - Damage Report

DWG 2



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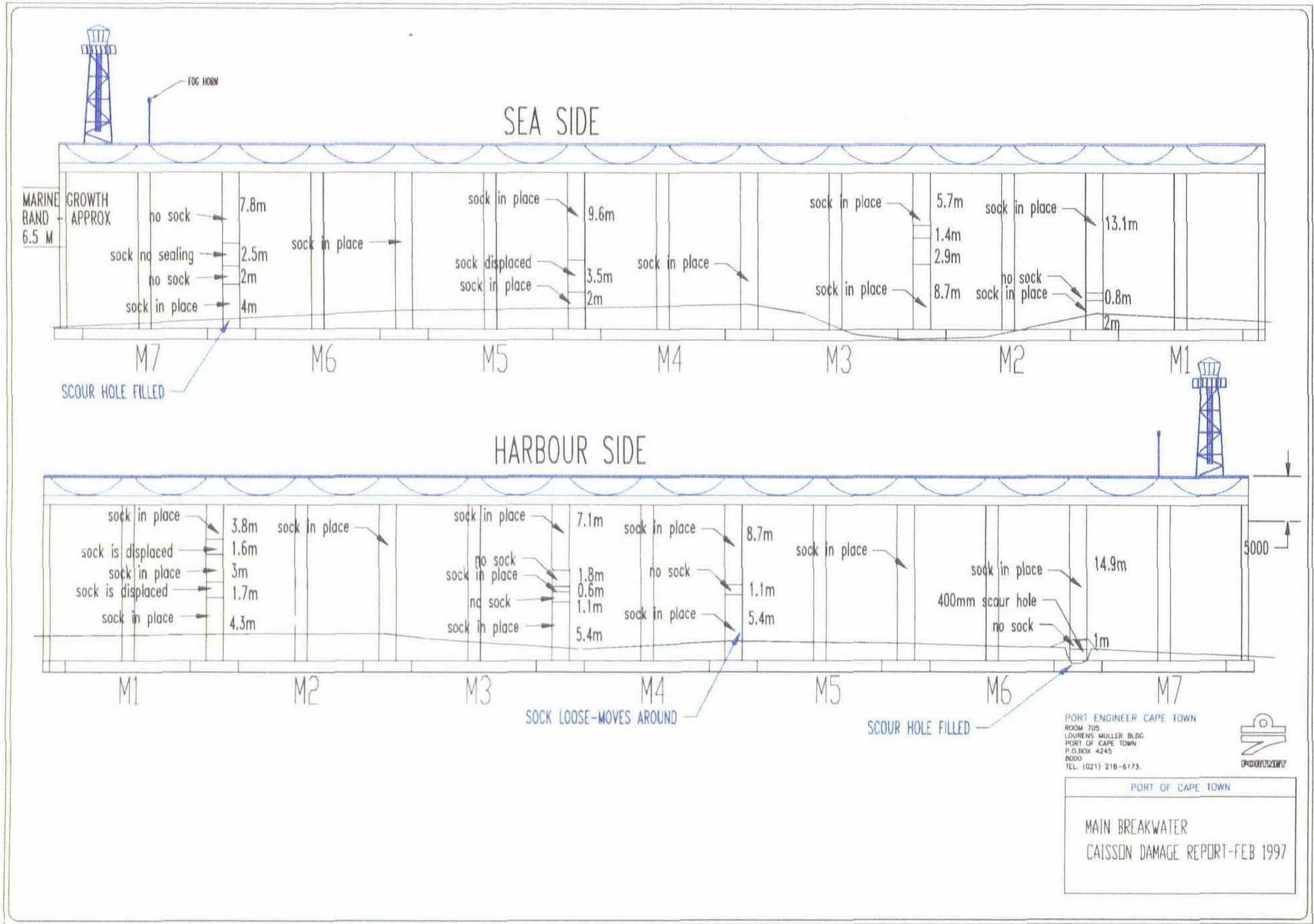
Joints M1 - M2, **M2 - M3,** *M3 - M4,* **M4 - M5** and *M5 - M6*: Above joints were surveyed and no noticeable changes had been observed by the divers.

Thank you.

Diver supervisor: Mr B van der POLL

Annexure B - Damage Report

DWG 2



Annexure C

Diving survey of scour beneath caissons on the breakwater by the CSIR

FAX MESSAGE

CSIR
EMATEK
P. O. Box 320
7599 Stellenbosch
South Africa

Telephone: (021) 887-5101
Telefax : (021) 887-5142

Earth, Marine and Atmospheric Science and Technology

CSIR

TO: Port Engineer - PORT OF CAPE TOWN — Fax No: 021- 252656 / 021 - 405 5020

ATTENTION: Mr D VISSER / D LOURENS — No of pages (incl this one): 3

FROM: Dave Phelp — Date: 13 March 1995

RE: DIVING SURVEY OF SCOUR BENEATH CAISSONS ON BREAKWATER

Further to the diving inspection done by myself and Mr Jefferies of Portnet on 1 March 1995, please find attached sketch giving further details of the scour hole between the last two caissons at the head of the breakwater, and my "off the cuff" comments below:

a) The Observed Situation

a1) As illustrated there is a gap in the double layer of 4 ton concrete blocks on the seaward side (approximately 4m long by 3m wide by 2m deep) directly above where the scour hole is located. This seems to be coincidental, as it is unlikely that wave forces would have moved the blocks.

a2) The blow hole is much larger on the seaward side (~500mm diameter hole, compared to a slot of 150mm by 300mm) than on the harbour side, and is located in the apex of the "V" where the rounded walls of the caisson meet and the flanged bases of the adjacent caissons come together.

a3) The hole does widen beneath the caissons as depicted in the details of the side elevation of original Portnet sketch.

a4) There is a simila but far smaller hole between the second and third caissons.

a5) Around the water level (wave impact zone), there are a number of places where the grout sock has failed, and in some places light is visible through the breakwater.

b) **Probable Causes**

b1) The main cause of the damage above water, is the failure of the grout socks under high impact and pressure forces during large waves breaking/slamming directly onto the face of the caissons.

b2) The scour beneath the caissons is a result of water flow beneath and between the caissons, caused by differential pressures (between the sea and harbour sides) resulting from wave action. These waves cause the greatest flow when there is a trough on the harbour side corresponding to a peak on the sea side (or visa-versa). This is worst where waves wrap around the head of the breakwater, and thus the scour hole is worst between the last two caissons.

b3) The original scour was probably caused by the grout sock not reaching the underside of the caisson to block off most of the flow. As the hole grew, so the water flow increased. The reason for the seaward hole being larger, is that some of the pressure surge is taken up in the space between the caissons.

b4) Besides the pressure force, there is an additional velocity force (dynamic force vs hydrostatic force). The funnel effect of the curved caissons and the "V" of the caisson base could be focusing this component of the force into the exact point where the hole is located.

c) **Possible Solutions**

c1) It is recommended that before any repairs are attempted, the size of the scour hole beneath the caissons should be established. This can either be achieved by a thin diver using a surface air line, or by underwater video (ex CSIR). Both will need underwater lighting and good conditions of no waves and good visibility.

c2) The space beneath the caissons should be filled with stone by pumping a mixture of say bentonite and stone. The hole should be plugged with say concrete. Then the gap in the blocks should be filled with additional blocks, with the first block placed as near as possible directly over the plugged hole.

c3) The main purpose of the repair, other than to reduce future scour, is to replace the stone beneath the caissons, because it is this stone which provides not only the support to the caissons, but the friction needed against sliding failure.

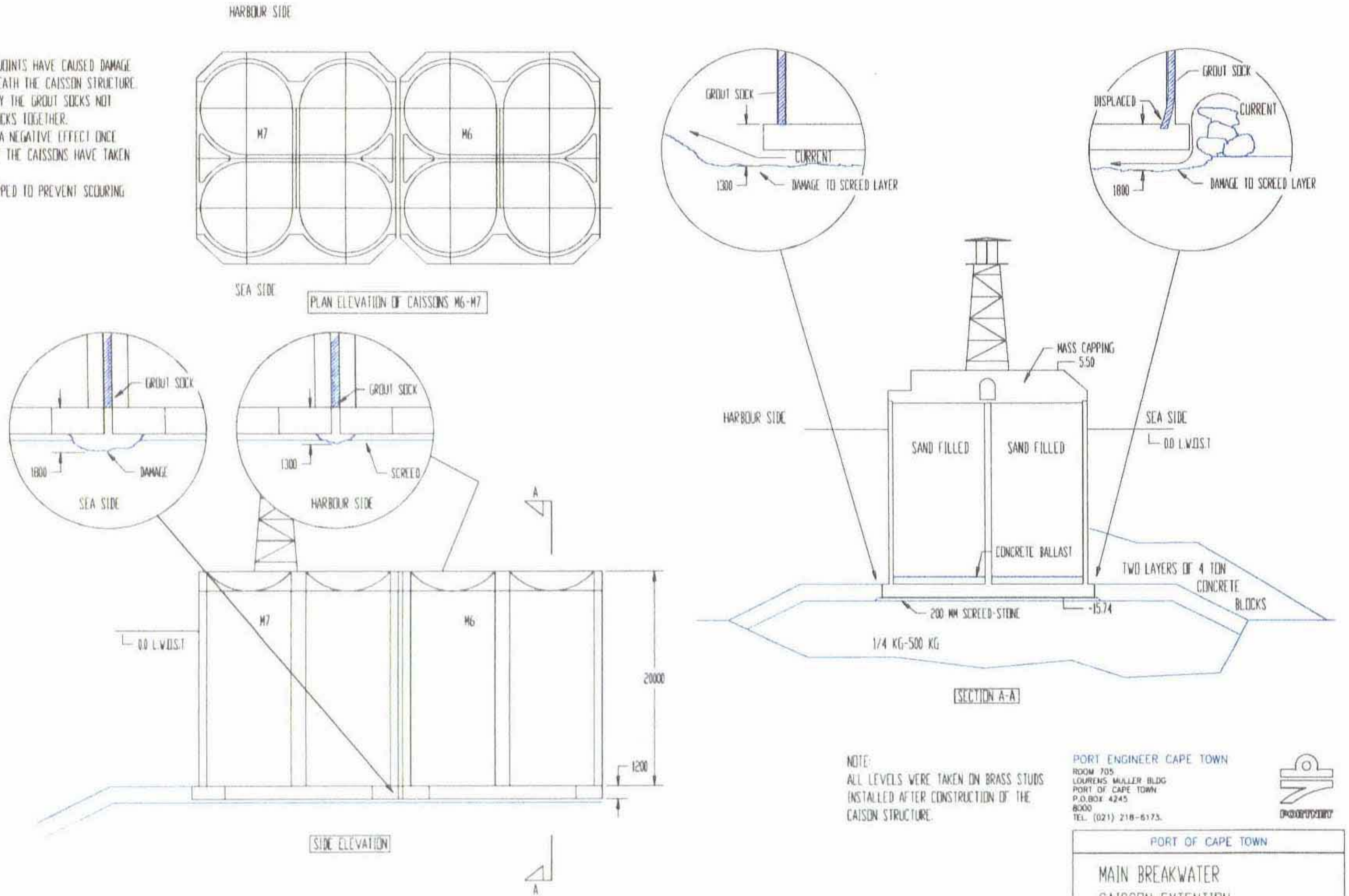
c4) The monitoring of the brass studs on the mass capping should continue as an early warning of settlement or sliding.

Kind regards

DAVE PHELP
COASTAL AND HYDRAULIC ENGINEERING

NOTES:
DAMAGE TO FOUNDATION

A FLOW UNDERNEATH CAISSON JOINTS HAVE CAUSED DAMAGE TO THE SCREED LAYER UNDERNEATH THE CAISSON STRUCTURE. THIS SITUATION WAS CAUSED BY THE GROUT SOCKS NOT SEALING THE TWO CAISSON BLOCKS TOGETHER.
THE GROUT SOCKS ONLY HAVE A NEGATIVE EFFECT ONCE A BREACHED ON BOTH SIDES OF THE CAISSONS HAVE TAKEN PLACE.
THIS CURRENT HAS TO BE STOPPED TO PREVENT SCOURING OF THE FOUNDATION LAYERS.



NOTE:
ALL LEVELS WERE TAKEN ON BRASS STUDS INSTALLED AFTER CONSTRUCTION OF THE CAISON STRUCTURE.

PORT ENGINEER CAPE TOWN
ROOM 705
LOURENS MULLER BLDG
PORT OF CAPE TOWN
P.O.BOX 4245
8000
TEL. (021) 218-6173.

PORT OF CAPE TOWN

MAIN BREAKWATER
CAISSON EXTENTION
DAMAGE REPORT : 31 AUGUST 1994

CHART DATUM HARBOURS

b) Probable Causes

b1) The main cause of the damage above water, is the failure of the grout socks under high impact and pressure forces during large waves breaking/slamming directly onto the face of the caissons.

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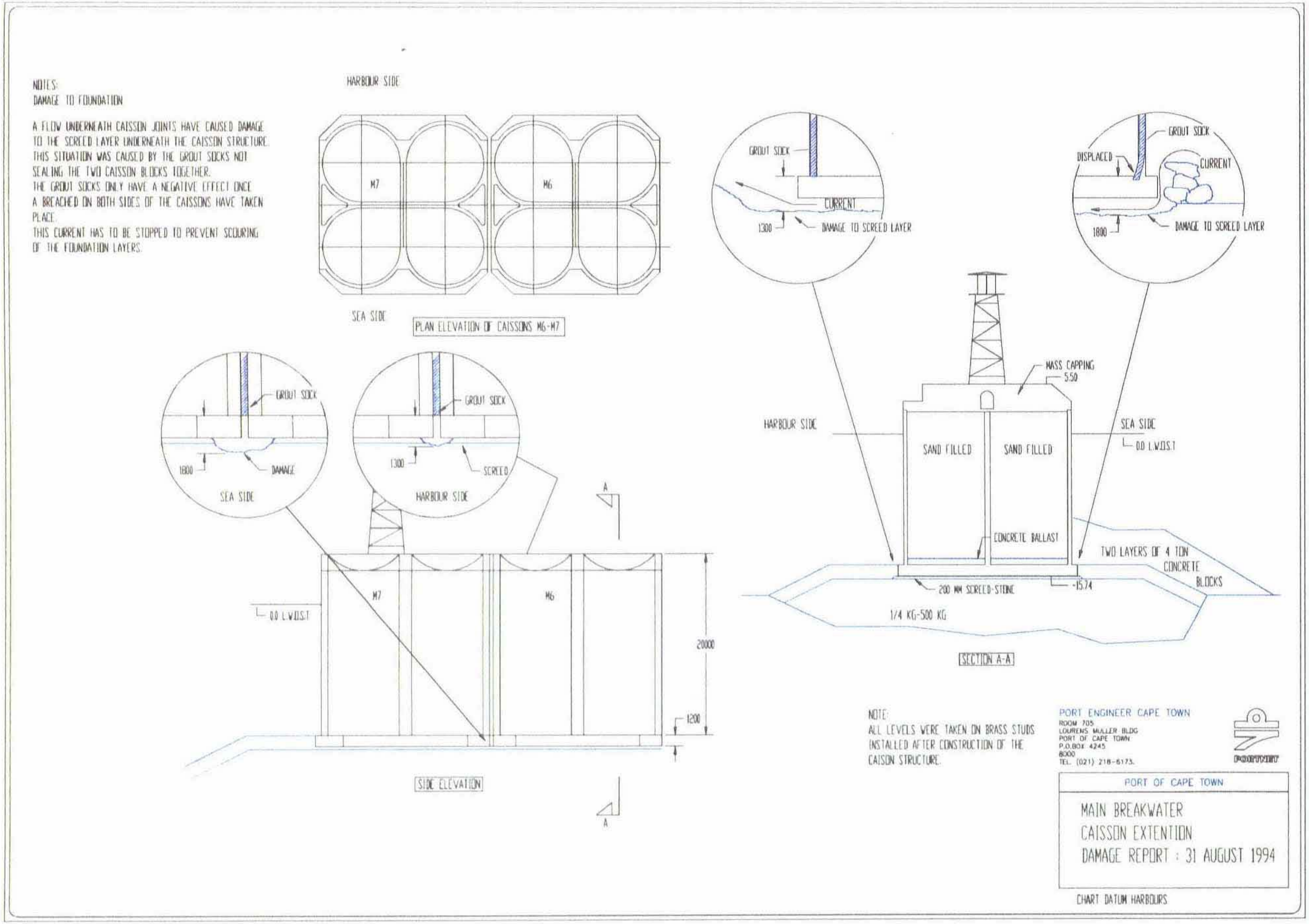
Kind regards

DAVE PHELP
COASTAL AND HYDRAULIC ENGINEERING



Annexure C - Damage Report

DWG 3

Annexure D

Delays to container vessels due to ranging effects in the port

The following are illustrated in the attached list:

1. Date

2. The name of the vessel

3. Type of vessel

4. The quay where the delay occurred

5. The time the delay occurred

6. The delay is expressed in a part of a minute

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					Ranging Delays	
96-01-11	Pongola	Conventional	Coastal	502	05:32	0.15
96-01-19	Sezela	Conventional	Coastal	501	17:06	0.33
96-03-05	Dina	Conventional	Deepsea	600	07:13	0.20
96-03-05	Dina	Conventional	Deepsea	600	07:33	0.27
96-03-29	Msc Carmen	Cellular	Deepsea	600	00:46	0.13
96-04-08	Msc Carla	Cellular	Deepsea	602	23:00	0.33
96-04-08	Sezela	Conventional	Coastal	502	23:26	0.39
96-04-09	Sezela	Conventional	Coastal	502	01:47	0.17
96-04-09	Msc Carla	Cellular	Deepsea	602	01:48	0.17
96-04-09	Sezela	Conventional	Coastal	502	02:47	0.17
96-04-09	Msc Carla	Cellular	Deepsea	602	03:31	0.20
96-04-09	Sezela	Conventional	Coastal	502	05:12	0.28
96-04-09	Msc Carla	Cellular	Deepsea	602	06:16	0.31
96-04-09	Msc Carla	Cellular	Deepsea	602	07:37	0.55
96-04-09	Sezela	Conventional	Coastal	502	09:27	0.15
96-04-09	Sezela	Conventional	Coastal	502	11:42	0.38
96-04-09	Sezela	Conventional	Coastal	502	12:07	0.18
96-04-09	Sezela	Conventional	Coastal	502	12:20	0.20
96-04-09	Sezela	Conventional	Coastal	502	12:46	0.28
96-04-11	UNI Valor	Cellular	Deepsea	604	19:33	0.20
96-04-12	Astra Peak	Conventional	Deepsea	601	01:56	0.17
96-04-14	Diego	Cellular	Deepsea	602	07:53	0.15
96-04-19	Charles Lykes	Conventional	Deepsea	604	10:53	0.12
96-05-07	Sezela	Conventional	Coastal	502	07:13	0.25
96-05-30	Freedom Container	Cellular	Deepsea	600	17:17	0.14
96-06-06	St Blaize	Conventional	Deepsea	603	03:53	0.22
96-06-06	St Blaize	Conventional	Deepsea	603	04:18	0.27
96-06-06	St Blaize	Conventional	Deepsea	603	05:01	0.15
96-06-06	St Blaize	Conventional	Deepsea	603	05:12	0.13
96-06-06	St Blaize	Conventional	Deepsea	603	05:42	0.12
96-06-06	Bunga Massatu	Conventional	Deepsea	601	06:00	0.15
96-06-07	Nolizwe	Cellular	Deepsea	603	07:50	0.89
96-06-07	Nolizwe	Cellular	Deepsea	603	23:12	0.17
96-06-08	Hansa Coral	Cellular	Deepsea	600	01:05	0.15
96-06-08	Hansa Coral	Cellular	Deepsea	600	03:49	0.10
96-06-08	Hansa Coral	Cellular	Deepsea	600	04:04	0.23
96-06-08	Nolizwe	Cellular	Deepsea	603	06:44	0.09
96-06-13	Nordlight	Cellular	Deepsea	601	12:51	0.18
96-06-17	Stefania	Cellular	Deepsea	601	15:31	0.21
96-06-17	Stefania	Cellular	Deepsea	601	16:14	0.15
96-06-18	Hansa Coral	Cellular	Deepsea	604	08:35	0.17
96-06-24	Infanta	Conventional	Deepsea	603	17:07	0.21
96-06-24	Infanta	Conventional	Deepsea	603	17:22	0.15
96-06-24	Infanta	Conventional	Deepsea	502	17:38	0.09
96-06-24	Gamtoos	Conventional	Coastal	502	18:02	0.42
96-06-30	Msc Giovanna	Cellular	Deepsea	601	00:49	0.08
96-06-30	Msc Giovanna	Cellular	Deepsea	601	01:05	0.10

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96-04-08	Msc Carla	Cellular	Deepsea	602	23:00	0.33
96-04-08	Sezela	Conventional	Coastal	502	23:26	0.39
96-04-09	Sezela	Conventional	Coastal	502	01:47	0.17
96-04-09	Msc Carla	Cellular	Deepsea	602	01:48	0.17
96-04-09	Sezela	Conventional	Coastal	502	02:47	0.17
96-04-09	Msc Carla	Cellular	Deepsea	602	03:31	0.20
96-04-09	Sezela	Conventional	Coastal	502	05:12	0.28
96-04-09	Msc Carla	Cellular	Deepsea	602	06:16	0.31
96-04-09	Msc Carla	Cellular	Deepsea	602	07:37	0.55
96-04-09	Sezela	Conventional	Coastal	502	09:27	0.15
96-04-09	Sezela	Conventional	Coastal	502	11:42	0.38
96-04-09	Sezela	Conventional	Coastal	502	12:07	0.18
96-04-09	Sezela	Conventional	Coastal	502	12:20	0.20
96-04-09	Sezela	Conventional	Coastal	502	12:46	0.28
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96-04-14	Diego	Cellular	Deepsea	602	07:53	0.15
96-04-19	Charles Lykes	Conventional	Deepsea	604	10:53	0.12
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96-06-06	St Blaize	Conventional	Deepsea	603	04:18	0.27
96-06-06	St Blaize	Conventional	Deepsea	603	05:01	0.15
96-06-06	St Blaize	Conventional	Deepsea	603	05:12	0.13
96-06-06	St Blaize	Conventional	Deepsea	603	05:42	0.12
96-06-06	Bunga Massatu	Conventional	Deepsea	601	06:00	0.15
96-06-07	Nolizwe	Cellular	Deepsea	603	07:50	0.89
96-06-07	Nolizwe	Cellular	Deepsea	603	23:12	0.17
96-06-08	Hansa Coral	Cellular	Deepsea	600	01:05	0.15
96-06-08	Hansa Coral	Cellular	Deepsea	600	03:49	0.10
96-06-08	Hansa Coral	Cellular	Deepsea	600	04:04	0.23
96-06-08	Nolizwe	Cellular	Deepsea	603	06:44	0.09
96-06-13	Nordlight	Cellular	Deepsea	601	12:51	0.18
96-06-17	Stefania	Cellular	Deepsea	601	15:31	0.21
96-06-17	Stefania	Cellular	Deepsea	601	16:14	0.15
96-06-18	Hansa Coral	Cellular	Deepsea	604	08:35	0.17
96-06-24	Infanta	Conventional	Deepsea	603	17:07	0.21
96-06-24	Infanta	Conventional	Deepsea	603	17:22	0.15
96-06-24	Infanta	Conventional	Deepsea	502	17:38	0.09
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