FILE: MANSOUR

CALIFORNIA SEA GRANT REPORT
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DEVELOPMENT OF A METHODOLOGY FOR THE DESIGN, CONSTRUCTION AND QUALITY ASSURANCE OF RUBBLE MOUND BREAKWATERS

BY

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(Gerwick; R/OT-5)

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INTRODUCTION

Aquatic structures suffer a severe punishment from nature. Storms, strong waves, currents, sea floor conditions, winds and marine fauna are the major elements responsible for this destruction.

Building marine structures in the ocean is not an easy task. It is also a very expensive operation. Unfavorable weather conditions could happen during the lifetime of the structure as well as during construction. For these reasons, until today, the design and the methods of construction of these structures are not well developed.

One of the oldest and most important offshore structures is a breakwater. Breakwaters are structures constructed for the purpose of forming an artificial harbor with a water area so protected from the effect of sea waves as to provide safe accomodation for shipping (Quinn,1972). They will break or dissipate the force of waves and thus prevent their incidence on an area it is desired to protect (Wiegel,1975).

With the advent of the very large crude oil carriers (VLCC) and deep draft iron ore carriers, and colliers, breakwaters have had to be constructed in much deeper water, so as to provide adequate depth (nominally 25m or 80 feet) behind their protection.

The importance of breakwaters is shown by their large inital cost but even more importantly, entire developments and projects depend on their continued function. Failure of a breakwater can often have far reaching consequences that greatly exceed the breakwater's initial cost. For many years these structures have been designed on the basis of empirical formulae and rules-of-thumb. However, as these structures have been built in deeper water serious scale effects with large quantity and cost over runs and at least one catastrophic failure (Sines breakwater in Portugal) have resulted.

For a large percentage of breakwaters, records indicate eventual failure at one time or another. Recently a major breakwater failed at Sines, Portugal shutting down facilities of several billion dollars. What is the underlying cause of these failures? Is it the type of material used, the methods of construction or the severe conditions occurring in service which reflect deficiencies in the design procedure?

To overcome these problems, researchers and specialists in this field have studied different ways to provide a better protective layer (armor layer) that will withstand the wave action. Several designs and shapes of the units of this layer have been developed. However, there are very few known current studies on the core, which is the mass portion of the breakwater. The interfaces between the core and the armor layer and between the core and the natural seafloor become critical. Control of tolerances has a direct effect on the quantity of the expensive armor, yet this control becomes increasingly difficult in deeper waters and greater exposure to wave action. Erosion of the core during construction becomes a significant factor, particularly at the leading edges.

Construction of a breakwater in specific and development of a harbor in general is a major project that the government should determine its overall objectives and importance before approving its validity. Since this project will occupy a substantial portion of land which is considered a natural resource it is imperative to review its benefits and advantages in accordance with the total system. A nontechnical study on this subject is very well presented by C. West Churchman in his book "The Systems Approach". In this book, Churchman discusses the planning and management interaction in government, business, industry, and human problems. Specifically, this review is aimed to spell out in detail the objectives of such a structure and the performance measures which will evaluate the use of its resources. A continuation of this discussion is presented by the same author in another book entitled "The Systems Approach And Its Enemies".

This report will focus on the design and construction of the core of rubble mound breakwaters.

The first chapter will present a brief history of breakwaters. It will also define in detail the functions of the core (quantitatively to the degree possible) in terms of its role in the total breakwater system.

The second chapter will present an investigation of some major breakwaters built in different parts of the world. It will also discuss the importance of the presence of the core material and its effect on the permeability and wave energy transmission.

Since a rational design of the total breakwater construction requires understanding of the interactive effects of the core with its foundation, its armor and the sea, a review of different sea floor conditions will be presented in the third chapter. It will also include several recommendations for the preparation of the sea floor.

Chapter Four will focus on the available rock material used for the construction of the core. The characteristics of the desired rock will be described along with the recommendations in this area, such as rock quality, durability, gradation and specific gravity. The filter layers between the core and the armor will also be addressed.

Chapter Five will present the effect of the construction procedures adopted on the adequacy, cost and performance of the final structure. This chapter will include a review of the available equipment and the current methods of construction.

The closing chapter (Chapter VI) describes the current methods used in the survey, and tolerance controls of the core of rubble mound breakwater. Control of tolerances has a direct effect on the quantity of the expensive armor, yet this control becomes increasingly difficult in deeper waters and greater exposure to wave action.

The Conclusion will present a complete review of the recommendations cited in the previous chapters as well as an overview of the total concept of the design and construction, from the practical aspects, of the core of rubble mound breakwaters.

CHAPTER I

FUNCTIONS OF THE CORE

HISTORY OF BREAKWATERS

Breakwaters are not the invention of our modern time. We should have to go back very far, indeed, into the history of artificial harbours to trace their origin. The historical development of ports is directly related to the growth and extension of maritime trade. Indeed there were ships upon the Red Sea long before the Pyramids were built, and there were vessels on the Mediterranean Sea and Persian Gulf by 7000 B.C.

Henry Cornick (1968) stated that, "Among the earliest ports ever built was the port on the Northern Egyptian coast. It was named A-Ur and was built on the Canopic branch of the Nile by 4000 B.C. Unfortunately its exact site is unknown. The next Egyptian harbor that followed was the great port of Pharos (Saville,1940). It was located between Ras-El-Tin and the rock of Abu-Bakar. The remains of its breakwater can be traced facing the sea to the north. This one was 2590 m long and consisted in the main part of two rubble mound structures between 40 and 60 m apart, each with an upper width of 8 to 12 m and 6 to 9 m high. These were made of very large blocks and the space between them was filled with smaller stone. The top width was between 55 and 76 m (Wiegel, 1964).

Actually, breakwaters were a common feature to most of the ports of the ancients. In the ancient harbours of Tyre which were built on the coast of Phoenicia around 1000 B.C., their breakwaters were formed by massive stones weighing as much as 15 tons each. "Their construction showed a great advance upon the work of Pharos. There were two walls of hewn stone keyed together with metal dowels, the space between the walls being filled with concrete." (Henry Cornick, 1968)

The Greek harbours, such as that built by Themistocles at Piraeus in 493 B.C. employed what would now be regarded as composite breakwater construction. "Pierre Perdue"(lost rocks) or heavy rubble was first tipped into the sea and followed by a superstructure of heavy blocks dowelled together.

Roman harbours were commonly protected by arched moles, generally regarded as the forerunner of the modern piled jetty. The Roman mole, whose purpose was threefold, allowed access round the harbour, acted partially as a breakwater and tended to inhibit general silt deposition by virtue of the flow through its openings (Townson, 1973).

It appears, therefore, that the alternative characteristics of breakwater design, namely the rubble mound and the vertical wall, were present before the end of the Roman civilization.

By 340 B.C., Alexander founded the city of Alexandria and built the port by connecting the island of

Pharos with the mainland by reclamation. The port of Alexandria is considered the main port of Egypt. At the present time the port authority is extending the breakwater another 600 m and reinforcing and repairing the old port.

The next figure (1.1) shows a cross section of a breakwater built in 1870 at the port of Alexandria, Egypt. It was then remodeled in 1905. The different layers of the breakwater were installed one on top of the other horizontally without any protection of cover layers of the slopes. The earlier design was a mound of two types of protection units, some natural rock and others concrete blocks, piled up to form the required cross section of the breakwater. Most of the blocks were placed towards the sea side and the natural rock was placed toward the harbour side. There were no core material or filter layers used in this design.

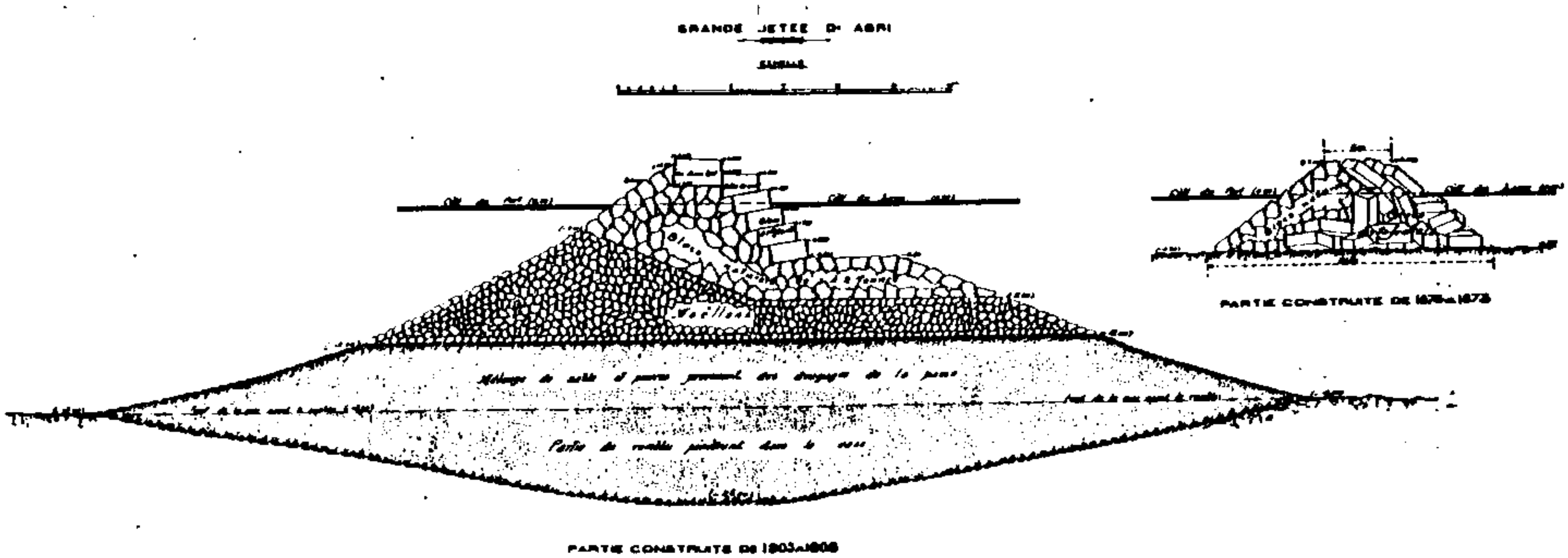


Figure 1.1: Alexandria, Egypt Breakwater

Christopher Columbus' voyage in 1492 to what he thought was India gave a great stimulus to international trade and incidentally thereby to breakwaters (Wells,1945). Not long after this event the Old Mole at Genoa was severely damaged by a storm in 1498. It was examined by Leonardo De Vinci, who observed the effects of large wave forces but apparently failed to suggest any far reaching solution (Hammond, 1956).

One of the best examples of breakwaters that is in existence at this time is the Plymouth breakwater located in England. It was constructed in 1806 in approximately 15m of water. This 15,000m long offshore breakwater is exposed to Atlantic gales. The concept of the design was based on dumping stone continuously and allowing waves to shape the mound into a stable profile. The exposed section of the breakwater above low water was armored with a layer of tightly fitting limestone blocks on a slope of 1:5. The breakwater is periodically maintained (Riley, 1958) but it has, almost certainly, experienced the one in one hundred year storm without any appreciable amount of damage to the structure. The next figure (1.2) shows a cross section of the Plymouth breakwater.

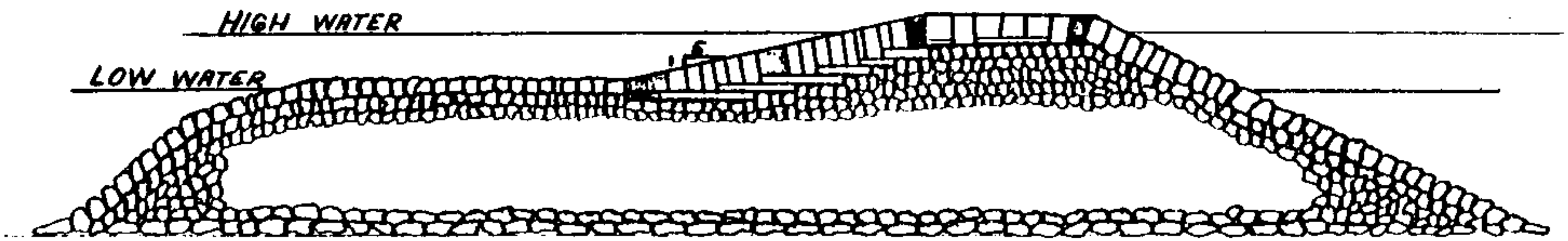


Figure 1.2: Plymouth Breakwater

FUNCTIONS OF THE CORE

The armor and the breakwater cap can be considered to be the superstructure of the breakwater. The core is the substructure. A breakwater is a complete system. It is designed by its different components, but it functions as one unit. The core of the breakwater constitutes the largest component by volume, hence it is desirable to reduce the unit cost of core material to the minimum. The greatest cost-effectiveness in quarrying and core construction is usually obtained by utilizing most or all of the quarried rock material in the combined core and armor layers.

The functions of the core of a typical rubble mound breakwater could be summarized as follows:

1. Furnish the mass
2. Provide stability
3. Provide support for the riprap and armor units
4. Reduce wave energy permeability
5. Act to transfer the wave forces and gravity loads to the foundation.

A knowledge of wave-structure-seabed interaction is very important in coastal hydrodynamic problems. The wave load transferred to the core through the armor units depends on the pressure gradients in these particles (Moshagen, Hermann, and Torum 1975). Understanding this complete system, as a whole, as well as each individual component by itself will lead to a rational design and a more comprehensive and thorough methodology for the construction of the core of rubble mound breakwater. This in turn will result in a more rational, economical and safe structure.

CHAPTER II

REVIEW OF SOME MAJOR BREAKWATERS

This chapter will present a review of some major breakwaters visited by the author in order to aquire a better understanding of the design and construction of the core of rubble mound breakwaters. This site investigation was undertaken during the early part of 1981.

SINES BREAKWATER, PORTUGAL

One of the most important harbours in Europe is the Port Sines, Portugal, which is mainly intended for oil and mineral tankers. In view of its excellent conditions, the government of Portugal planned this industrial base associated with an ocean terminal. Sines is practically the southwestern tip of Europe. It lies 160 km south of Lisbon. The harbour of Sines is a deep water seaport with depths reaching 50 meters at a short distance from the shore and is designed to berth ships up to 500,000 dwt. This capacity together with its geographical situation at the intersection of the main Atlantic Ocean routes, demonstrates the port's location as a highly competitive base for short storage and redistribution of bulk solids and liquids.

The main breakwater is 2 km. long. For its construction, about six million cubic meters of rock fill was used. Beside providing shelter from the Atlantic Ocean for the port, the breakwater supports oil pipelines for offloading to mainland storage facilities.

The breakwater is an artificial rubble mound with a rock fill core armored with larger stones. On the seaward side the stone is blanketed with 42 tons of concrete dolos.

The breakwater has a concrete superstructure composed of a wave deflector and a platform for laying pipes running all along its length. Figure (2.1) represents a layout of the Sines Harbour including the main breakwater and the three berths which are the crude oil terminals.

Figure 2.1: Layout of Sines Harbor, Portugal

The design of the breakwater started in March, 1972 and the construction followed in August, 1973. The core of this new breakwater was made out of quarry fill rock brought from a site located near the port. The filter layers were made out of heavier cut stone.

Being a non developed region, very little of its geology and nothing of the engineering geological properties of their formations in land and offshore were known when the project started.

Zwamborn,1979 presented the design of the main elements used in construction:

ELEMENT TYPE	**WEIGHT OF UNITS**	**CHARACTERISTICS**
Core material	1 to 3000 kg	specific gravity 2.9 to 3.1
Underlayer Stone	3 to 6t	1: 1.25 slope
Toe protection Stone	16 to 20t	1: 1.33 slope
Dolos trunk	40t	1: 1.5 slope
Dolos head	40t	1: 2 slope
Concrete Capping	4000 to 5000t	15m longsections

According to specifications two grades of core material were specified:

a. The principal grade TOT (the term used by the Portuguese referring to quarry run material used in the core) complies with the following:

1. Core material shall not contain overburden or any clayey organic or other deleterious material.

2. It shall consist of rock evenly graded from 1 kg to 3000 kg.

3. It may contain broken rock fines under 1 kg not exceeding 5% by weight.

4. The quantity of rock under 10 kg in weight shall not exceed 15% by weight.

b. The selected grade (selected TOT) used in the main cross section only above level -2.5 meters CD and in the route of the breakwater is similar but the final material under 10 kg is restricted to between 5% and 10% by weight.

The selection of a quarry for the exploitation of suitable rock was an important engineering geological task. The study started with geophysical exploration campaign consisting of electrical resistivity profiles aiming at the elaboration of a resistivity map of the area and of seismic refraction soundings. A program of borings was prepared firstly to check the validity of these geophysical exploration and secondly to obtain the necessary geological and geotechnical information about the rock mass considered of good quality. A total of 31 rotary boreholes were drilled dipping from 45° to 90° their length ranging from 25 to 60 meters (Oliveira, Rodrigues, Coelho, 1978).

The results of this investigation concluded that the quarry chosen seemed suitable for the purpose of the construction of the rubble mound breakwater. The rock consisted of gabbro and diorite.

Most of the rubble core was dumped by 1,000 ton hopper barges in the middle of the core to form the berm up to elevation -15m in the winter and to -10m in the summer. The remainder of the core was placed directly by dump trucks of 50 tons to 65 tons. The underlayer stone was placed by 1,400tm floating cranes and a 1,000tm crawler-mounted crane. The floating cranes were also used to place the 16 to 20 ton toe protection.

Figure (2.2) represents the design cross section of the Sines breakwater.

Figure (2.3) shows the progress of core construction by month.

Figure (2.4) is a schematic of different methods of construction used in placing the core material and the underlayer stone.

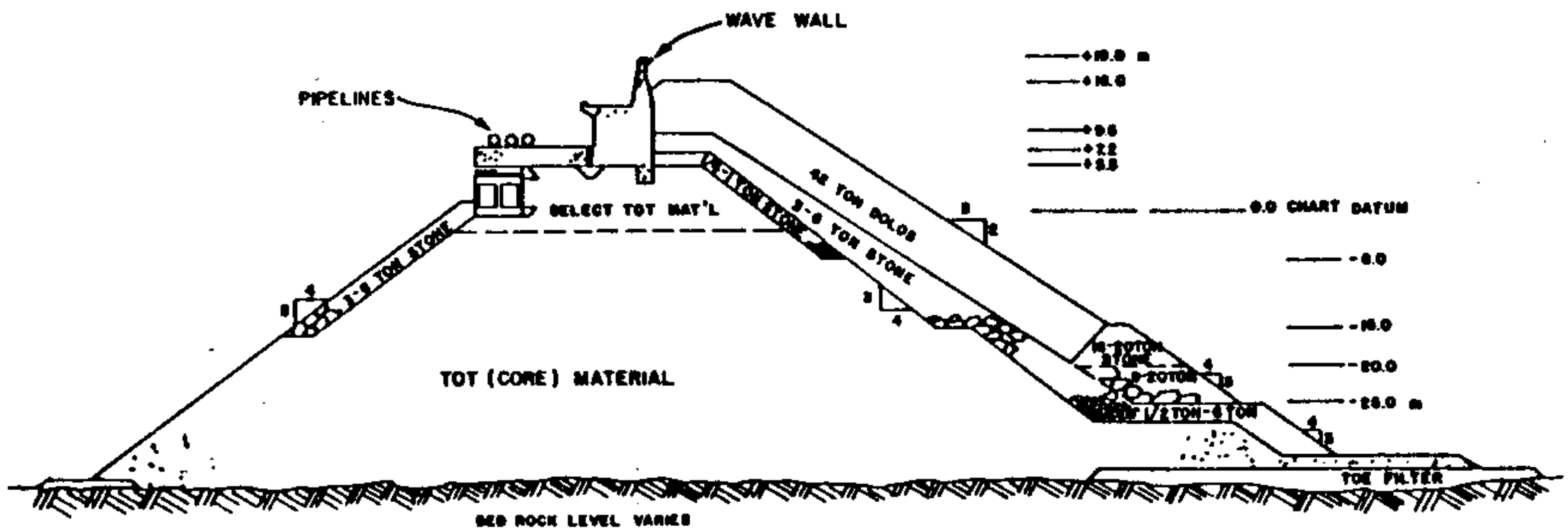


Figure 2.2: Cross Section of Sines Breakwater, Portugal

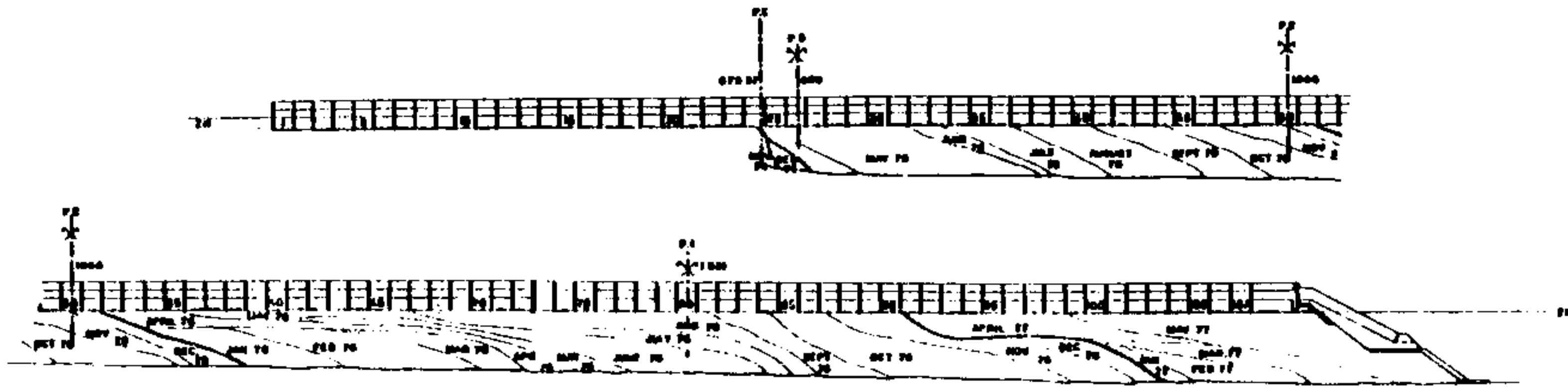
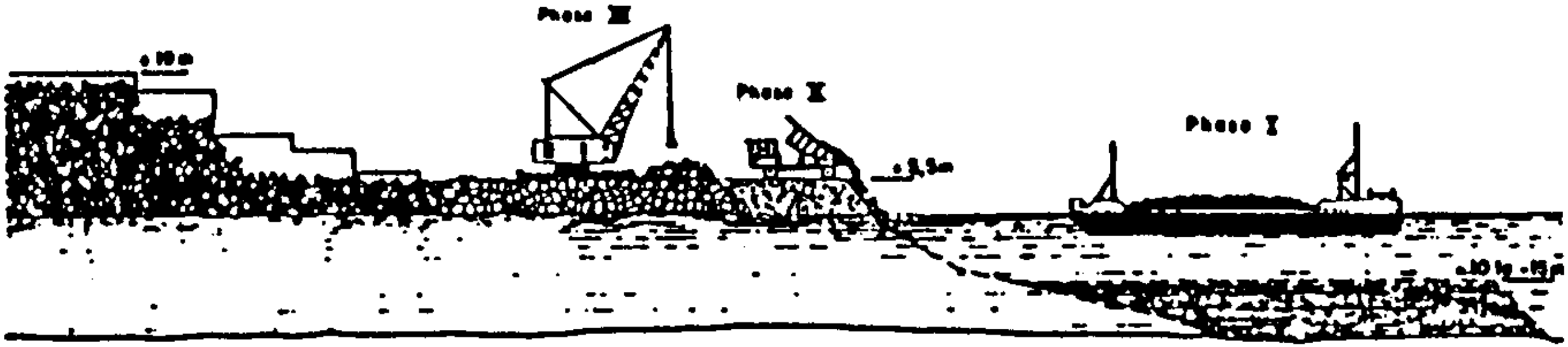


Figure 2.3: Monthly Progress Of Core Construction On The Sines Breakwater
(Port Sines Investigating Panel,1982)



BARGE AND TRUCK DUMPING OF CORE MATERIAL AND
PLACING OF UNDERLAYER STONE

Figure 2.4: Methods Of Construction, Of The Core Material
Sines Breakwater, Portugal
(Port Sines Investigating Panel,1982)

On the 26th of February, 1978 a severe storm destroyed most of the breakwater. In December, 1978 and in February, 1979 more storm action removed most of the armor units and also some temporary remedial works placed in the Fall of 1978. The construction of the breakwater was not completed when the first storm occurred.

The cross section of the breakwater after the storm has a much flatter slope than the original design. The large stones and some broken dolos were found at the bottom of the slope and the smaller rock was at the top (select TOT). Figure (2.5) represents the profile of the cross section before and after the storm.

Some remedies were made as attempts to stop a complete failure of the remainder of this structure:

1. The berm was extended toward the sea then a slope of 1 : 2 was made for a horizontal distance of 42m.
2. This slope changes to 1:8 for another horizontal distance of 16.8m and then changes to 1:3 for a distance of 44.7m.
3. There is a level distance afterwards of 6m and the contour of the breakwater takes another slope of 1:3 for a 46.5m horizontal distance.

4. The toe of the breakwater is leveled 15m towards the sea.

5. The total horizontal distance from the wave wall is approximately 211m.

6. The change in the cross section was made in the seaward side only (a change of almost 100m in width).

7. Instead of using dolos this time they used blocks of concrete of 90 ton weight.

Figure (2.6) represents part of the breakwater subjected to the wave action on a calm day. This picture was taken in March, 1981 on a visit to the Sines Port.

Figure (2.7) illustrates the quarry for the rock used in the Sines breakwater. This quarry is located 5 km from the construction site.

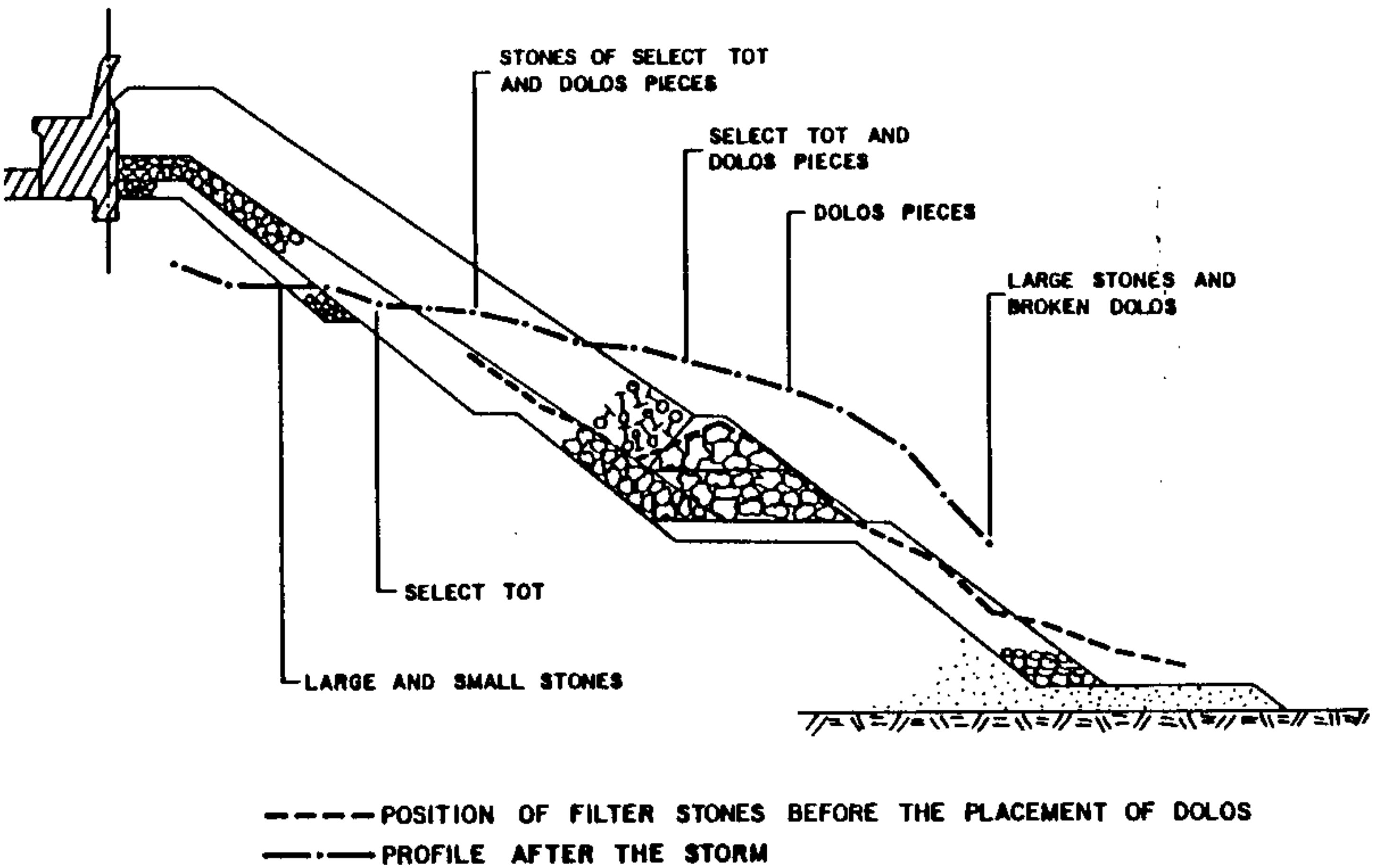


Figure 2.5: Profile Of The Cross Section Of Sines, Before And After The Storm Of 1978 (Port Sines Investigating Panel,1982)

Figure 2.6: Sines Breakwater Subjected To Wave Action

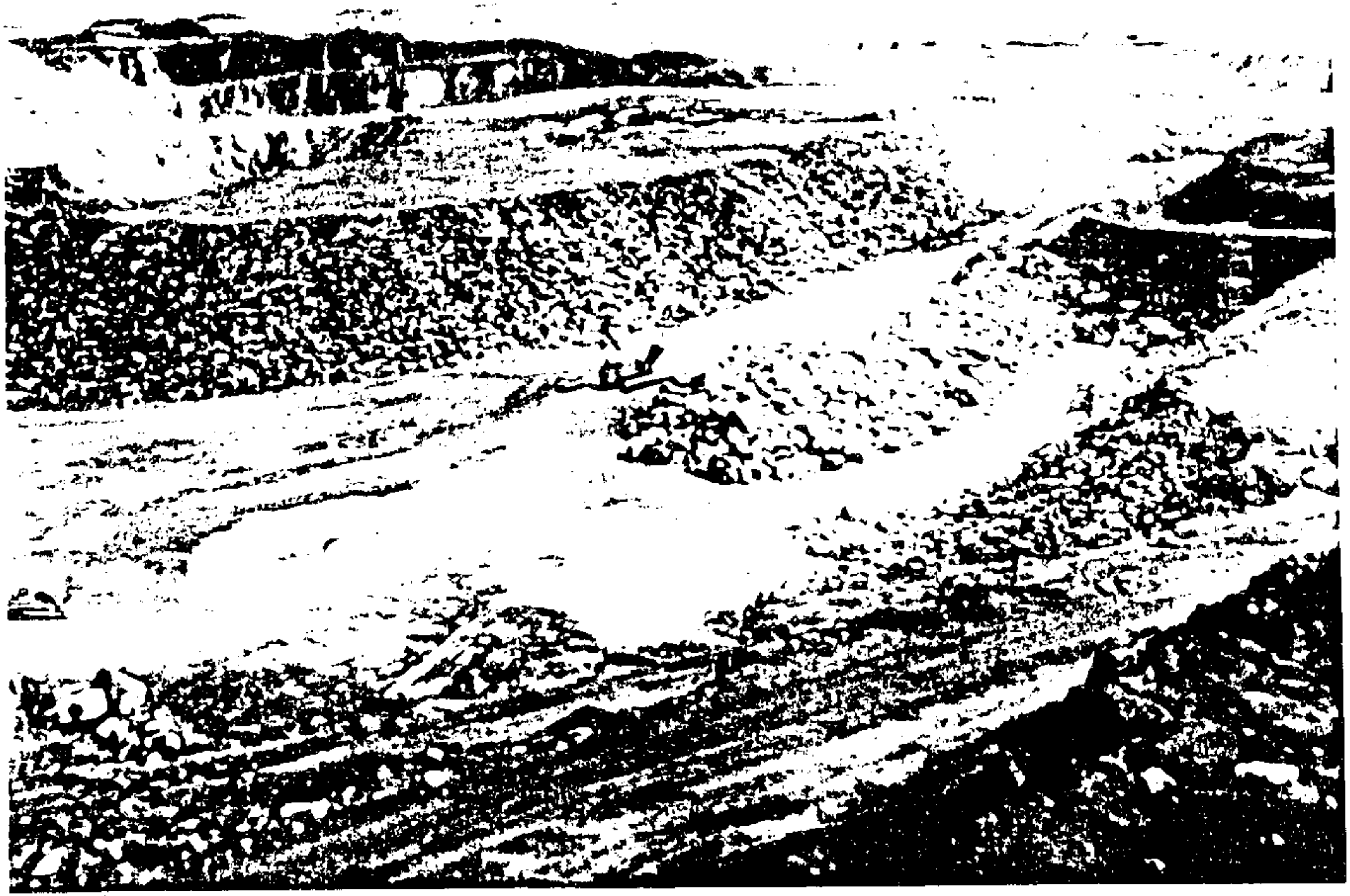


Figure 2.7: Quarry For Sines Breakwater

BILBAO BREAKWATER, SPAIN

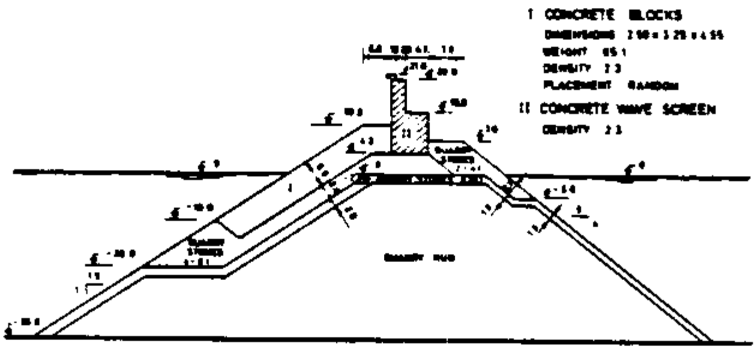
The breakwater at the oil harbour of Bilbao has a total length of 2,400m. The construction was completed in 1976. The core of the breakwater consists of quarry run material. The filter layers have two thicknesses: 2m on the sea side and 1.3m on the harbour side. It was made out of 0.35 ton quarry stone. There is also a second filter toward the sea side made of heavier stones 4 to 8 tons in weight. The thickness of this layer is 3.4m. The front armor layer consists of 65 ton concrete blocks in a thickness of 6m. The dimensions of these blocks are 2.5 X 3.25 X 4.55m. The density is 2.3. These blocks were placed randomly.

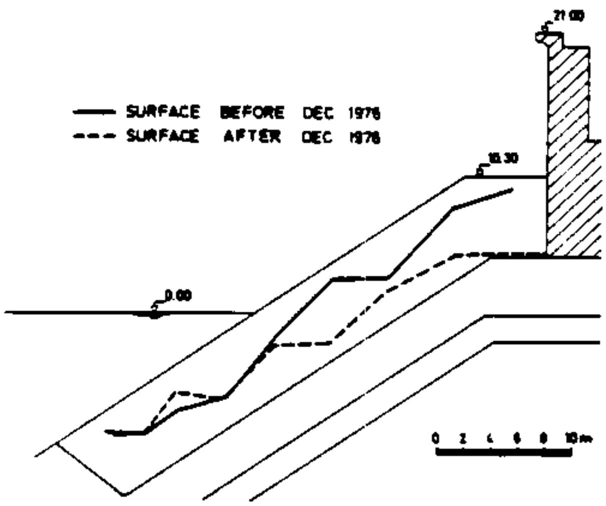
In the first days of December, 1976 the breakwater at Bilbao was subjected to a heavy storm, which did major damage to the structure. Actually, this was the second storm that this breakwater was subjected to during the same year. The first storm occurred in March, 1976 and there were no repairs made at that time. (Torum, Mathlesen and Escutia, 1979)

Figure (2.8) is a photograph of the breakwater located at Bilbao, Spain taken during the visit of March, 1981.

Figure (2.9) illustrates the design cross section of the breakwater. It also shows the profile before December, 1976, and after the storm.

Figure 2.8: Bilbao Breakwater, Spain


I CONCRETE BLOCKS
DIMENSIONS
WEIGHT
DENSITY
PLACEMENT RANDOM
II CONCRETE WAVE SCREEN
DENSITY
QUARRY STONES
QUARRY RUN


SURFACE BEFORE DEC 1976
SURFACE AFTER DEC 1976
21.00
10.30
0.00
0 2 4 6 8 10m

The water depth inside the breakwater was maintained at 31m to facilitate the large oil tankers up to 500,000 dwt. The rock used in the core was quarried from the shoreward end of the dike at Punta Lucero.

In completing the repairs of the breakwater the cross section was changed and became wider. On the Harbor side it was increased 15m and on the sea side it was increased by 50m. The thickness of the filter layers were increased from 1.5m to 2.7m. The slope was also changed from 1 : 1.5 to 1 : 2. The armor blocks of 65 tons were replaced by heavier blocks of 150 tons. The concrete wave screen or the wave wall was widened from 4m to 12m.

Figure (2.10) represents a mix of different sizes of rock used in the core. This picture illustrates the gaps in the gradation of the rocks used in the breakwater. Voids tend to happen when there is no care taken in the gradation of the rocks used, which in turn could lead to hydrostatic pressure as well as air pressure inside the core during the wave action.

Figure (2.11) is a photograph of the face of the quarry used for the production of the core material as well as the filter layers.

Figure (2.12) illustrates part of the Bilbao breakwater after the repairs. This picture was taken in March 1981.

Figure 2.10: Different Sizes Of Rock Used In The Core Of Bilbao Breakwater, Spain

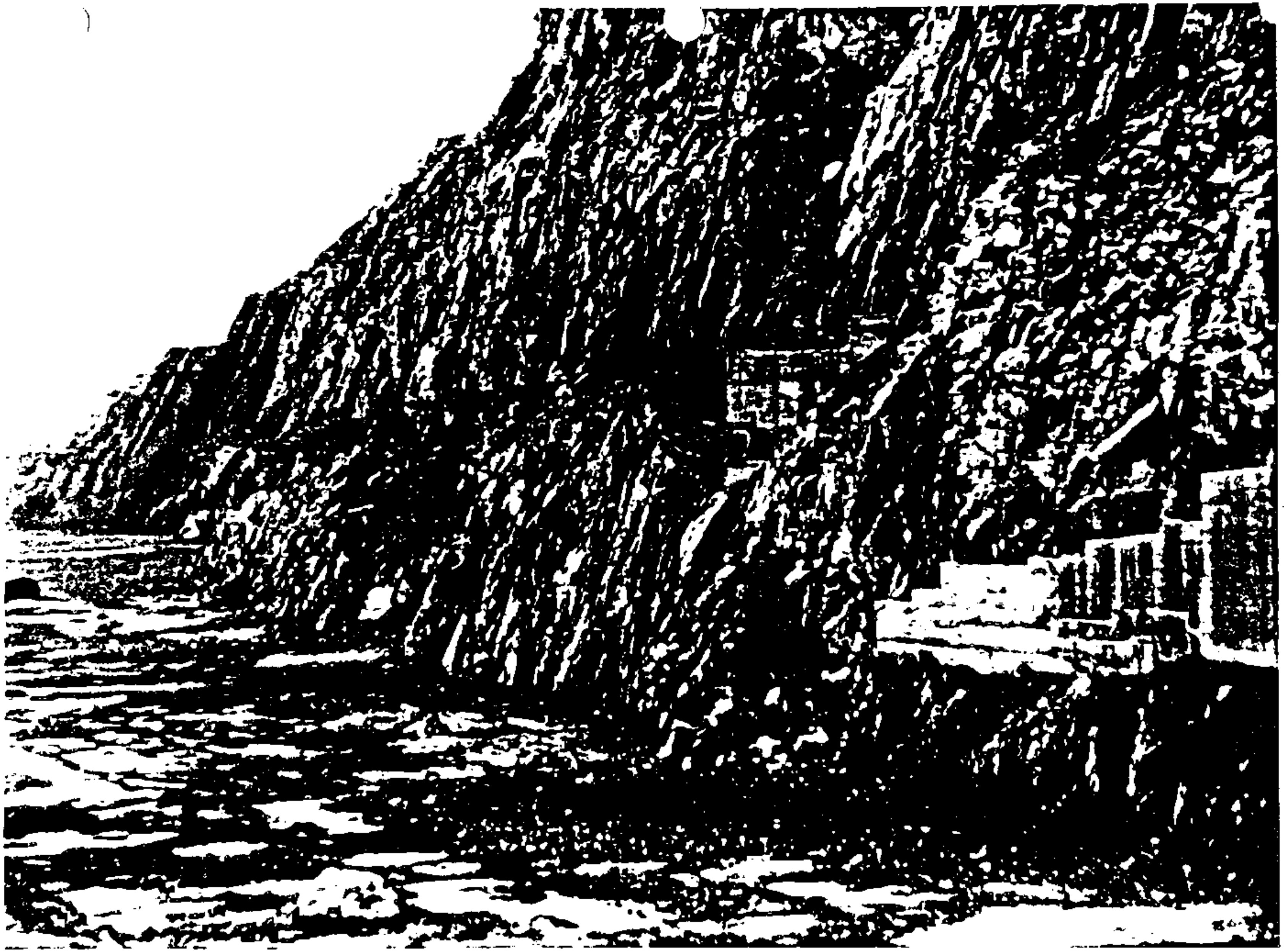


Figure 2.11: Quarry Used For The Core Of Bilbao Breakwater

Figure 2.12: Part of Bilbao Breakwater After The Repairs

Part of Bilbao breakwater is presented in the following picture, figure (2.13), which illustrates the size of the breakwater in comparison to the size of a man standing on the cap.

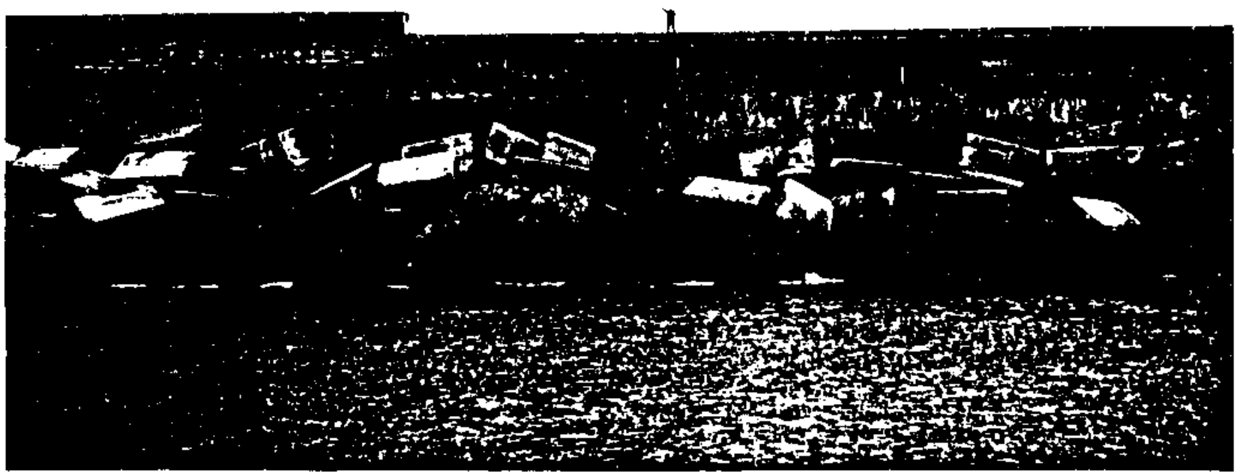


Figure 2.13: Comparison In Size Of Man Against Bilbao Breakwater

HUMBOLDT BAY, CALIFORNIA

Humboldt Bay is located on the northern coast of California, approximately 450 km north of San Francisco. The entrance to the harbour is protected by two rubble mound jetties and is dredged to a 12.2m depth.

The difference between a breakwater and a jetty is not significant. For this reason the rubble mound Humboldt jetty has been included in this chapter.

The structures were initiated in 1889, Magoon (1974) stated that the quantity of stone placed for repair has been greater than the quantity placed in the initial construction (more than one million tons).

Figure (2.14) is a schematic of the seaward head South jetty.

A timber trestle was used in the construction of these jetties in 1888. Four-pile bents sixteen feet apart formed the trestle. These bents supported two standard gauge tracks of forty pounds T rails.

A layer of brush mattresses was used to provide the jetties with a good base before dumping the core rock material. These brush mattresses were made out of grillage of poles that were bound together at every intersection with strong wire. Bundles of brush (12 feet long) were placed upon the grillage in successive layers. Once this grillage reaches the thickness of 6 feet, another grillage is placed on top of it. Long screws were used to tie the two grillages together and compress the brush to two thirds of its original volume. The grillages were then bound together by wires and the screws are removed. Small rocks were placed on top to provide the mattress with sufficient weight to serve as ballast.

The core rock material was constructed from rocks brought to the site by cable cars. The rock was dumped in the water by opening the car doors and releasing the load. The trestle was built to last only long enough to complete the construction of the jetties.

Due to lack of maintenance, these jetties deteriorated rapidly. Between 1911 and 1925 they were rebuilt again. A crane mounted on tracks was used in the new construction (the cap method). Sea conditions and limitations of floating equipment forced the construction to be conducted from the jetty crest.

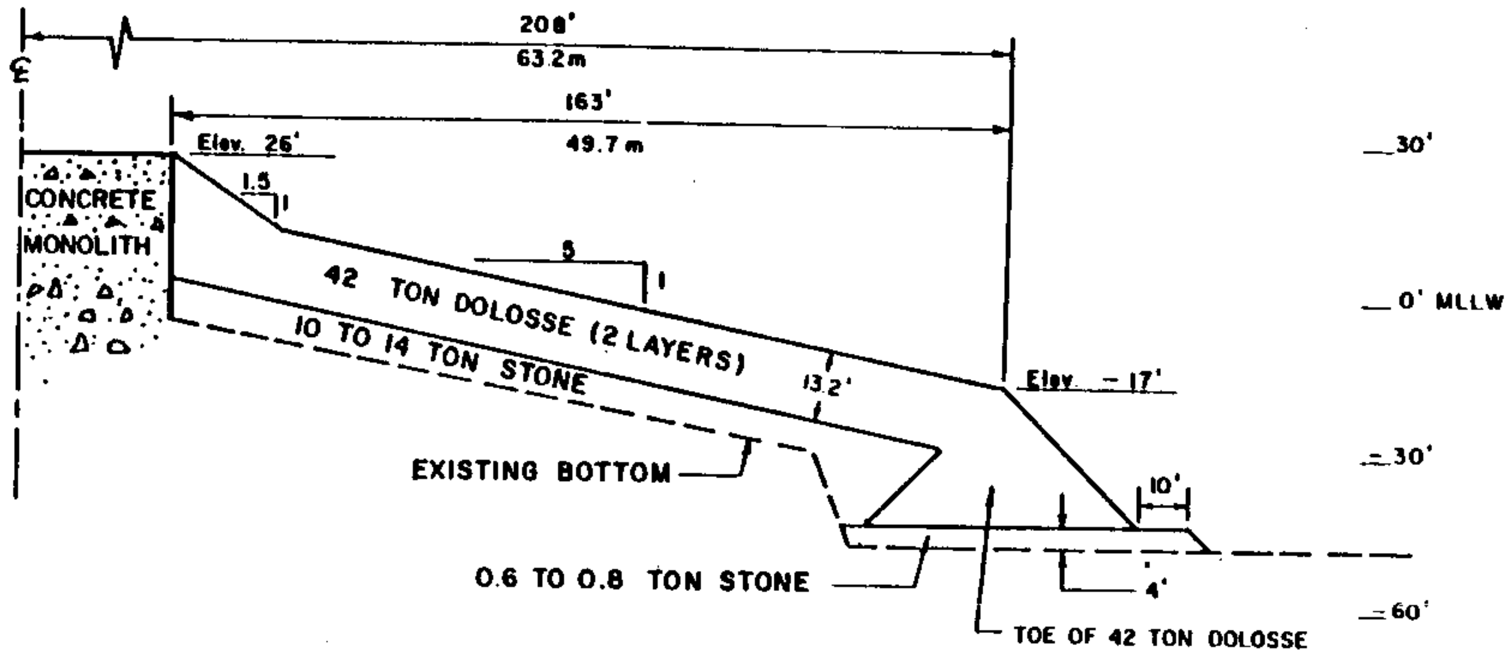
The jetties at Humbolt Bay Harbor have required constant maintenance during the past years.

In 1930 and 1957, more repairs were required. They consisted mainly on placing armor stones over the previous slope which was 1:1.5.

A severe storm deteriorated the north and south jetties during the winter of 1957-1958. Several experts in this field credit the damage to the light weight of armor stone used, because of the limitations on the equipment used at that time. The main repairs consisted of heavier blocks and concrete capping. The concrete was reinforced by bars and track rail.

Another storm washed away most of the armor blocks during the winter of 1964-1965. More damage occurred in 1970 and destroyed the heads of the jetties. The repairs were conducted by a crane (model VC 4600 Manitowoc-Ringer). A Few breakages have occurred since that time, but according to Magoon, Sloan and Shimizu (1976), the structural integrity of the jetties is not endangered.

A cross section of the jetty built in 1926 is shown in figure (2.15). It also shows the construction method used at that time.



TYPICAL CROSS SECTION

SEAWARD HEAD, SOUTH JETTY
HUMBOLDT BAY
CALIFORNIA, U.S.A.

Figure 2.14

Figure 18. Jetty construction.

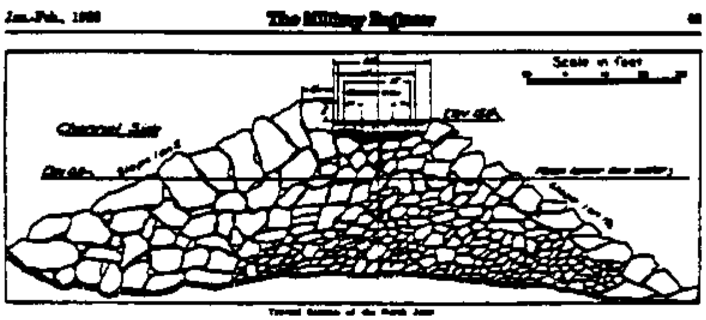


Figure 2.15: Cross Section Of Humboldt Jetties, California And Method Of Construction (Magoon, Sloan, Shimizu, 1976)

CRESCENT CITY HARBOUR, CALIFORNIA

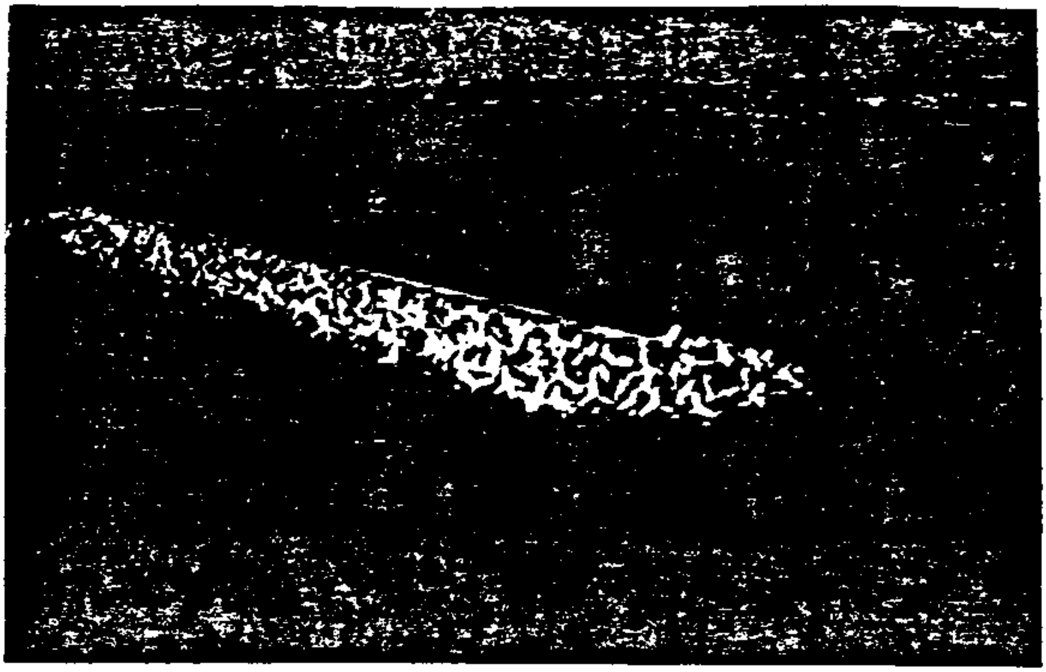
The rubble mound breakwater protects Crescent City Harbour. Its length is 1,447m and the slopes vary from 2.5 to 1 through 4 on 1. Five hundred feet of the breakwater was extensively damaged during the storm of the winter of 1956-1957. Repairs of the breakwater were completed in June of 1957 by replacing the missing layers with 25 tons tetrapods.

In February, 1960 another storm occurred and damaged the breakwater extensively again. Some of these 25 ton tetrapods were moved more than 100 feet away from the breakwater.

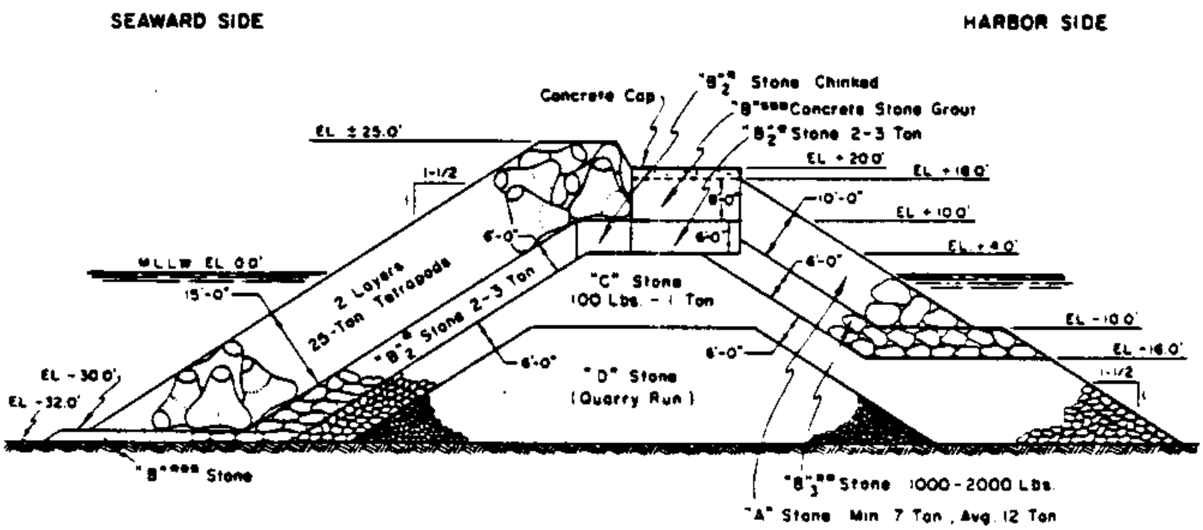
The core of this breakwater is made out of quarry run material, with a slope of 1 : 1.5 and a height of 22 feet. The seabed is at an elevation of -32 feet. This quarry run core is protected by another layer of class C stone with rocks weighing between 100 lb. to one ton. The thickness of this layer is 6 feet.

A filter layer of class B stone weighing 2 to 3 tons protects the core material. The thickness of this layer is also 6 feet. The armor layer, tetrapods on the seaward side and class A stone (average weight 12 tons) on the harbour side, cover the filter layers.

The next figure (2.16) is a photograph of the breakwater and a cross section showing the different sizes of stone used in the construction of the Crescent City breakwater.



Cresent City, California (1957)



* "B"2 – One ton variation to 7 ton max

** "B"3 – 1/2 ton to 1 ton min. – 7 ton max. as available.

*** "B" – 1 ton to 7 tons or to suit depth conditions at seaward toe.

Figure 2.16: Crescent City Breakwater, California
U.S. Army Corps Of Engineers, Shore Protection Manual

FAILURE OF THE CORE

In the last twenty years more harbours have been built in deeper water than previously and often at very exposed sites. The design of rubble mound breakwaters is currently a matter of controversy following the failure of some of these recent structures. Most of the attention is directed to the failure of the outer armor layer and neglecting the other components of the breakwater. The failure of the core could be a major factor in the eventual destruction of the breakwater or it could lead to severe wave energy transmission to the harbour side.

Eugene Harlow (1980) reported that "Fine material from the core of the Sines breakwater has been raised up near the face of the capwall. This implies relative motion between core and capwall." This was evident from a photograph presented by Zwamborn,(1979) describing the damage to the Sines breakwater. Harlow stated that "Although the photos do not reveal the nature of the motion or which component moved first, the armor being moved down the slope or the core being ejected. Plumes of water and spray were observed during wave attack, spouting upward from joints in the horizontal slab forming part of the capwall. These plumes demonstrate extreme fluid pressures existing in the core material beneath the concrete."

The next two figures demonstrate this phenomena.

Figure (2.17) is a photograph of the high impact of the waves against the breakwater built at Diablo Canyon, California.

Figure (2.18) shows the high pore pressure inside the breakwater,as shown by the plumes of water and air ejected from the relief openings in the breakwater cap.

Figure 2.17: Wave Action On Diablo Canyon, California Breakwater

Figure 2.18: Plumes Of Water And Air Shooting Up Through The Relief Openings Of The Diablo Canyon Breakwater, California

Hannoura and McCorquodale (1979) presented a paper in the Civil Engineering In The Oceans IV Conference discussing the environmental wave forces on rubble mound breakwaters. This paper focused on the effect of air entrainment on the hydraulic conductivity of the flow media and the results of the instantaneous pressure distribution due to the wave impact. It determined also the inertia coefficient of the crushed rock.

Even though Hannoura and McCorquodale's paper was directed to the front slope of the rubble mound breakwater in order to determine the stability of the armour units, the experiment could be extended to cover the effect of pore pressure inside the core and the resulting consequences on the breakwater as a total system.

Harlow(1978) also stated that "Repeated impacts and large sudden pore pressures in the boundary layers of the core will produce shearing forces along inclined planes and small shifts in particle position with each impact. These shifts will produce increasing forces in the legs of armor units as they try to dowel the slope, tending to break any that are securely locked into the lower planes. As the core looses its capacity to support the armor, sloughing downward and seaward, a space must appear at the interface with the capwall, opening a gap into which successive wave impacts can, like a hydraulic ram, split the two components still further apart."

Sollitt and Cross (1972) concluded that for a rubble mound breakwater:

1. The transmission coefficient decreases with decreasing wave length, breakwater porosity and permeability and increasing wave height and breakwater width.

2. The reflection coefficient decreases with decreasing breakwater width and wave length and increasing porosity and permeability. Correlation between experiment and theory is best when the incident wave height exceeds the particle diameter of the medium.

A suggestion, has been made by Harlow, that an existing computer program for analyzing the seepage pressures in dams could be utilized in the study of the pore pressures in the core of rubble mound breakwaters. He also recommended a model study for the determination of the angle of internal friction, permeability of the core, the direction and magnitude of the seepage pressures and the slope stability for various breaking wave patterns.

Half Moon Bay breakwater lost most of the class C small core material due to the pounding of waves on the outer slopes during storms. This phenomenon occurred because the gradation of core rock and filter layers that were placed during construction was not properly selected. The results of the loss of this material transformed the breakwater into a permeable structure transmitting wave energy in the harbour side. Needless to say, this could lead to the eventual collapse of the remainder of the breakwater due to the lack of support from the core.

CHAPTER III

BREAKWATER FOUNDATION

The principle function of a breakwater is to provide the protection to make loading and unloading vessels possible without risks of damage to the vessels or berthing facilities resulting from wave and/or current action during the process (BRUNN,1981).

The site selection for a breakwater is dependent on the site selection of the entire port. Per Bruun (1981) reported that "the choice of a particular location for the establishment of a port depends upon many factors including:

1. Land requirements.
2. Requirements to depth and space.
3. Requirements for the protection of the harbor against wave action.
4. Current action .
5. Sedimentation to the extent possible.

Richards, Ling and Gerwick (1976) stated that there are two methods in site selection, as follows:

a. Determined by the Structure:

In this method they summarized the methodology of offshore site investigation as follows:

1. Select potential regions.
2. Compare. Select most desirable regions.
3. Select suitable areas within these regions.
4. Compare and select candidate areas.

5. Select suitable candidate sites within these areas.
6. Compare. Rank the sites.
7. Select primary candidate sites.
8. Investigations 1 through 7 are based on available information. At this stage limited studies on primary candidate sites will be performed.
9. Compare benefits of candidate sites with environmental engineering costs.
10. Select prime site.

b. Determined by the Site

In the second method, the site is provided due to the location of the facilities that the port will service. This site will control the design and construction of these structures.

Treadwell, Ridley and Magoon (1980) reported that "Consideration of regional geothechnical conditions early in the site selection process may reduce costs significantly by identifying sites which might require extensive variations from anticipated designs in order to mitigate potential geologic and seismic hazards. If such a site cannot be avoided, knowledge of the potential hazards should result in more realistic site investigation programs, analysis, designs, and cost estimates."

In either method, complete and detailed hydrographic surveys are necessary. This phase of the investigation should be complete to the extent of supplementing standard survey procedures with bottom samplings, measurements of existing currents, and measurements, if possible of amounts and directions of materials transported by these currents. This data will:

1. Furnish information on foundation conditions.
2. Provide a permanent record of conditions prior to construction.
3. Through use of prior surveys, old charts, maps, etc., indicate any changes the bottom has undergone.

Foundation conditions must be known accurately in order to properly design the breakwater to be constructed at a particular site. The foundation analysis can be made by means of probings, washborings or drillings. In the majority of cases, the rock mound type breakwater is normally selected, unless the bottom is so soft and of such great depth that an excessive amount of rock would be required to reach a satisfactory supporting medium. Magoon, Sloan and Foote (1974) reported " that a few instances have been noted where settlement of the flexible portion of a rubble mound structure with a concrete cap has resulted in a hole or gap under the rigid cap." This has occurred both at the Crescent City Outer Breakwater and the Noyo Harbor north jetty near Fort Bragg, California.

Brunn (1981) reported that "Foundations for marine structures deserve as much if not more careful study than foundations for land structures. Wave forces acting against a rubble structure have been found to attack the natural bottom and the structure foundation even at depths usually thought to be little affected by such forces. A rubble structure may be protected from settlement resulting from leaching, piping, undermining or scour, by a bedding layer or blanket. Blanket mattress protection is explained in more detail at the end of this chapter. Experience indicates that using a bedding layer to protect foundations of rubble mound structures from undermining is advisable except where:

A. The depths of water are greater than twice the maximum wave height.

B. The anticipated current velocities are smaller than those necessary to move the average size of foundation material, and the material has sufficient strength to prevent settlement of the individual pieces of stone.

C. The foundation is hard, durable material, such as bed rock.

When large stones are placed directly on a sand foundation at depths insufficient to avoid wave and current action on the bottom (as in the surf zone) and if an adequate bedding layer is not provided, the rubble will settle into the sand until it reaches the depth below which the sand will not be disturbed by the currents. Actually, this phenomenon is one of local liquefaction of the sand, allowing the rock to settle while the sand flows out from under it.

The planning phase of bathymetric survey will involve an evaluation of the existing data available from navigation and specific purpose charts. "Boat sheets" usually provide sounding data in much greater abundance than the published navigation chart, which is prepared from the boat sheets made during the actual bathymetric surveys.

For general survey, side-scan sonar may serve to cover the area rapidly and to identify natural or artifact obstacles. Wide-beam echo sounders may be satisfactory for general bathymetry. For these surveys, conventional navigation using existing navigation networks may be adequate for location control. On the other hand, detailed site surveys may require a degree of accuracy and precision that may not easily be obtainable using common state-of-the-arts methods. In these cases, precise depth determination may necessitate the use of narrow-beam echo sounders and/or deep-towed transducers; location control often will require deploying a special navigation network.

Ships may utilize accurate electronic aids for navigation, bottom-founded acoustic beacons, or other methods of location control. Echo sounders or pressure sensors deployed by submersibles may be effective for accurate depth determination in limited areas. With the use of inertial guidance and transponders on the sea floor, it has become possible to make very accurate and detailled surveys. However, these methods are very costly, and cannot be used in the surf zone.

Identification of surface soil materials adequate for some siting purposes may be obtained from nautical chart bottom notations, from atlases published by hydrographic or oceanographic agencies, and from the quality of seafloor traces appearing on echograms.

Sampling methods can be approximately divided according to penetration distances below the seafloor. Surface sampling, to a depth of a few meters, can be undertaken by grab-type samplers or short cores from a stationary vessel. Free-fall type short corers or grab-samples can be taken from ships underway. Box gravity-type corers often provide high quality short cores. Short cores or grab samples can also be obtained from a submersible or by divers.

Brown (1971) suggested the use of a piston coring device which can retrieve undisturbed samples of the ocean bottom. This device can yield eleven ft. cores in soft clays, six ft. cores in hard-packed sand, and one ft. cores in stiff clays. Needless to say, sampling of loose surficial sediments is a very difficult task as the structure cannot be preserved during sampling. Core penetrometer tests may be used as an indication of the relative density of sands.

Sampling to a depth of about 30 - 50 m is conventionally done using marine-geological types of piston corers. For all piston corers, if the piston is movable relative to the bottom during the drive stroke, the piston corer is dynamically operated. If the piston is fixed with respect to the seafloor during the drive stroke, then the piston cover operates statically. The latter method is preferable.

FOUNDATION FAILURES

Foundation failures can be classified as follows:

1. Shear failures.
2. Migration of sand under wave and current actions.
3. Scour.
4. Settlement
5. Liquefaction.

1. SOIL FAILURE DUE TO ITS OWN INSTABILITY: SHEAR FAILURE OR LANDSLIDE.

Underwater shear failures can be initiated by dynamic forces due to waves, earthquakes, and construction operations. These shear failure,s once initiated can propagate, leading to more extensive damage.

The shear strength characteristics of sands and inorganic silt (unless the soil is exceptionally loose) are calculated by Terzaghi and Peck (1967) as:

$$c = (p - u_w) \tan (\emptyset) = \bar{p} \tan (\emptyset)$$

Where:

c = shear strength

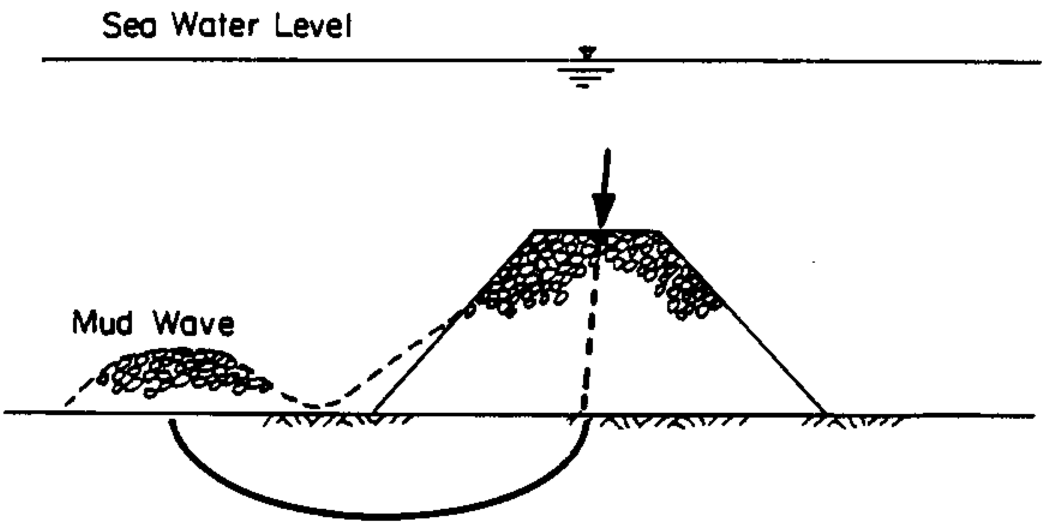
p = confining pressure

u_w = pore water pressure

$\emptyset$ = angle of internal friction

$\bar{p} = p - u_w$ = effective pressure

Figure (3.1) shows a shear failure occurring during the dumping of rock onto the seafloor. Such shear failures are due to both the static load and the dynamic shock, the later diminishing the effective shear strength by causing a sudden rise in pore pressure.



SHEAR FAILURE DUE TO STATIC LOAD AND DYNAMIC SHOCK

Figure 3.1: Shows a shear failure occurring during the dumping of rock onto the seafloor. Such shear failures are due to both the static and the dynamic shock, the latter diminishing the effective shear strength by causing a sudden rise in pore pressure.

2. **MIGRATION OF SAND UNDER WAVE ACTION, SEEPAGE:**

Rocking motion may induce seepage forces (pumping action) in the soil, resulting in tunnelling (internal erosion). Soil compaction or drainage methods may be possible solutions.

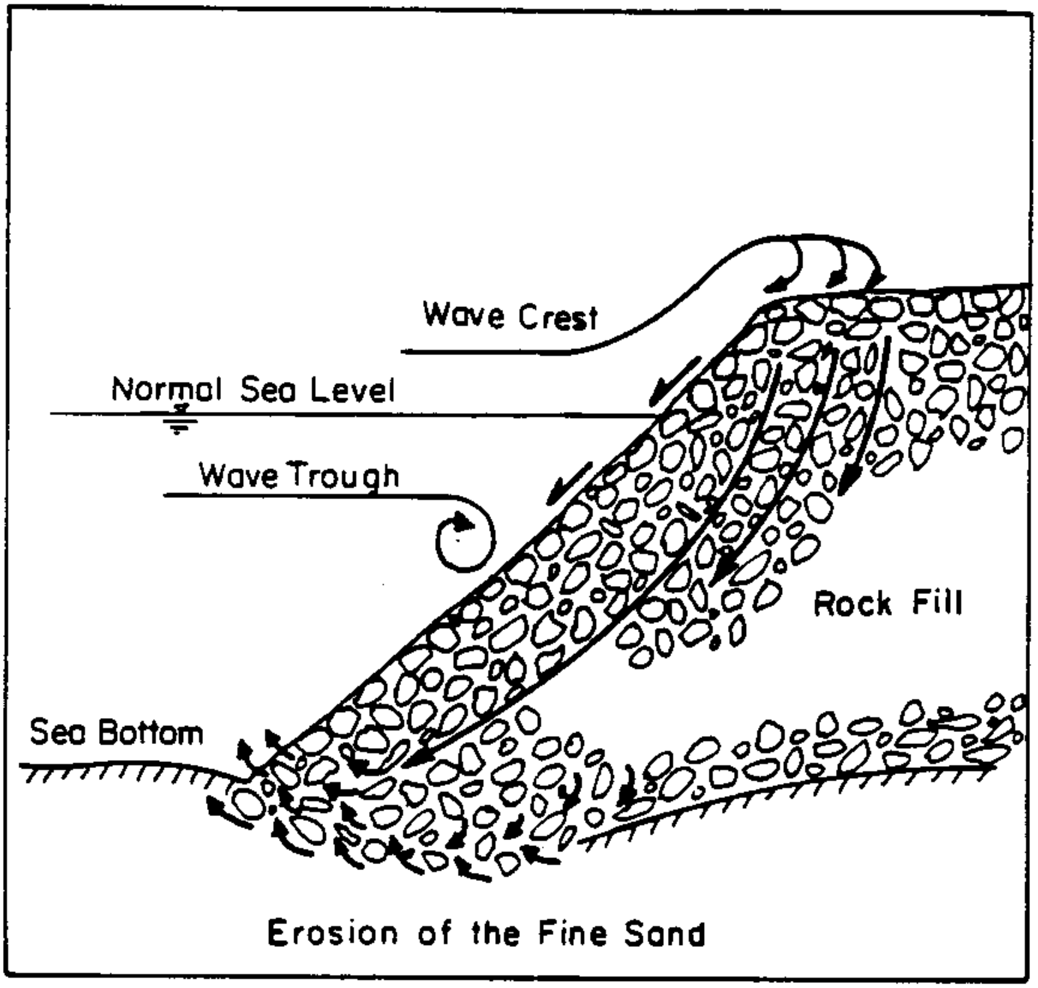
3. **SCOUR**

Massive breakwaters placed on the ocean floor cause local changes in the current pattern in that particular area. This can lead to local scouring, especially if the seafloor is a sandy bottom. Scour holes can then affect the stability of the breakwater and cause failure.

Scour is the removal of seafloor soils caused by currents and waves: the potential for scour is particuarly great in locations with sand and silt at to the seafloor. Scour can result in removing vertical and lateral support for the rock near the periphery, causing undermining of the toe rock.

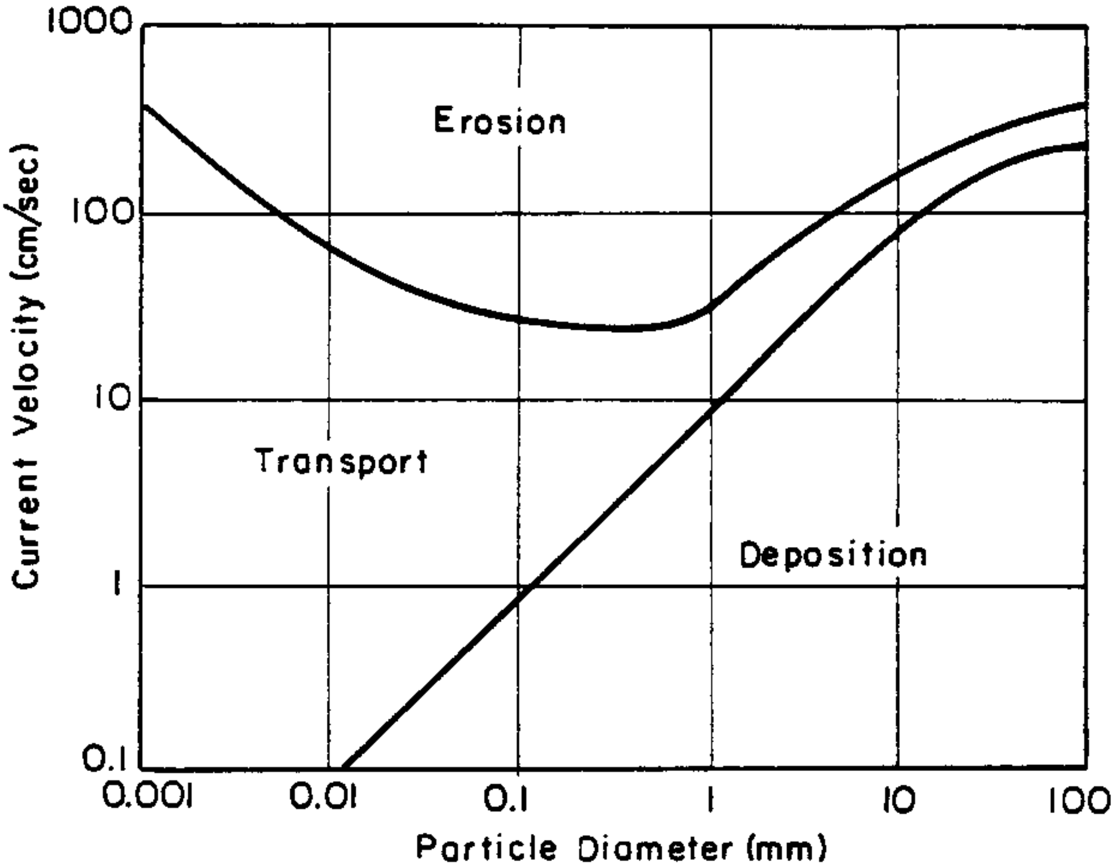
Zeevaert(1957) reported that the spreading of the rock mound because of erosion is a very important phenomenon that may take place because of erosion at the foot of the slope of partially submerged breakwaters on fine cohesionless sediments, such as fine sand. This phenomenon is facilitated as the fine sand becomes loose during spontaneous liquefaction. Let the figure (3.2) be the slope of a partially submerged breakwater. As the crest of the water wave travels along the slope, the water fills up the voids left in the rock mound. During the following trough of the wave, because of the lower water level, the water flows strongly out from the inside of the rock fill, producing a strong erosion in the fine sand at the base. Therefore, the stability of the slope may be lost and spreading may take place. Furthermore, the combined effect of the uplift pressure and the horizontal force produced by the rapid drawdown as the trough of the wave passes along may eventually produce a total spreading of the fill. The shearing strength at the base of the rock mound may be not enough to counteract the internal water force developed in the rock mound.

The grain size of cohesionless soil has a great influence on the amount of scour; the larger the size, the less the scour. The figure (3.3) represents a relationship between flow velocity, particle transport and erosion susceptibility as a function of grain size,(Zeevaert,1957).



EROSION IN THE ROCK MOUND

Figure 3.2
(Zeevaert,1957)



TRANSPORT AND EROSION SUSCEPTIBILITY AS
A FUNCTION OF GRAIN SIZE AND FLOW VELOCITY

Fig 3.3
(Zeevaert, 1957)

4. SETTLEMENT

When a heavy breakwater is constructed on fine sand, silt and clay sediments, large settlements of the fill may occur due to excessive loading of the soils. Settlement may also be the result of exceeding the allowable mechanical properties of shearing strength and compressibility of the soft clay deposit, (Zeevaert,1957).

Excessive settlements and shear failures may be avoided by controlling the rate of deposition (or even carrying it out in stages) and by avoiding dynamic effects, so as to allow the soil to gain in strength before it has to resist the maximum load. Another solution that has proven to be very effective is to widen the base course (reducing the slope) and even placement of berms on either side of the breakater so as to counter the shear forces under the breakwater proper.

5. LIQUEFACTION

Liquefaction is a phenomenon in which a cohesionless soil loses its strength, due to excessive pore water pressure build-up during an earthquake, storm waves, or shock, and acquires a degree of mobility sufficient to permit local or area movements. The essential factors influencing liquefaction potential are:

a. Soil Type (grain size distribution)

Uniformly graded materials are more

susceptible to liquefaction than well-graded material. For uniformly graded material, fine sands tend to liquify more easily than do coarse sands.

b. Relative Density (void ratio). Loose sand (D_R < 70 %) may liquify.

c. Initial Confining Pressure.
The liquefaction potential of a soil is reduced by an increase in confining pressures (overburden pressure).

d. Intensity of Ground Shaking or Impact Shock.

e. Duration of Ground Shaking.

Although factors d. and e. can not be controlled, factor c. can be increased by surcharge, and factor b. can be improved by compaction. In the case of imported fill material, factor a. may be controlled. In many other types of projects (other than breakwaters) various compaction techniques are frequently carried out in order to prevent liquefaction, since they are usually practicable and economical.

Provision of drainage, so that the excess pore water pressure can escape, is another solution. This can be accomplished by placing a filter blanket of small rock on sea floor.

In no case should a breakwater be used where the strength of the bottom is not sufficient to support the load without excessive settlement, as this may eventually result in the failure of the breakwater due to uneven settlement. However, it may be possible to consolidate the soft material by:

1. **Dumping rock** until a stabilized base has been built and allow it to settle for a period of time prior to constructing the upper part of the breakwater. Waiting allows the remolding effect from the dumping of the rock to dissipate and the clay to regain its shear strength.

2. **Placing a wide blanket of rock** first, say three times the width of the breakwater base. The wide blanket will confine the soil under the breakwater proper. Combined with a period of waiting, the critical strength at the edges of the dike will be improved.

3. **Excavate (dredge) a trench** to firm material and refill with rock or other good foundation material.

Dredging could be done by one of the following types of equipment:

CLAMSHELL

In this method a floating crane is employed, with a clamshell bucket (different sizes are available). This operation is very slow and currents may partially refill these trenches before the placing of the firm foundation materials. Therefore, the trench may need to have a supplemental cleaning operation just prior to placement of the rock. See figure (3.4) showing a clamshell dredging operation. Another disadvantage of this operation is that it is very difficult to dredge the seafloor in an even and uniform pattern. On the other hand a clamshell dredge may be operated in moderate sea states and even in the surf. It is not depth limited.

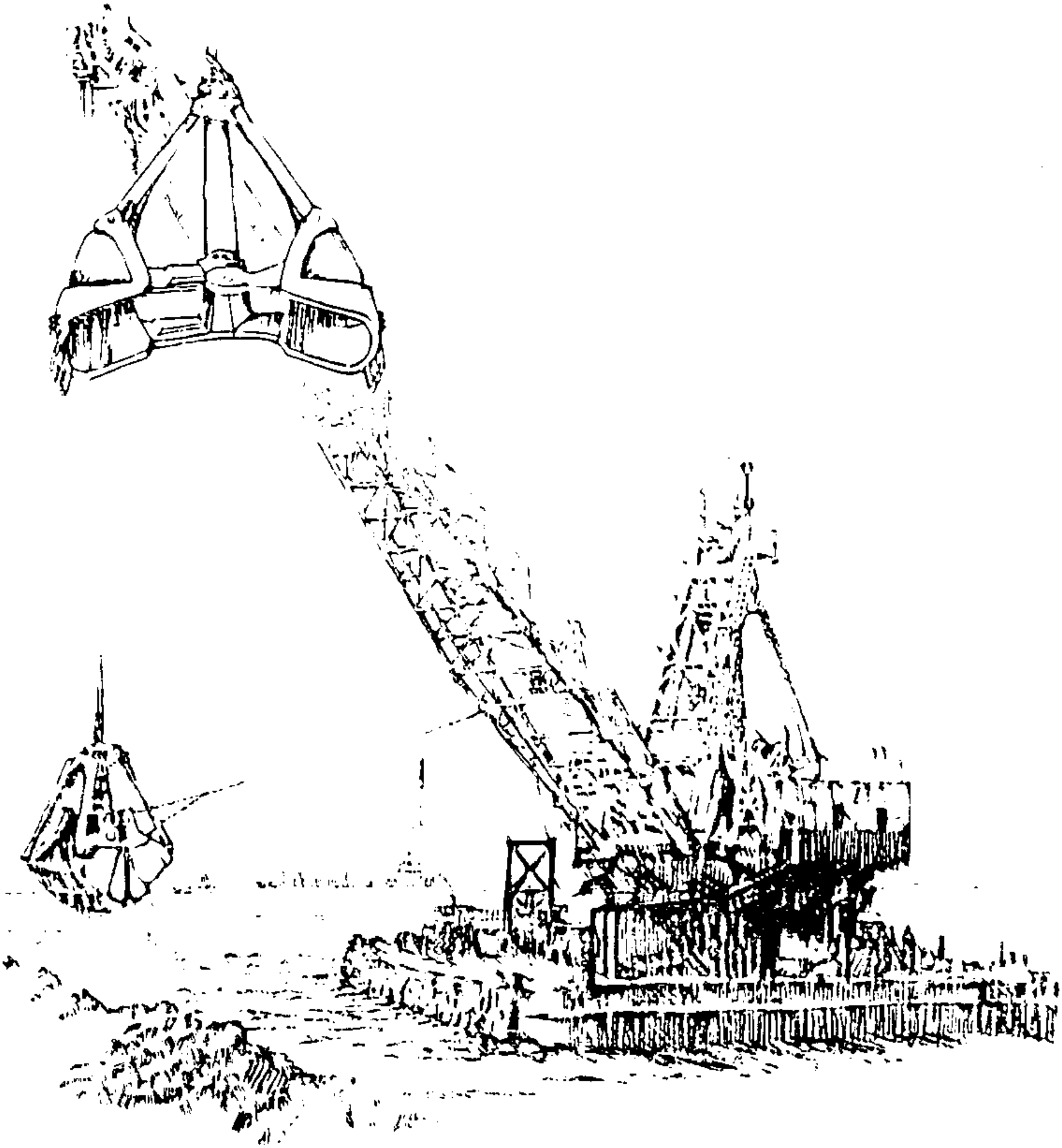


Figure 3.4: Clamshell Dredging Operation

HYDRAULIC DREDGE

There are many different types and sizes of hydraulic dredges. Depending on the nature of the sea, the depth and the work required a specific type of hydraulic dredge may be selected.

Figure (3.5) represents a Cutter Suction Dredge of the Beaver 4000 type.

Figure (3.6) represents a Hydroland Cutter Suction Dredge.

Figure (3.7) represents a dredging operation showing the drege Umpqua "Fisher" loading a dump barge in the San Francisco Bay.

Figure (3.8) represents a "Seal" bucket dredge, Umpqua, dredging in the San Francisco Harbour.

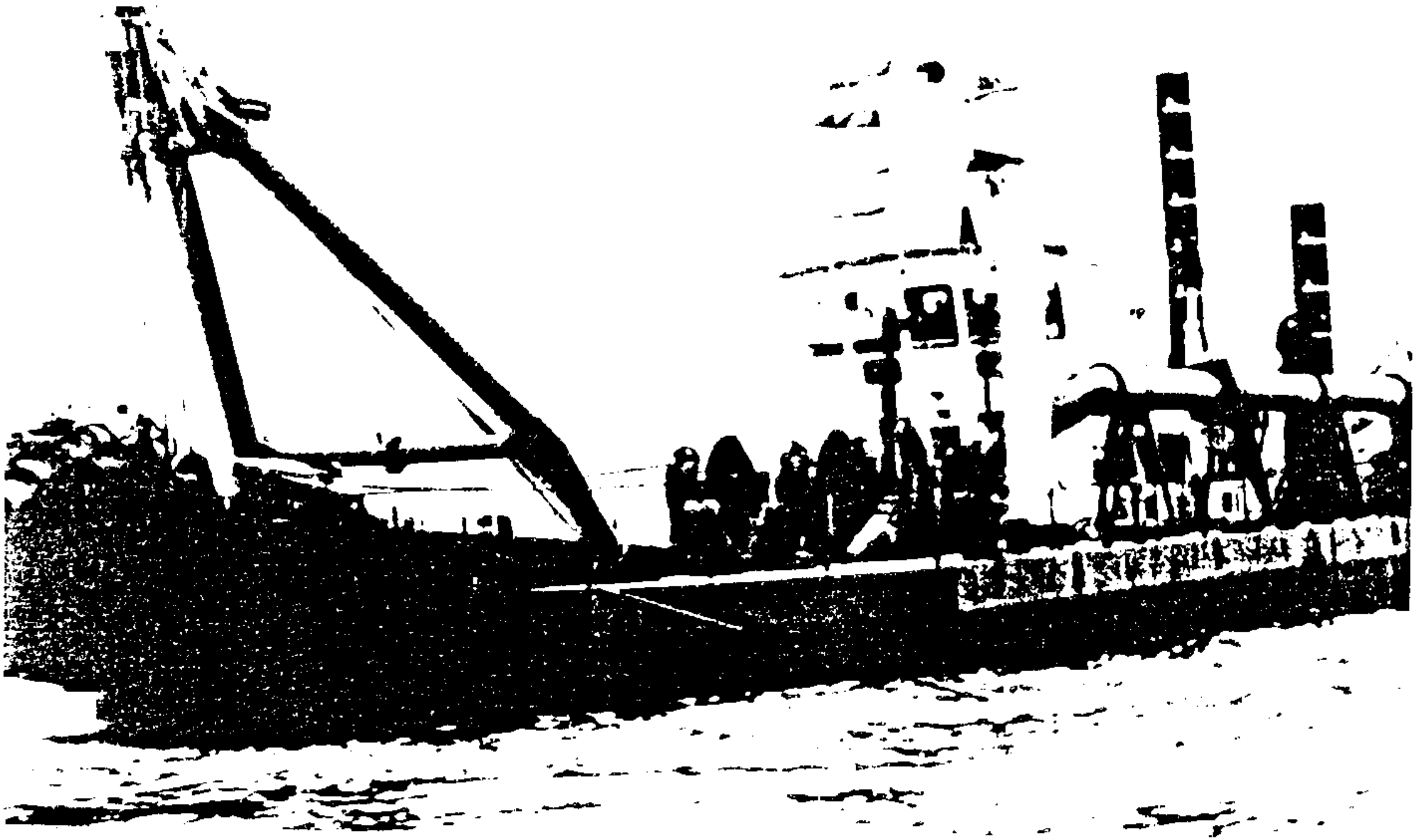


Figure 3.5: Cutter Suction Dredge of the Beaver 4000 type.

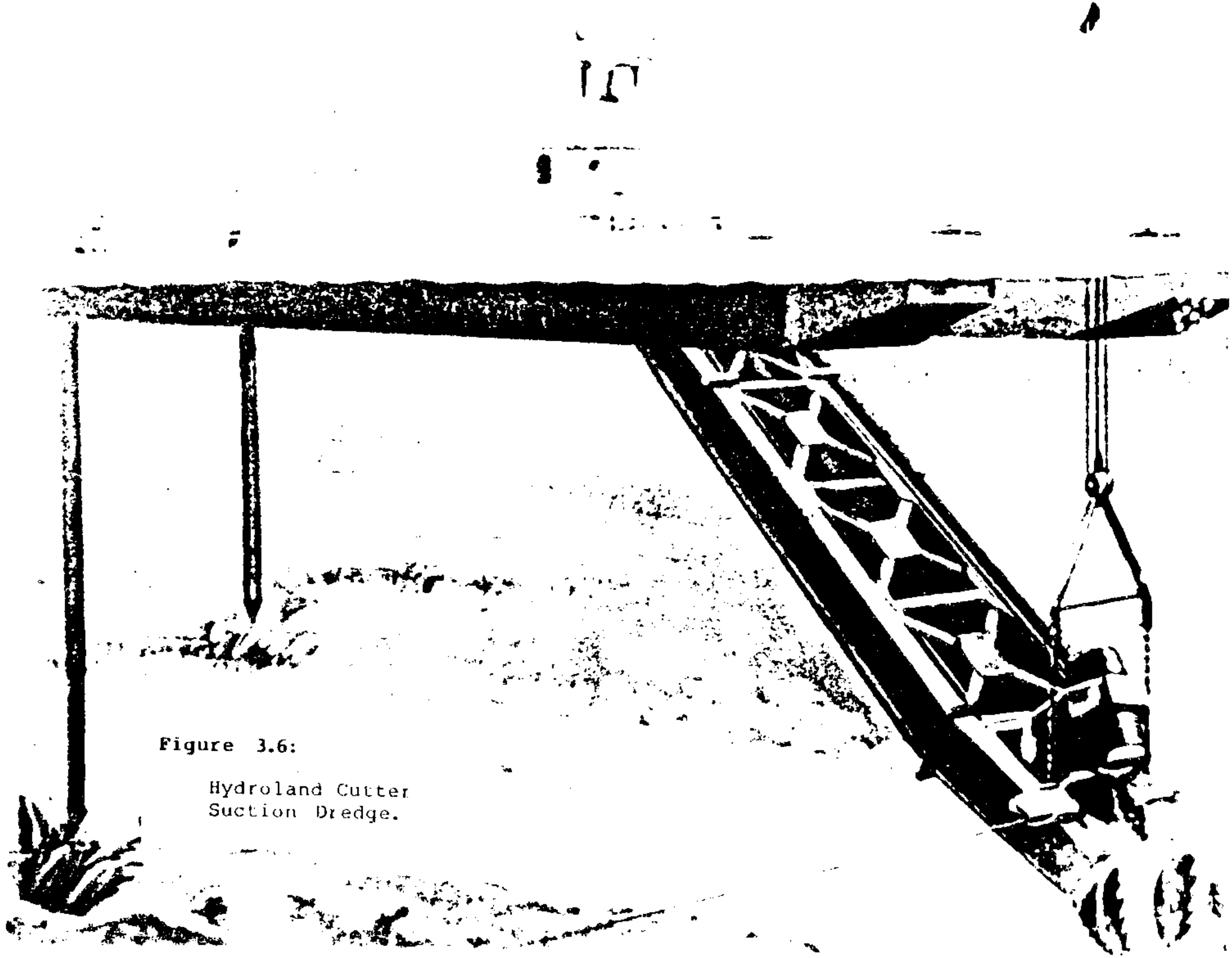


Figure 3.6:

Hydroland Cutter
Suction Dredge.

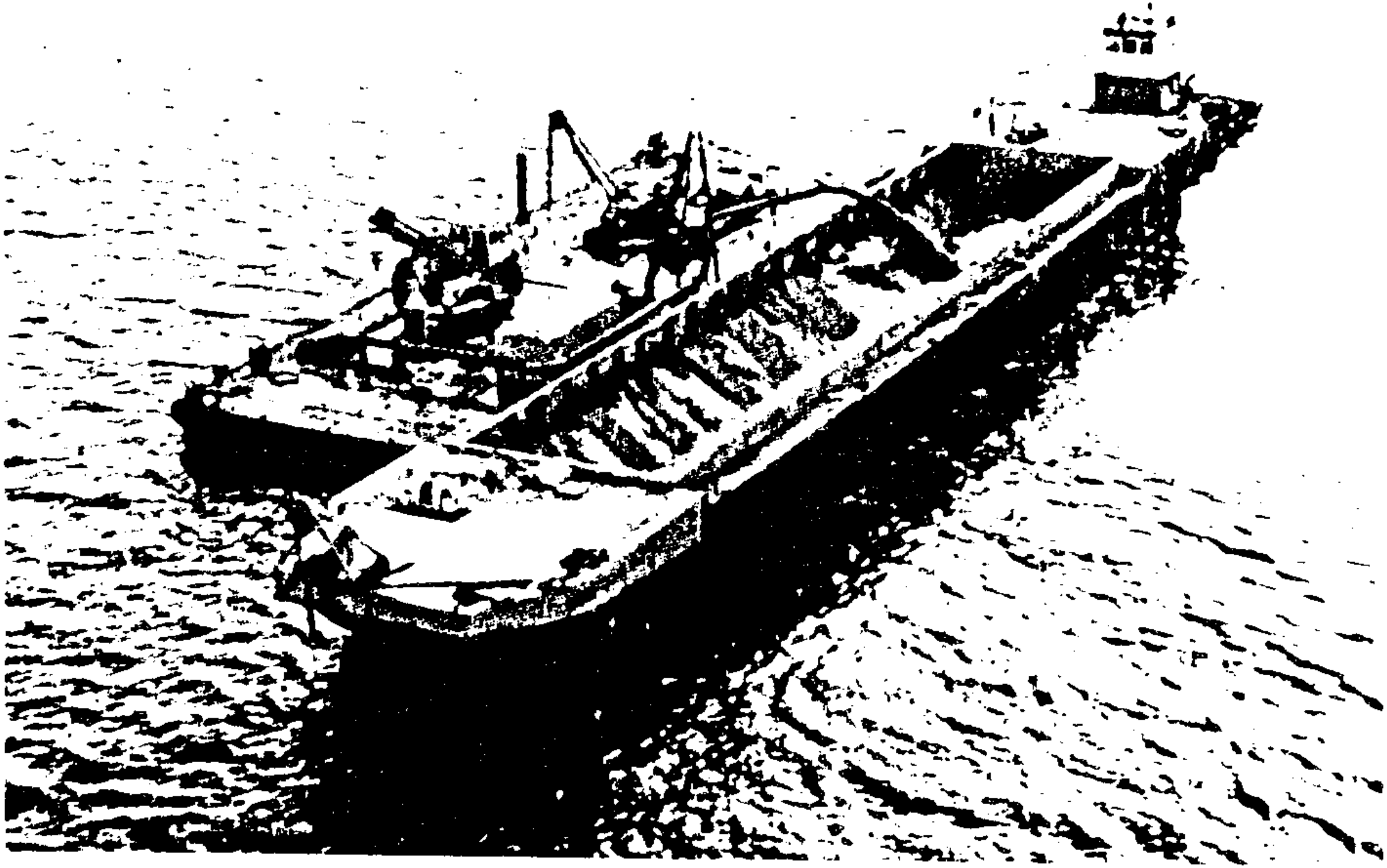


Figure 3.7:
A dredging operation showing the dredger Umpqua "Fisher" loading a dump barge in the San Francisco Bay.

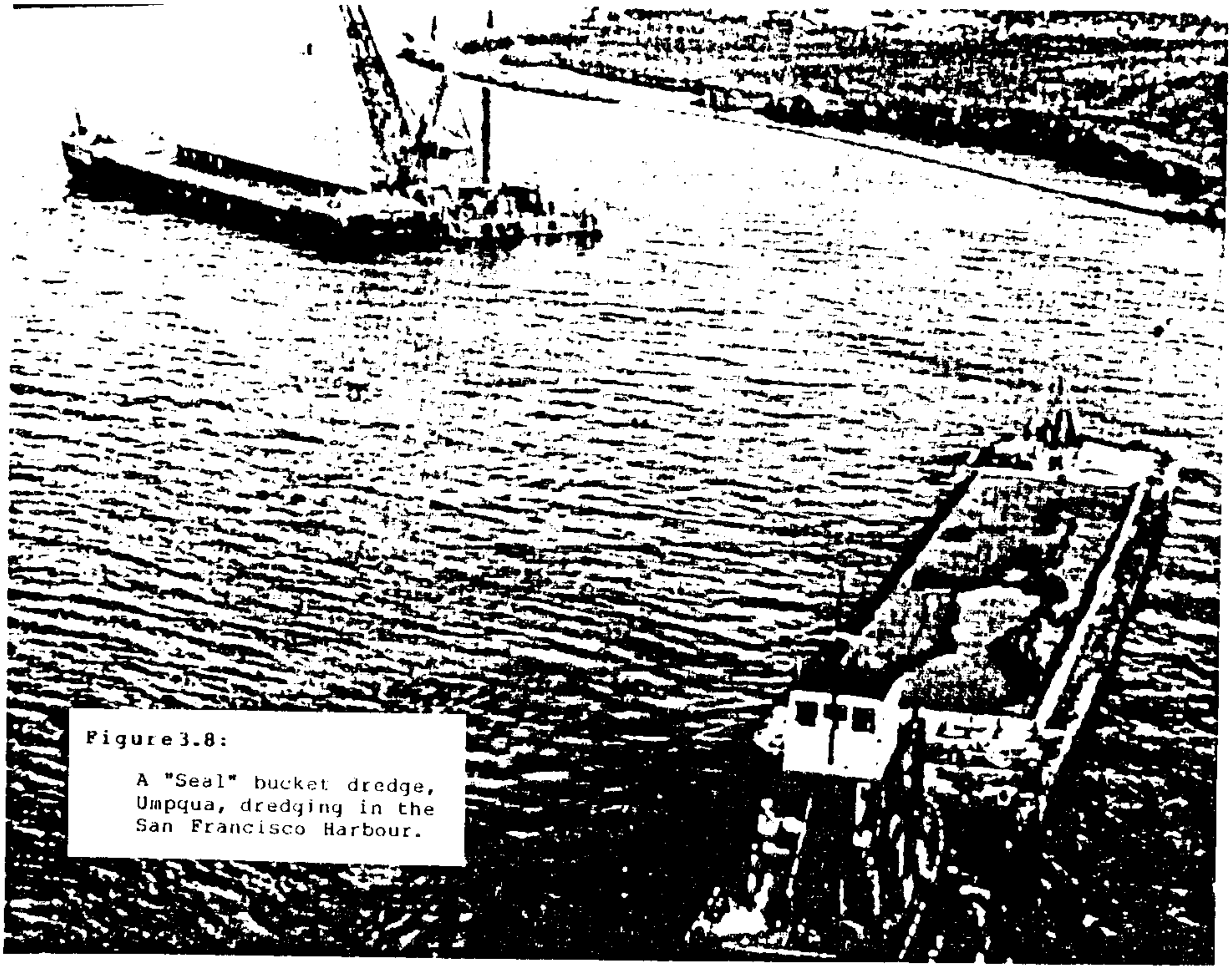


Figure 3.8:

A "Seal" bucket dredge, Umpqua, dredging in the San Francisco Harbour.

DIFFERENT SOLUTIONS FOR SOIL IMPROVEMENT TECHNIQUES TO STRENGTHEN OFFSHORE FOUNDATIONS

Soil improvement techniques can be classified as follows:

1. Vibro-compaction and vibro-displacement compaction.
   A. Blasting
   B. Heavy duty penetrating vibrators (Terraprobe, Vibroflotation (see figure 3.9), Toyomenka, Dutch Vibratory Probe, (Tomen) etc..)
   C. Vibratory rollers (surface compaction)
   D. Compaction pile
   E. Heavy tamping (dynamic consolidation) also known as the "Menard Method"

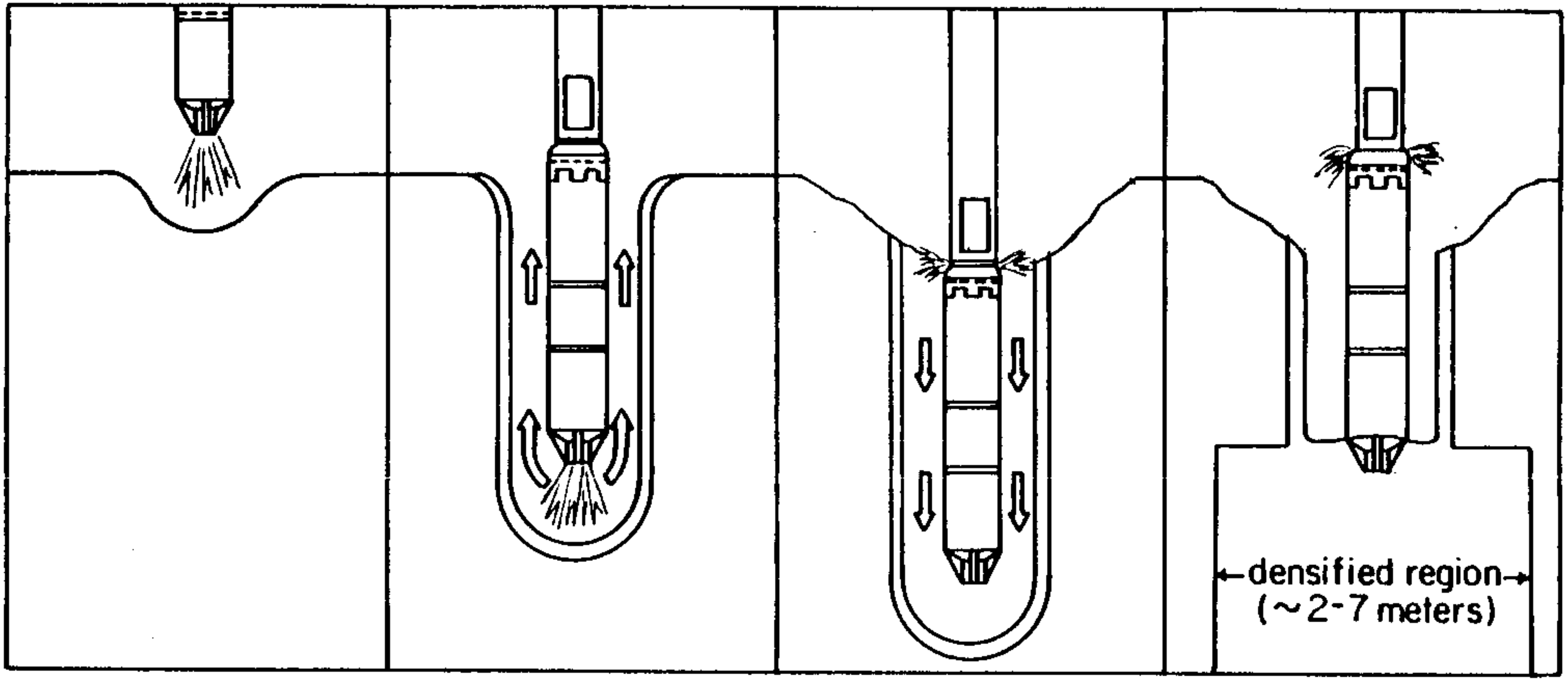
2. Grouting and Injection
   A. Particulate grout
   B. Chemical grout

3. Preloading (surcharging). This may be accompanied by wicks or sand drains to facilitate drainage of the foundation soils.

4. Displacement. Rapid and continous dumping can be used to create shear failures in the weak surficial soils, so as to push a mud

wave ahead of the rubble fill.

The technique applicable to a specific project is determined by a number of factors, especially the grain size of the soil to be stabilized and the sea conditions at the site.



Vibroflot is positioned and lower water jet turned on

Vibration and water jet induce liquefaction as vibroflot penetrates the ground under its own weight

Lower jet is turned off when vibroflot reaches desired depth. Upper jets turned on to aid backfilling and eliminate arching of backfill

Compaction takes place as vibroflot head vibrates.The device is raised to the surface in one-foot-per-minute lifts

Figure 3.9: Schematic of Densification by Vibroflotation (After Vibroflotation Co. Brochure)

1. VIBRO-COMPACTION AND VIBRO-DISPLACEMENT COMPACTION

The basic concept of vibro-compaction is that vibration and shock waves in loose saturated cohesionless soils cause liquifaction followed by densification and settlement accompanying the dissipation of excess pore water pressures. The effectiveness of this method is greatly reduced if there are more than 20% of silt or 5% clay size material, primarily because the hydraulic conductivity of such material is too low to allow rapid drainage following liquifaction.

Vibro-displacement compaction is similar to vibrocompaction except that the vibrations are supplemented by active displacement of the soil. **Appendix B** shows one such system used to consolidate land fills: this could also be applied in shallow water.

Blasting transmits shock waves through the soil which momentarily liquefy it. Then as it drains and reconsolidates, it acheives a higher densification and strength.

Vibratory rollers act directly on the seabed and, just as in dry land applications, result in densification of the surficial soils.

Terraprobe and compaction pile give the vibration to the probe or pile first, and then the vibration is transfered to the soil. The stress wave excited at the pile top may move through a pile without loss, because the frictional resistance of water is negligible. Therefore, vibration energy may be transmitted to the soil almost completely.

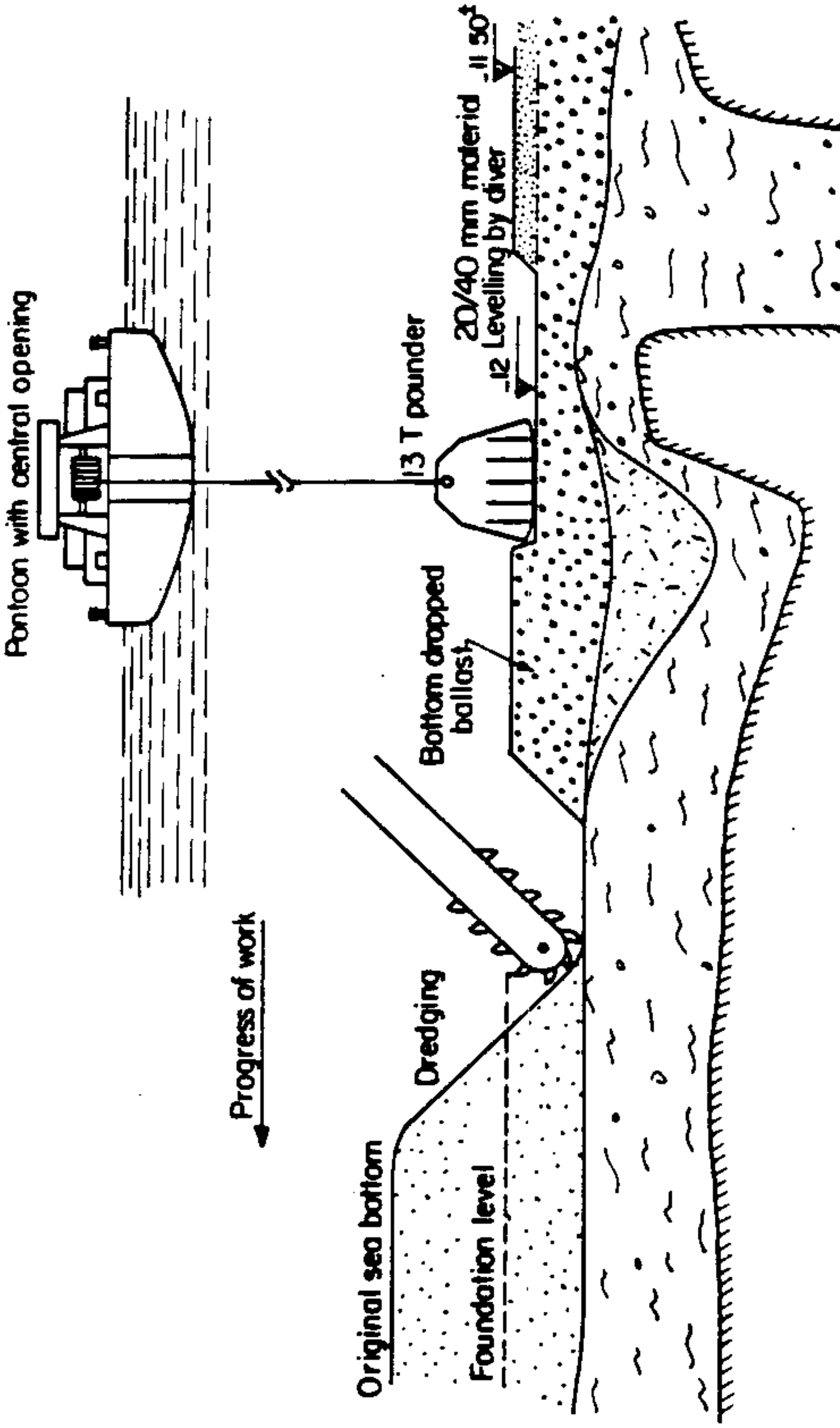
The Japanese have carried out underwater compaction using a sea bottom crawler tractor: this however is of questionable applicability to the typical breakwater.

DYNAMIC CONSOLIDATION

"Dynamic Consolidation" is the term used to denote the method of compaction in which a large weight is repeatedly dropped, so as to cause local shock waves. A sudden rise in pore pressure is followed by liquefaction, then reconsolidation in a more dense state.

The efficiency of dynamic consolidation will be reduced in underwater operation due to the viscous damping force of water.

Dynamic consolidation may work well in consolidation of underwater fills up to 4-5 m thick. Dynamic consolidation can be applied to a great range of various soil types. It can be carried out in moderate sea conditions. Figure (3.11) shows a schematic of the operation procedures of such a system.



DYNAMIC CONSOLIDATION

Figure 3.10

(from Menard Booklet)

BLASTING

Mitchell (1976) states that, although blasting is one of the most economical stabilization methods, with low costs, it suffers the disadvantages of non-uniformity, potential adverse effects on adjacent structures, and dangers associated with the use of explosives in populated areas, or near to other constructed facilities. In the water, fish will be killed.

Most of these methods for soil consolidation can be applied to offshore breakwater foundations, depending on the availability of equipment and the type of soil to be compacted, and the sea conditions at the site.

Figure (3.11) shows the applicable grain size ranges used in different stabilization methods. The ranges vary from clay to gravel depending on the nature of the sea floor material.

Figure (3.12) displays a schematic of the different methods of compaction mechanics described earlier.

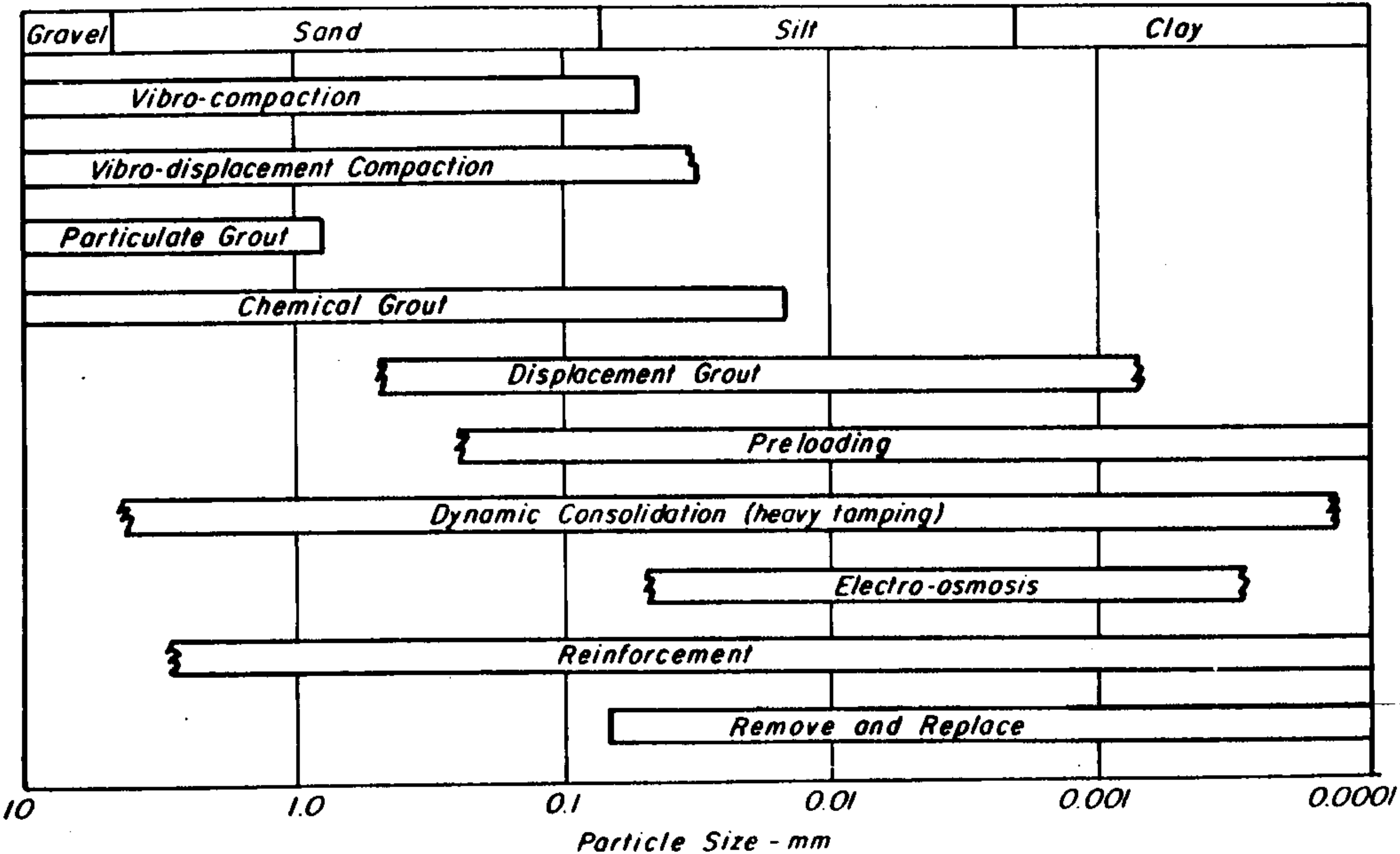


Figure 3.11: Applicable Grain Size Ranges For Different Stabilization Methods

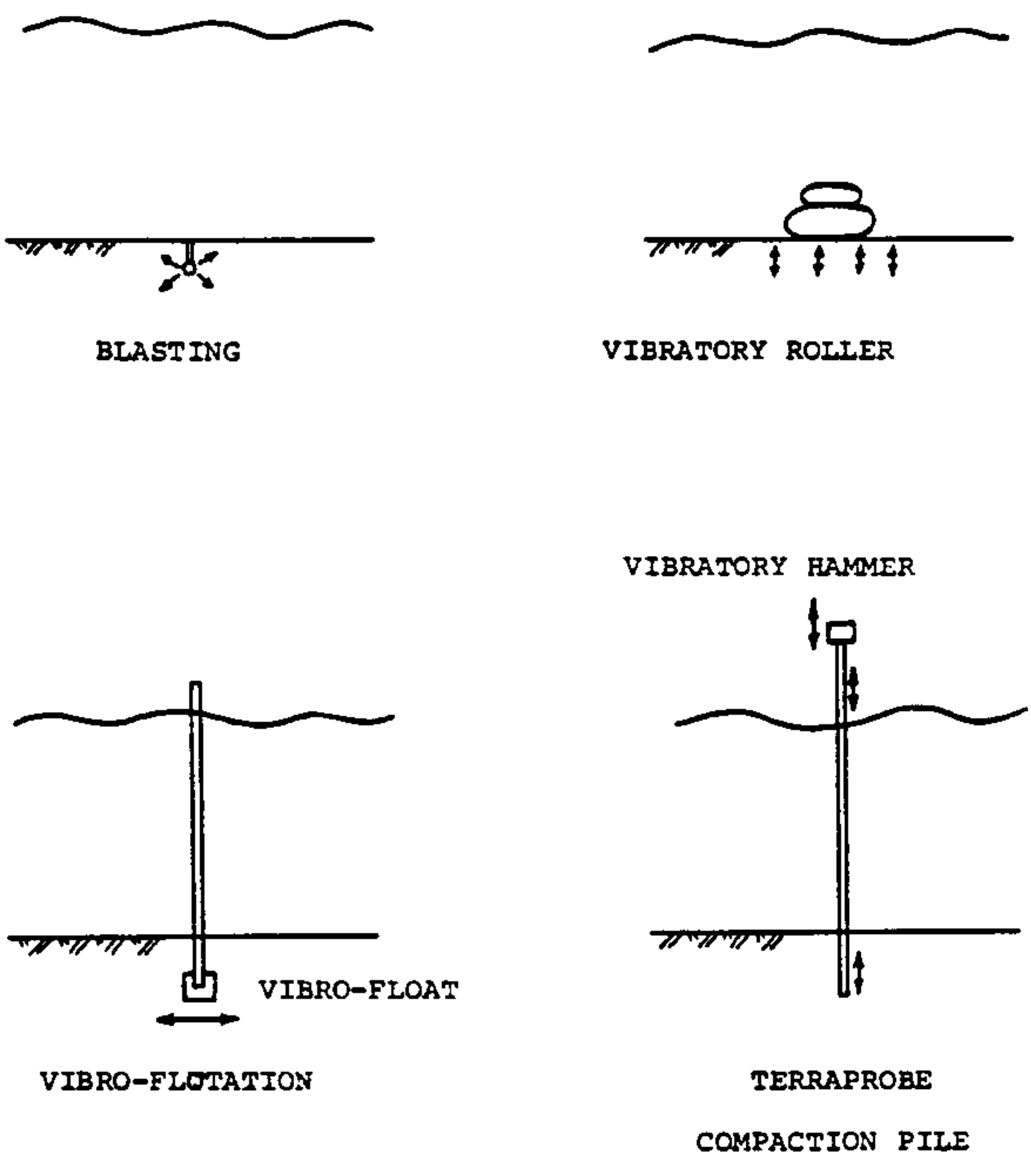


Figure 3.12: A schematic of the different methods of compaction mechanics described earlier.

The following tables (III.1,2,3,4,5,) taken from the Army Corps of Engineers design manual (1976) represent the different methods of soil stabilization for foundations of structures. They detail the relationship between these methods and the soil conditions, effective depth and the area in which they can be most effective. They also note the special material and equipment required for each method, as well as their advantages and limitations.

STABILIZATION OF SOI)R FOUNDATIONS OF STRUCTURES

	METHOD	PRINCIPLE	MOST SUITABLE SOIL CONDITIONS/ TYPES	MAXIMUM EFFECTIVE TREATMENT DEPTH	ECONOMICAL SIZE OF TREATED AREA	SPECIAL MATERIALS REQUIRED	SPECIAL EQUIPMENT REQUIRED	PROPERTIES OF TREATED MATERIAL	SPECIAL ADVANTAGES AND LIMITATIONS	RELATIVE COSTS (1976)
VIBRO-COMPACTION	BLASTING	Shock waves and vibrations cause liquefaction, displacement, remolding	Saturated, clean sands; partly saturated sands and silts after flooding	20 m	Small areas can be treated economically	Explosives, backfill to plug drill holes	Jetting or drilling machine	Can obtain relative densities to 70-80%; may get variable density	Rapid, inexpensive, can treat small areas; variable properties, no improvement near surface, dangerous	Low ($0.25-$1.00 per m^3)
	TERRAPROBE	Densification by vibration; liquefaction induced settlement under overburden	Saturated or dry clean sand	20 m (Ineffective above 4 m depth)	>1500 m^2	None	Vibratory pile driver and 750 mm dia open steel pipe	Can obtain Relative Densities of 80% or more	Rapid, simple, good underwater; soft underlayers may damp vibrations, difficult to penetrate, stiff overlayers, not good in partly saturated soils	Moderate $1.50-$3.25 per m^2, $2.00/$m^2$ average
	VIBRATORY ROLLERS	Densification by vibration; liquefaction induced settlement under roller weight	Cohesionless soils	2-3 m	Any size	None	Vibratory roller	Can obtain very high relative densities; upper few decimeters not densified	Best method for thin layers or lifts	Low
VIBRO-DISPLACEMENT COMPACTION	COMPACTION PILES	Densification by displacement of pile volume and by vibration during driving	Loose sandy soils; partly saturated clayey soils; loess	20 m	Small to moderate	Pile material (usually sand or soil plus cement mixture)	Pile driver	Can obtain high densities, good uniformity	Useful in soils with fines, uniform compaction, easy to check results; slow, limited improvement in upper 1 - 2 m	High
	HEAVY TAMPING (Dynamic Consolidation)	Repeated application of high intensity impacts at surface	Cohesionless soils best, other types can also be improved	15-20 m	>5000 m^2	None	Tamper of 10-40 tons high capacity crane	Can obtain high relative densities, reasonable uniformity	Simple, rapid, suitable for soils with fines; usable above and below water; requires control, must be away from existing structures	<Vibro-flotation
	VIBROFLOTATION	Densification by vibration and compaction of backfill material	Cohesionless soils with less than 20% fines	30 m	>1500 m^2	Granular backfill	Vibroflot, crane	Can obtain high relative densities, good uniformity	Useful in saturated and partly saturated soils, uniformity	$10.00-$25.00/m >$1.00/$m^3$ May be about half compaction piles or concrete piles

Table III.1

Prepared for Corps of Engineers Design Manual (1970).

METHOD	PRINCIPLE	MOST SUITABLE SOIL CONDITIONS/ TYPES	MAXIMUM EFFECTIVE TREATMENT DEPTH	ECONOMICAL SIZE OF TREATED AREA	SPECIAL MATERIALS REQUIRED	SPECIAL EQUIPMENT REQUIRED	PROPERTIES OF TREATED MATERIAL	SPECIAL ADVANTAGES AND LIMITATIONS	RELATIVE COSTS (1976)
PARTICULATE GROUTING	Penetration grouting-fill soil pores with soil, cement, and/or clay	Medium to coarse sand and gravel	Unlimited	Small	Grout, water	Mixers, tanks, pumps, hoses	Impervious, high strength with cement grout, eliminate liquefaction danger	Low cost grouts, high strength; limited to coarse-grained soils, hard to evaluate	Lowest of the grout systems
CHEMICAL GROUTING	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate	Medium silts and coarser	Unlimited	Small	Grout, water	Mixers, tanks, pumps, hoses	Impervious, low to high strength, eliminate liquefaction danger	Low viscosity, controllable gel time, good water shut-off; high cost, hard to evaluate	High to very high \$30/m^3 min-\$80/m^3 typical
PRESSURE INJECTED LIME	Lime slurry injected to shallow depths under high pressure	Expansive clays	Unlimited, but 2 - 3 m usual	Small	Lime, water, surfactant	Slurry tanks, agitators, injection	Lime encapsulated zones formed by channels resulting from cracks, root holes, hydraulic fracture	Rapid and economical treatment for foundation soils under light structures	\$2.50 to \$3.00/m^2 of ground surface area
DISPLACEMENT GROUT	Highly viscous grout acts as radial hydraulic jack when pumped in under high pressure	Soft, fine-grained soils; foundation soils with large voids or cavities	Unlimited, but a few m usual	Small	Soil, cement, water	Batching equipment, high pressure pumps, hoses	Grout bulbs within compressed soil matrix	Good for correction of differential settlements, filling large voids; careful control required	Low material, high injection
ELECTROKINETIC INJECTION	Stabilizing chemicals moved into soil by electro-osmosis	Saturated silts, silty clays	Unknown	Small	Chemical stabilizer	DC power supply, anodes, cathodes	Increased strength, reduced compressibility	Existing soil and structures not subjected to high pressures; no good in soil with high conductivity	Expensive

Prepared for Corps of Engineers Design Manual (1976).

Table III.2

	METHOD	PRINCIPLE	MOST SUITABLE SOIL CONDITIONS/ TYPES	MAXIMUM EFFECTIVE TREATMENT DEPTH	ECONOMICAL SIZE OF TREATED AREA	SPECIAL MATERIALS REQUIRED	SPECIAL EQUIPMENT REQUIRED	PROPERTIES OF TREATED MATERIAL	SPECIAL ADVANTAGES AND LIMITATIONS	RELATIVE COSTS (1976)
PRECOMPRESSION	PRELOADING	Load is applied sufficiently in advance of construction so that compression of soft soils is completed prior to development of the site	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	-----	>1000 m^2	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment; large water tanks or vacuum drainage systems sometimes used; settlement markers, piezometers	Reduced water content and void ratio, increased strength	Easy, theory well developed, uniformity; requires long time (sand drains or wicks can be used to reduce consolidation time)	Low (Moderate if vertical drains are required)
	SURCHARGE FILLS	Fill in excess of that required permanently is applied to achieve a given amount of settlement in a shorter time; excess fill then removed	Normally consolidated soft clays, silts, organic deposits, completed sanitary landfills	-----	>1000 m^2	Earth fill or other material for loading the site; sand or gravel for drainage blanket	Earth moving equipment; settlement markers, piezometers	Reduced water content, void ratio and compressibility; increased strength	Faster than preloading without surcharge, theory well developed; extra material handling; can use sand drains or wicks	Moderate. Sand drains cost \$3.30-\$6.60/m
	DYNAMIC CONSOLIDATION	High energy impacts compress and dissolve gas in pores to give immediate settlement; increased pore pressure gives subsequent drainage	Partly saturated fine grained soils, quaternary clays with 1-4% gas in micro bubbles	20 m	>15000-30000 m^2	None	Tamper of 10-40 tons, high capacity crane	Reduced water content, void ratio and compressibility; increased strength	Faster than preloading, economical on large areas; uncertain mechanism in clays, less uniformity than preloading	<preload fills with sand drains
	ELECTRO-OSMOSIS	DC current causes water flow from anode towards cathode where it is removed	Normally consolidated silts and silty clays	10-20 m	Small	Anodes (usually rebars or aluminum), cathodes (well points or rebars)	DC power supply, wiring, metering systems	Reduced water content and compressibility, increased strength, electrochemical hardening	No fill loading required, can use in confined area, relatively fast; non-uniform properties between electrodes, no good in highly conductive soils	High

Prepared for Corps of Engineers Design Manual (1976).

Table III.3

	METHOD	PRINCIPLE	MOST SUITABLE SOIL CONDITIONS/ TYPES	MAXIMUM EFFECTIVE TREATMENT DEPTH	ECONOMICAL SIZE OF TREATED AREA	SPECIAL MATERIALS REQUIRED	SPECIAL EQUIPMENT REQUIRED	PROPERTIES OF TREATED MATERIAL	SPECIAL ADVANTAGES AND LIMITATIONS	RELATIVE COSTS (1976)
REINFORCEMENT	MIX-IN-PLACE PILES AND WALLS	Lime, cement, or asphalt introduced through rotating auger or special in-place mixer	All soft or loose inorganic soils	>20 m	Small	Cement, lime, asphalt, or chemical stabilizer	Drill rig, rotary cutting and mixing head, additive proportioning equipment	Solidified soil piles or walls of relatively high strength	Uses native soil, reduced lateral support requirements during excavation; difficult quality control	Moderate to high
	STRIPS AND MEMBRANES	Horizontal tensile strips or membranes buried in soil under footings	All	A few m	Small	Metal or plastic strips, polyethylene, polypropylene or polyester fabrics	Excavating, earth handling, and compaction equipment	Increased bearing capacity, reduced deformations	Increased allowable bearing pressure; requires overexcavation for footings	Low to Moderate
	VIBRO-REPLACEMENT STONE COLUMNS	Hole jetted into soft, fine-grained soil and backfilled with densely compacted gravel	Soft clays and alluvial deposits	20 m	>1500 m^2	Gravel or crushed rock backfill	Vibroflot, crane or Vibrocat, water	Increased bearing capacity, reduced settlements	Faster than precompression, avoids dewatering required for remove and replace; limited bearing capacity	Moderate to high ~$30/m; <pile penetration
THERMAL	HEATING	Drying at low temperatures; alteration of clays at intermediate temperatures (400-600°C); fusion at high temperatures (>1000°C)	Fine-grained soils, especially partly saturated clays and silts, loess	15 m	Small	Fuel	Fuel tanks, burners, blowers	Reduced water control, plasticity, water sensitivity; increased strength	Can obtain irreversible improvements in properties; can introduce stabilizers with hot gases	High
	FREEZING	Freeze soft, wet ground to increase its strength and stiffness	All soils	Several m	Small	Refrigerant	Refrigeration system	Increased strength and stiffness; reduced permeability	No good in flowing ground water, temporary	High

Prepared for Corps of Engineers Design Manual (1976).

Table III.4

	METHOD	PRINCIPLE	MOST SUITABLE SOIL CONDITIONS/ TYPES	MAXIMUM EFFECTIVE TREATMENT DEPTH	ECONOMICAL SIZE OF TREATED AREA	SPECIAL MATERIALS REQUIRED	SPECIAL EQUIPMENT REQUIRED	PROPERTIES OF TREATED MATERIAL	SPECIAL ADVANTAGES AND LIMITATIONS	RELATIVE COSTS (1976)
MISCELLANEOUS	REMOVE AND REPLACE (with or without admixtures)	Foundation soil excavated, improved by drying or admixture, and recompacted	Inorganic soils	10 m ?	Small	None, unless admixture stabilizers used	Excavating and compaction equipment, dewatering system	Increased strength and stiffness, reduced compressibility	Uniform, controlled foundation soil when replaced; may require large area dewatering	High
	STRUCTURAL FILLS (with or without admixtures)	Structural fill distributes loads to underlying soft soils	Soft clays or organic soils, marsh deposits	-----	Small	Sand, gravel, flyash, bottom ash, slag, expanded aggregate, clam shell or oyster shell, incinerator ash	Compaction equipment	Soft subgrade protected by structural load-bearing fill	High strength, good load distribution to underlying soft soils	Moderate to high ($12/m^3)

Prepared for Corps of Engineers Design Manual (1976).

Table III.5

STABILIZING THE SEABED FOUNDATION

Experience in many breakwaters placed on seabed sands shows that the surficial layers may locally liquefy during the dumping of the core rock, thus allowing the rock to work down into the sand. Later, under service conditions, the pore pressures may be increased by the dynamic pounding of the waves, leading to local liquifaction, settlement of the breakwater, and migration of sand up into the core. This phenomenon has occured on the Oregon coast and on the East Coast of India.

One solution, as practiced in India, is to place an initial filter course of small rock, blanketing the sea floor. Then the core rock is dumped on this. The Dutch, in carying out their Delta Plan, have developed a number of schemes, based on the principle of laying a filter carpet or mattress on the seafloor sands. This mattress is designed to initially protect the sands from scour and additional loosening by wave action until the core can be placed. Then the mattress functions as a filter, to provide relief of pore pressure without upward migration of the sands.

These practices of soil densification, followed by laying of a filter mattress, are currently being carried out in a very sophisticated manner on the Oosterschelde Storm Surge Barrier in The Netherlands. This mammouth project requires the construciton of an underwater rock dike across the mouth of the Eastern Scheldt. Later, caissons will be set on the dike.

For the Oosterschelde Storm Surge Barrier, a trench was first dredged in the unconsolidated sands, then filled back with small rock. They then employed vibrating probes to penetrate the loose underlying sands and compact them by vibration. Figures (3.13) and (3.14) show the barge mounted plant and the sequence of operation.

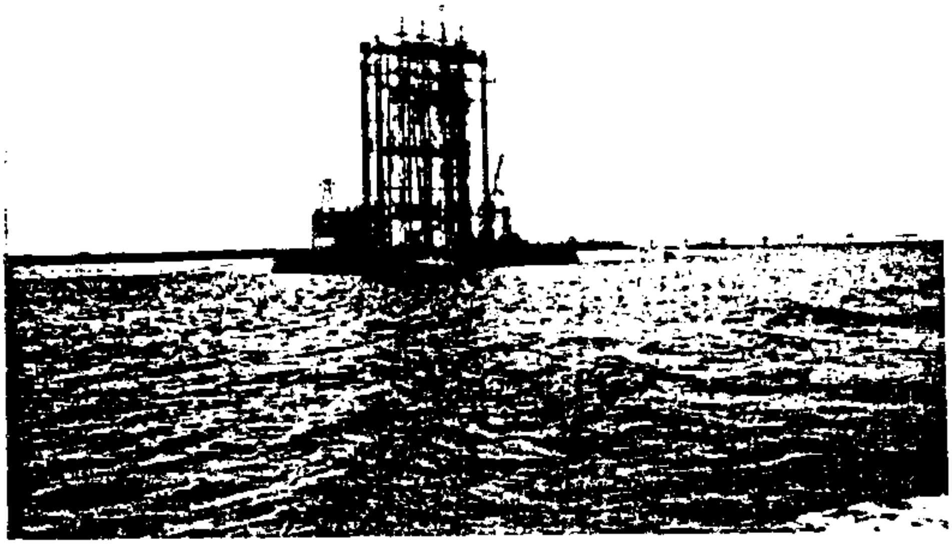


Figure 3.13: Deep Compacting Barge Mytilus Used In The Eastern Scheldt Seafloor.

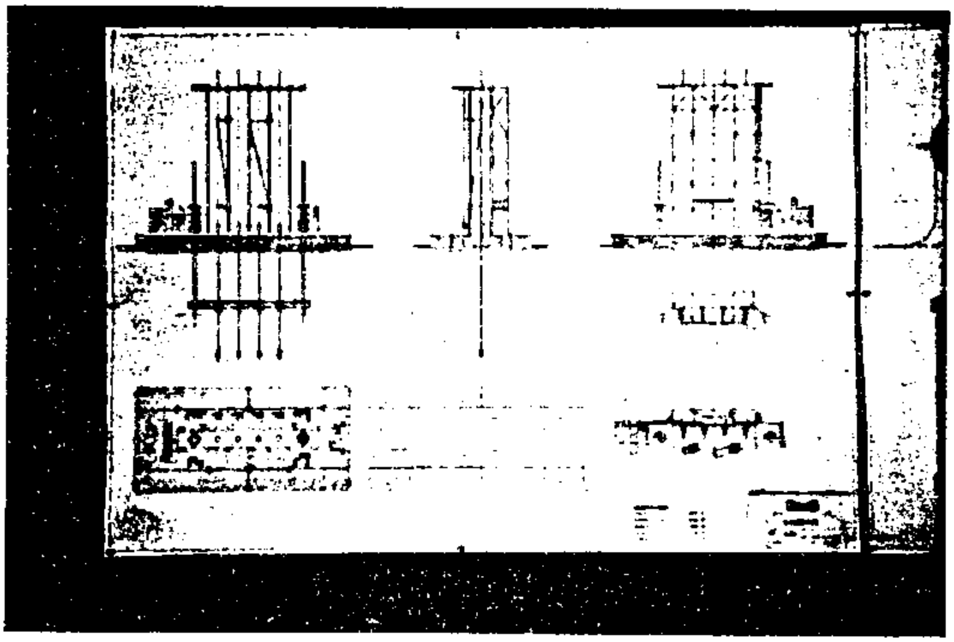


Figure 3.14: Schematic Showing The Deep Compacting Barge At Work

The second step involved the placement of a protective layer of filter fabric mattresses weighted with concrete blocks. Then the construction of submerged dike followed. **Appendix A** illustrates a brochure describing the filter assembly plant used in the Oosterschelde Storm Surge Barrier.

The next section presents different methods of placement of mattresses or filter cloth on the bottom of the sea to prevent migration of sand or small soil particles from foundation. These filter mattresses act to relieve the pore pressures generated in the sands below during the dumping of the breakwater core and subsequently by the dynamic impact of breaking waves. They also serve to protect the seabed from erosion and disturbance during the initial periods of breakwater construction.

FASCINE MATTRESS WITH POLYPROPYLENE FILTRE CLOTH

The mattress consists of a polypropylene filter cloth (weight 700 g/sm) with fixed lashings (polypropylene ropes), one layer of willow (thickness 100 mm. when loose) and two frameworks of wieps in the form of squares at 1 m. centres.

The machine-made wieps have a circumference of not less than 350 mm. throughout.

The total thickness of these mattresses is about 400 mm. (including 0,400 t./sqm stone).

These mattresses protect a sandy bottom far better then fascine mattresses without filtre cloth.

They are practiced by the Deltaworks in Holland.

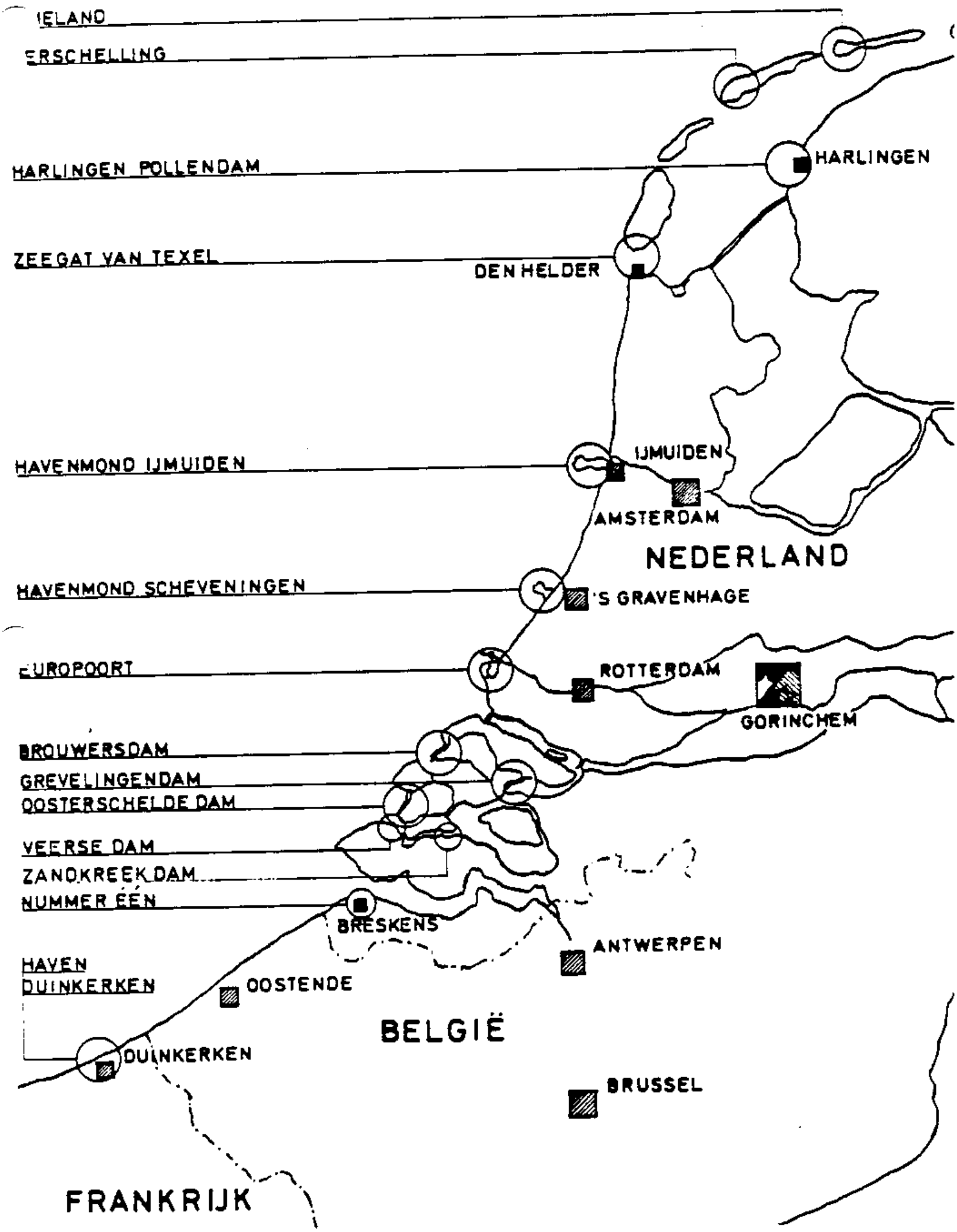
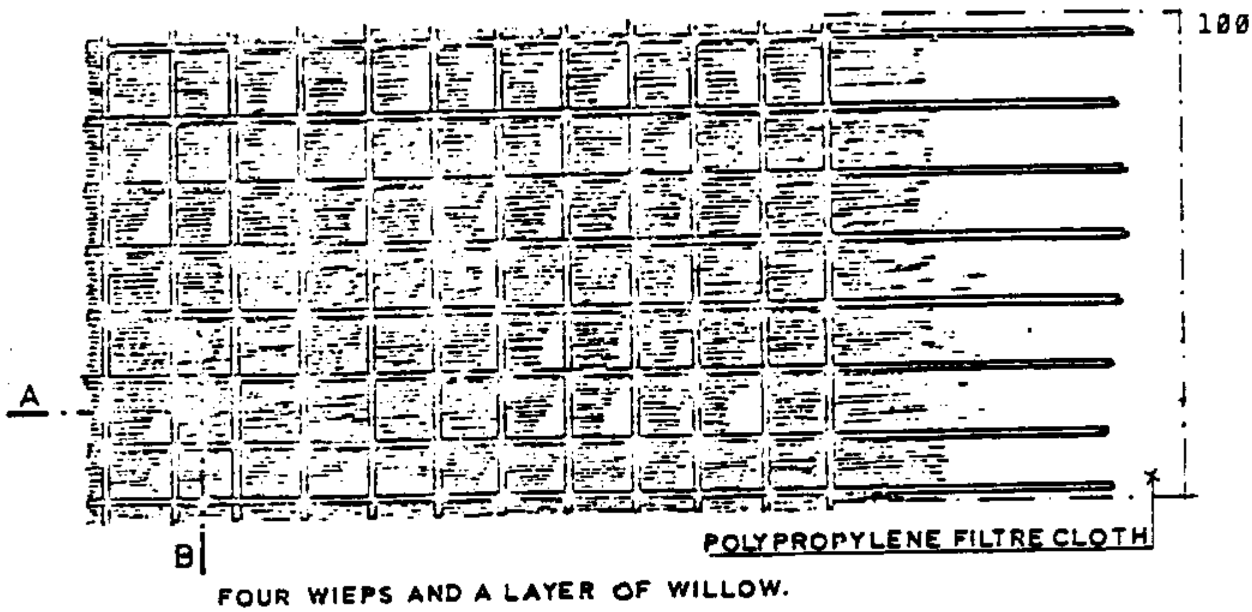
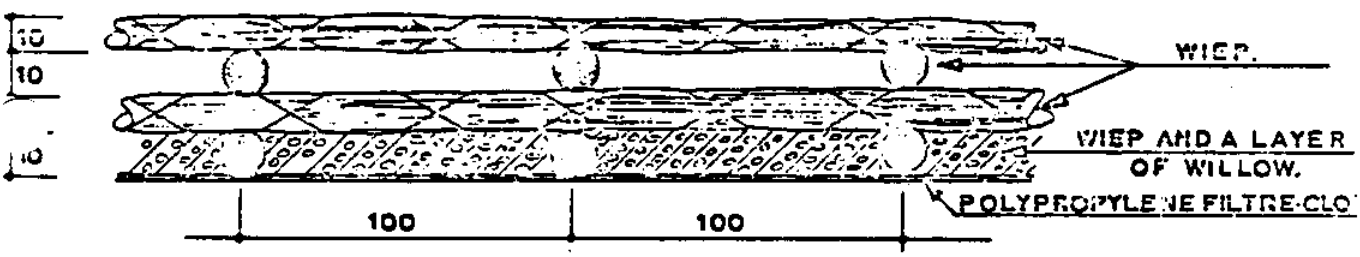


Figure3.15: Map of the Delta Works and Seacoast, Dikes of The Netherlands

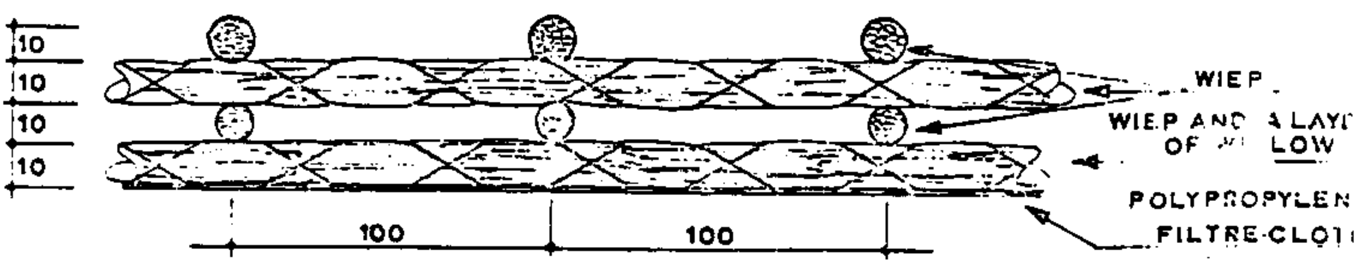
Figure 3.16: Placement of Filter Fabric Mattresses on Seabed



FASCINE MATTRESS



SECTION. A.



SECTION. B.

Figure 3.17: Fascine Mattress With Polypropylene Filter Cloth

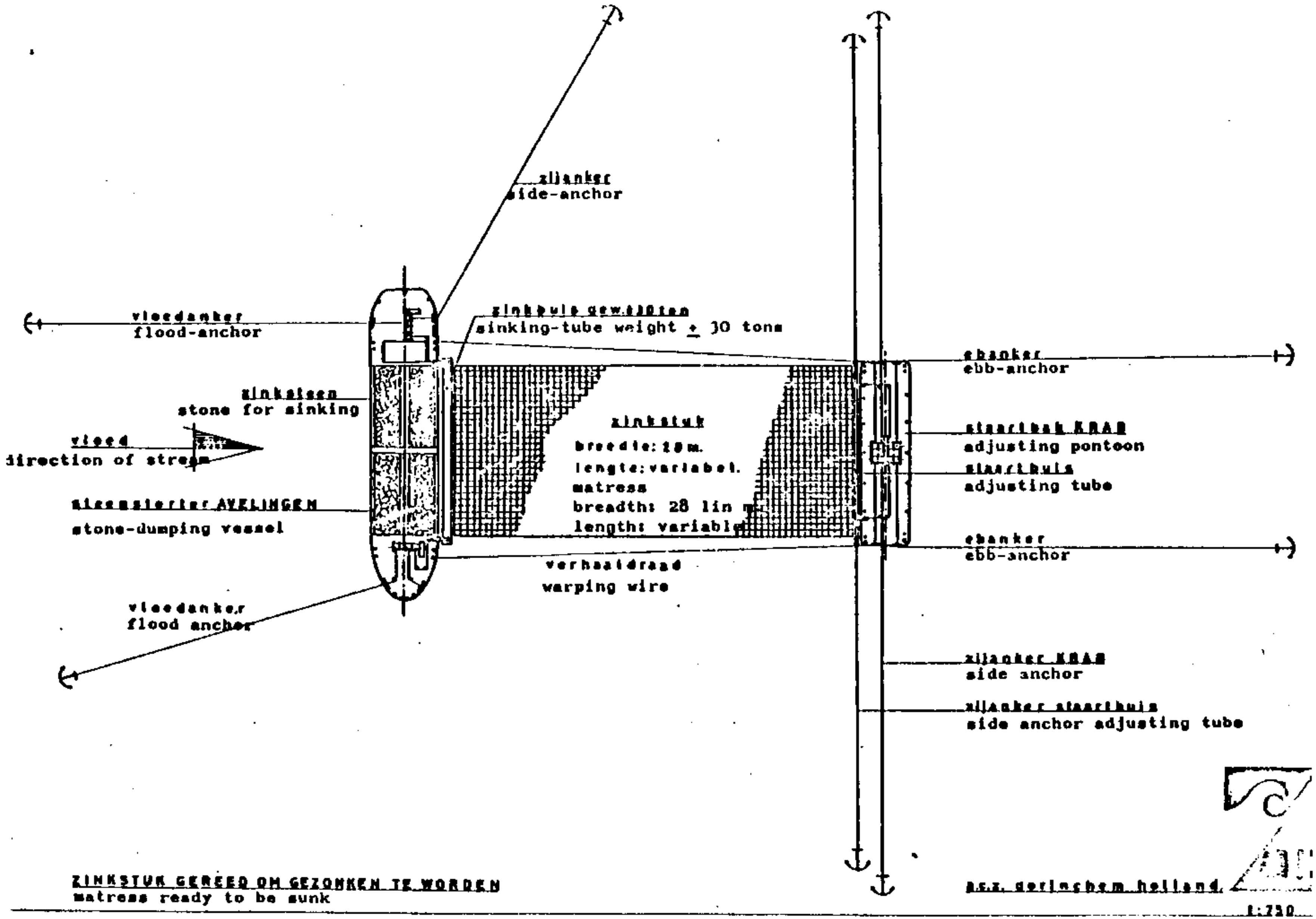


Figure 3.18: Mattress Installation Scheme

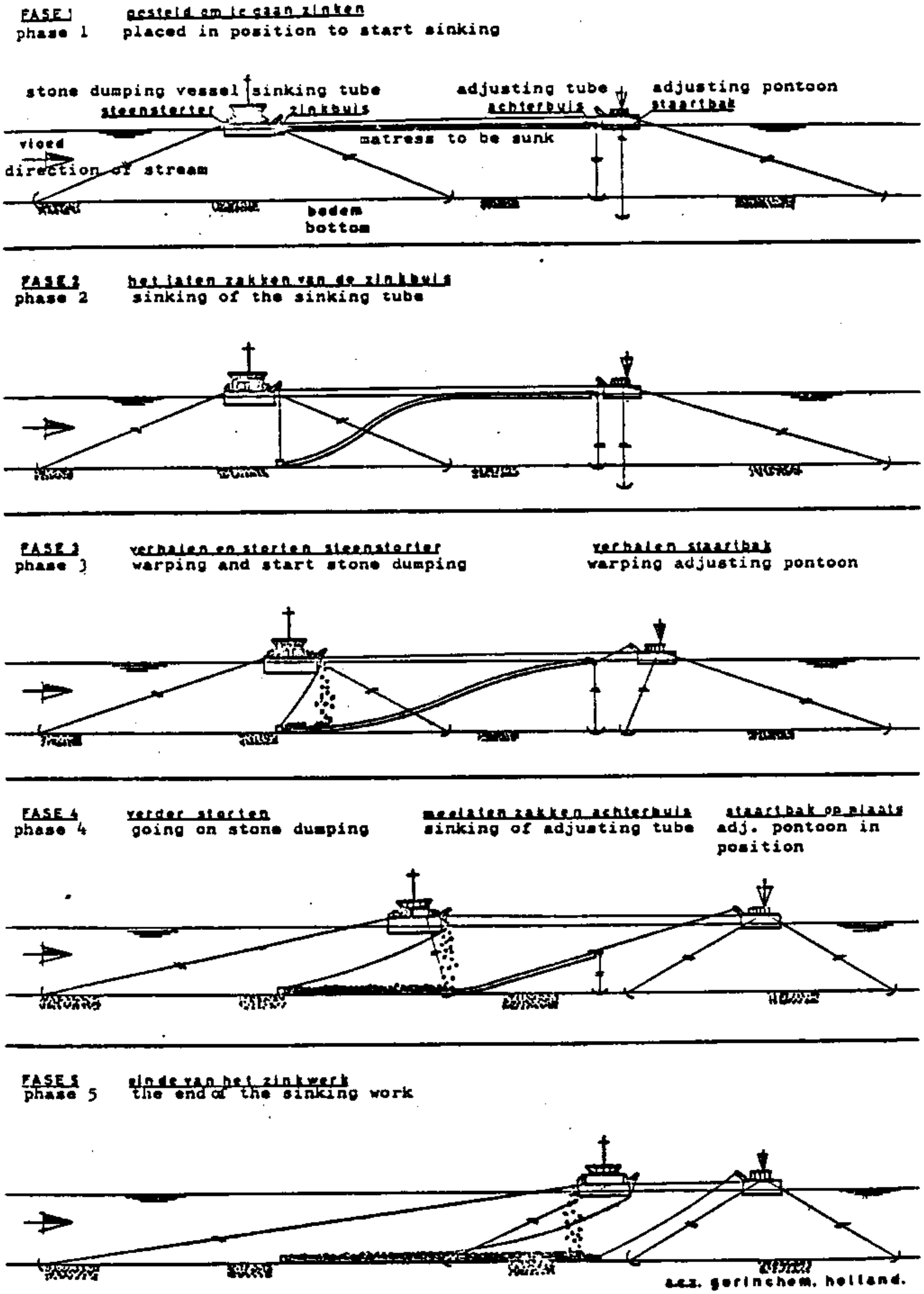


Figure 3.19: Laying of Filter Fabric Mattress on Seabed: Installation Scheme

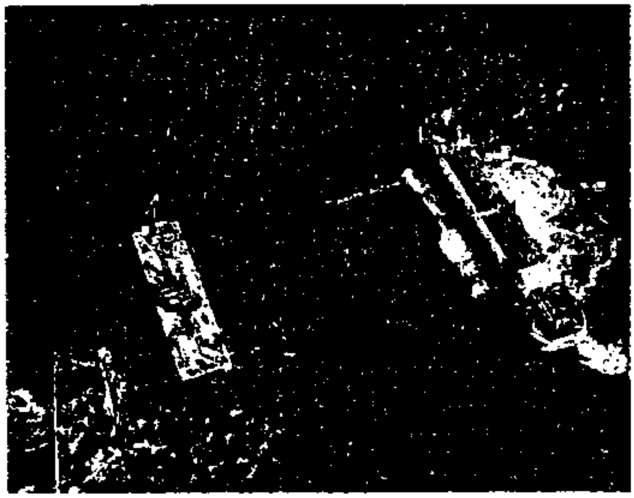


Figure 3.20: Laying Mattress

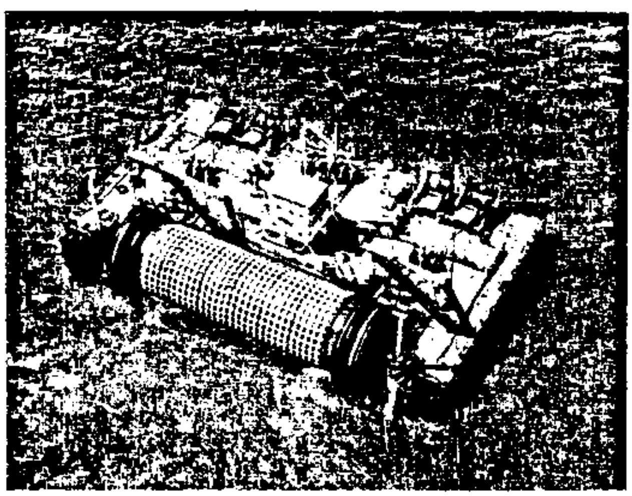


Figure 3.21: Laying Mattress

ADVANTAGES OFFERED BY PLASTIC FILTER FABRICS

1. The filtering ability (permeability and particle retention) is factory controlled and cannot be altered due to careless placement.

2. They possess good tensile strength, thus preventing rupture during subsequent rock placement.

3. Quick visual inspection assures architects and engineer the filter is in place as designed. (Screening and inspection of graded material, and inspection of placement to insure proper thickness and compaction are eliminated).

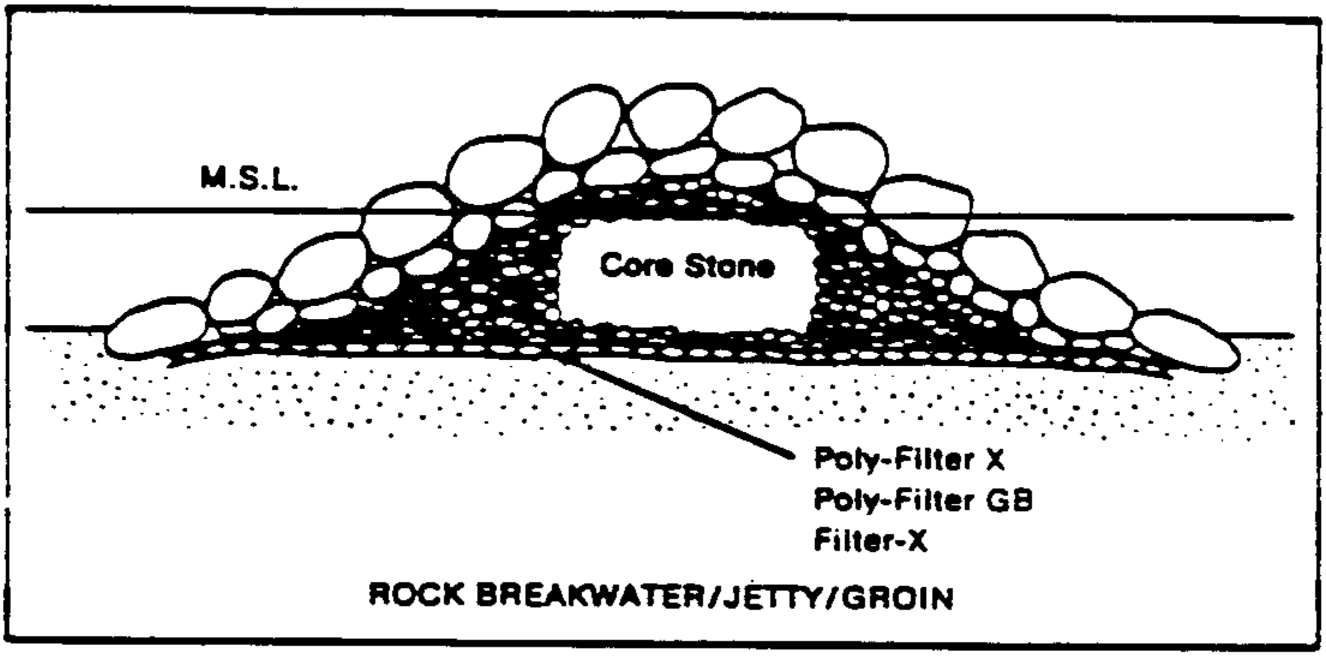
4. Permit greater opportunity for consistency in filter design.

5. Geographic location and availability of materials (sand and gravel) are eliminated as economic considerations in the design of the filter system.

Relief of water presssure and prevention of loss of soil is important in any design; it is especially critical in a rock structure. Since a rock structure has no independent strength, it depends upon the soil for its stability; if soil leaches thru the armoring units, the structure will fail.

Poly-Filter X, Filter-X, and Poly-Filter GB have been successfully used to stabilize structures armored with rock, sand-cement riprap, interlocking concrete block, and Gabions.

Figures (3.22) and (3.23) illustrate a filter fabric applied under a breakwater base.



The Architect/Engineer has difficulty with conventional filter blankets in breakwater, jetty, or groin construction, since the lightweight stone is often washed away before the heavier units are in place. ***POLY-FILTER X® / FILTER X®*** used as the base filter has eliminated this problem in many coastal projects.

Figure 3.22: Filter Fabrics Applied Under Breakwater

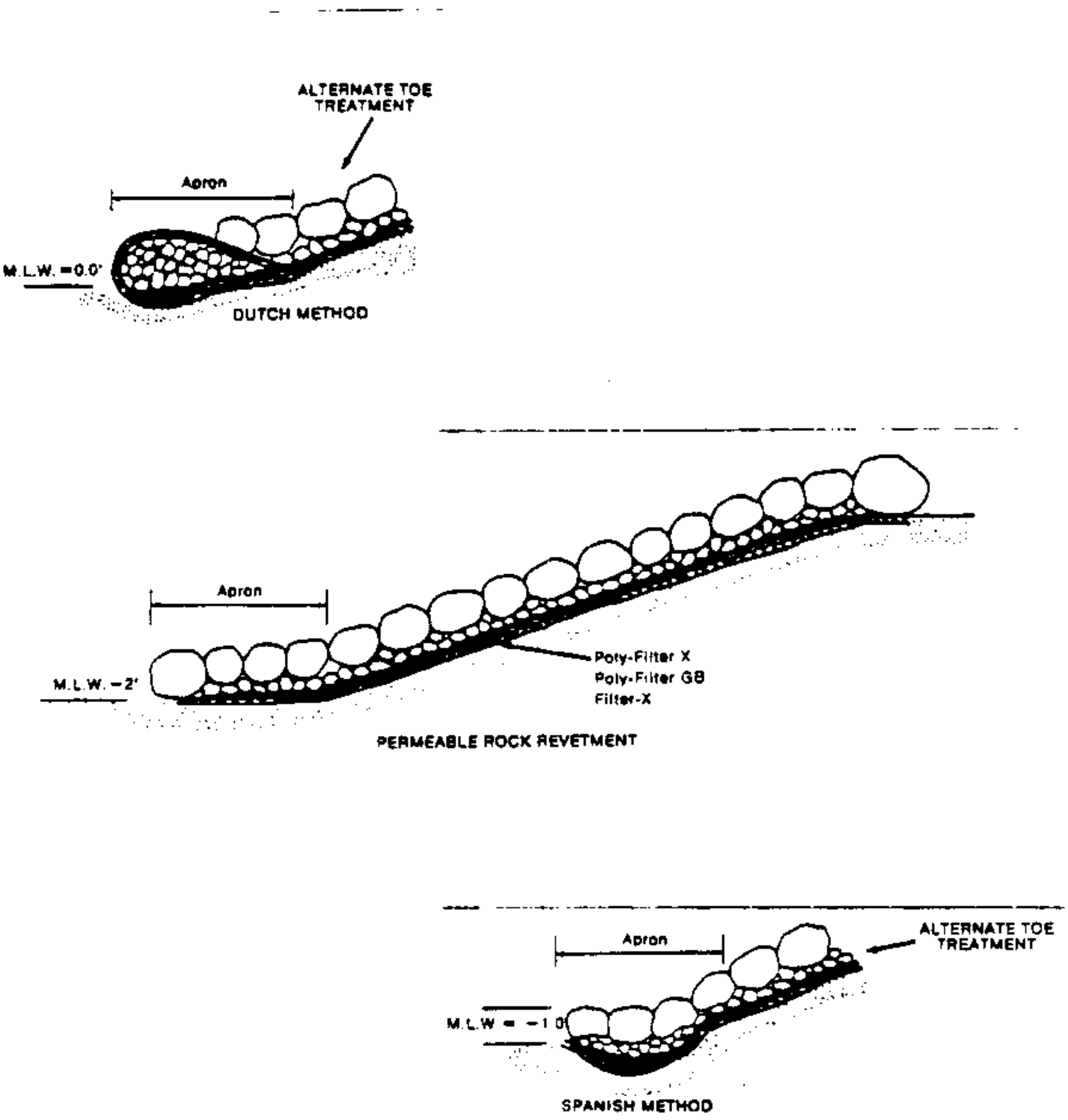


Figure 3.23: Filter Fabrics Applied Under Breakwater

APPLYING A SANDMASTIC CARPET ON THE SEABED

Another method of preparing the seabed is illustrated by the figures (3.24 and 3.25). In this method the converted ship (by the name "Jan Heijmans") lays large sand-asphalt mattresses. Recent research reported by Prof. C. Monosmith at the University of California at Berkeley indicates that rubber asphalt may produce a more flexible and resilient matrix for the sand-asphalt carpet.

The asphalt layers can be applied under water at a depth up to 20m (66 ft.) by a vessel equipped with asphalt making machinery. The nylon carpeting is unrolled from a trolley towed along the sea bottom. The carpeting has pockets at regular intervals filled with sand for ballasting. The ballasting of these underwater protective layers is also done mechanically.

Appendix I represents some illustrations and schematics demonstrating the sandmastic application process.

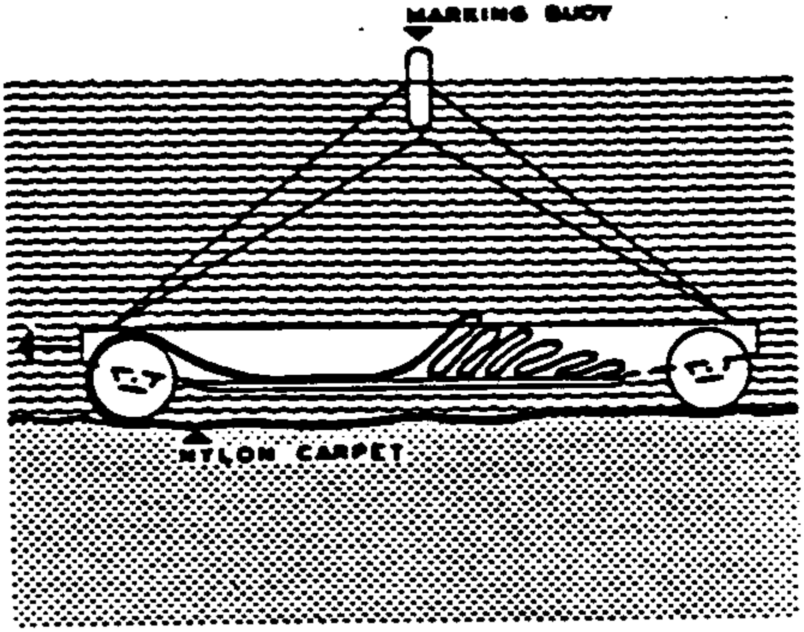


Figure 3.24: A Roller Trolley Used for Laying Nylon Cloth

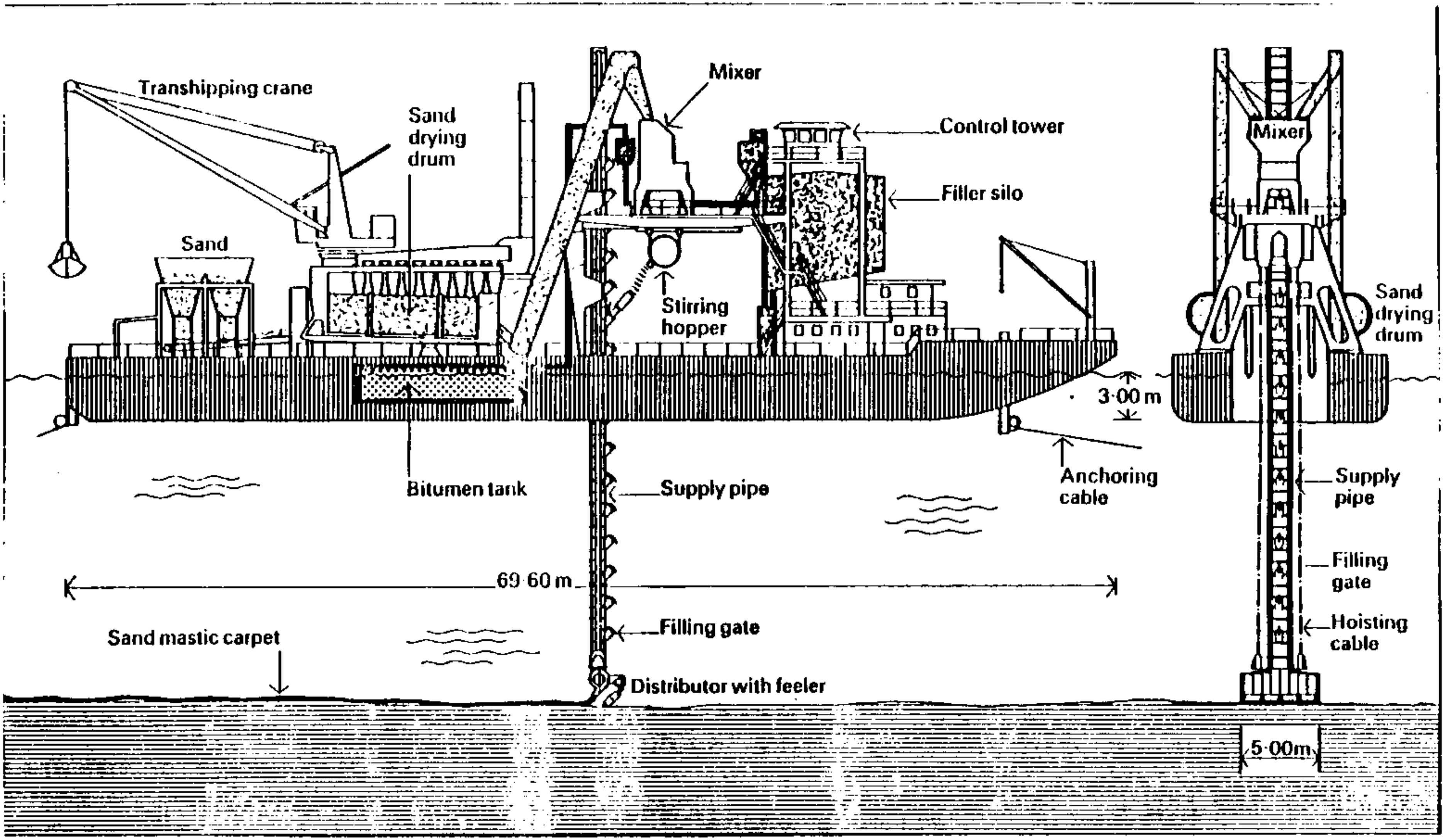


Figure 3.25: Cross Section Through The "Jan Heijmans" (1969)

CHAPTER IV

MATERIAL

The core of rubble mound breakwaters consists of rubble of different sizes, which ideally is graded and placed so as to develop maximum compactness with a minimum of cavities. The minimum size of its constituents is limited due to the fact that during construction these particles will be placed through the water, either by dump trucks or barges, and therefore must be of sufficient size to prevent washing and to be stable on the sea floor.

The core material usually consists of Class C rock which is a residue from quarry operations or consists of dredged material, or in some cases of sands (e.g. the Los Angeles - Long Beach Breakwater).

MATERIAL SPECIFICATIONS

Contract specifications for materials vary from one structure to another, depending on many different criteria such as the purpose of the breakwater, its location, the climate and environment in which it is built, the material available for construction, etc.....

One of the first requisites in the design and construciton of rubble mound breakwater is the identification of a reliable source of stone (Wakeling,1977) The core is normally selected from quarry run ranging in size from fines to individual pieces weighing 200 pounds with the average size stone weighing 25 pounds. No more than 5% by weight should pass a No. 4 sieve when tested in accordance with ASTM designation C136 (see **Appendix C**). All overburden and organic material must be excluded. If the core material consists of particles of sand or gravel, then no more than 10% percent fines should be passing the number 200 sieve and in some specifications, this number is reduced to 5%. In general, there should be a limit on the maximum size as well, so as to prevent large voids forming under the rock. Maximum limits have been prescribed ranging from 3" to 24" with 12"being the most common.

PROPERTIES OF MATERIAL

Breakwater materials must be sound, durable, hard, free from laminations or cleavages, and of such character that it will not disintegrate from the action of air, sea water, or the conditions to be met in handling and placing.

The rock chosen for the protective layers and also for the core-material, should undergo adequate testing to determine its acceptability. In a number of breakwaters, most notably San Ciprian in northern Spain, the core rock has not been properly tested and has turned out to be unsound in sea water. In addition, the model test was made from rock from a different source than the chosen quarry.

Wherever possible, service records of the rock should be considered. The tests employed are not exclusively designated for breakwaters, but in many cases are chosen from ASTM procedures that are used for aggregates for concrete.

<u>The agencies that have set specifications for rock products are</u>:

ASTM

The American Society for Testing and Materials

AASHO

The American Association of State Highway Officials

The U.S. Department of Transportation

(significantly, the Federal Highway Administration)

The Army Corps of Engineers

Various State Highway Departments

The ASTM and AASHO have developed standardized tests to evaluate the physical properties of rock pertaining to its similarity as aggregate.

1. DENSITY OF STONE

Rocks should have as high a density as practicable but in any case not less than 160 pounds per solid cubic foot. It is the underwater or buoyant unit weight that determines the stability of the basic structure: thus an increase in air density of 5% can produce an increase in underwater unit weight of 8%. Stability is normally believed to vary as the cube of the underwater unit weight, thus the stability ratios are 1:1.27, representing a 27% difference in performance with only a 5% increase in unit weight. Conversely a 5% under density produces a 23% reduction in stability. The density based on the saturated surface dry specific gravity of the stone is determined in accordance with ASTM designation C 127. The minimum value for the bulk specific gravity is 2.60. See **Appendix C** for more details.

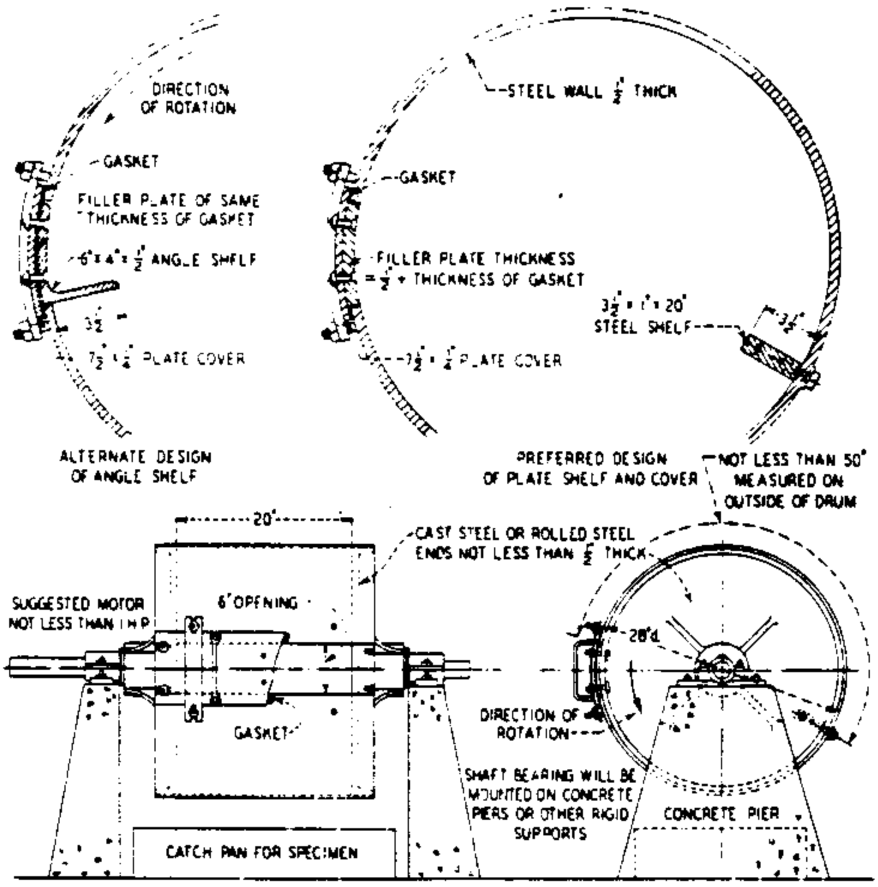
2.**DURABILITY OF STONE**

Tests that identify deleterious materials are also very important. The presence of large quantities of organic matter, chert, clay, coal, shale or other soft particles may interfere with one of the most important characteristics of the rocks for breakwater construction; its hardness and its specific density, as well as leading to potential disintegration in seawater.

A qualitative evaluation of rock durability relates to the geological origin and history of the rock; and is based on, in the first instance, a petrographic examination of the mineral and crystallographic makeup of the rock. Large scale macro studies of hand specimens and field observations of large fragments and any exposed rock faces are made to determine weak seams, such as poorly healed joints or fractures and incipient cracks. All of the above factors influence the potential durability of the rock.

3. ABRASION RESISTANCE

Abrasion tests measure the resistance of aggregates to abrasion.The Los Angeles Abrasion Test ASTM C-535 is normally employed. (This is based on the ability of rock fragments to remain intact when subjected to the extensive abrasion and impact which occurs when these fragments are rotated with iron balls in a drum, reflects the toughness and durability of the rock.) A maximum acceptable value is 45% loss in 500 revolutions. The Los Angeles Abrasion machine is illustrated in the following figure (4.1).



Metric Equivalents

in.	1/4	1/2	1	3 1/2	4	6	7 1/2	20	28	50
mm	6.4	12.7	25.4	89	102	152	190	508	711	1270

Figure 4.1: Los Angeles Abrasion Testing Machine

4. **SOUNDNESS**

Soundness tests also give a rough indication of the porosity, permeability, and tensile strength of the rock. This can be determined by the Soundness Tests in Magnesium and Sodium Sulphate - ASTM C-88, (These tests determine the resistance of rock to disintegration by saturated solutions of magnesium and sodium sulphate. These tests will determine also how much the rock weakens from the expansion of ice or salt crystals in its pore spaces. The tests are designed to simulate weathering action and to supplement information on the service records of the material when exposed to actual weathering conditions.) For typical acceptable values, see **Appendix C.** These tests are indicative but not conclusive: they should be considered in conjunction with service experience wherever available.

5. **ABSORPTION AND SPECIFIFC GRAVITY**

Specific gravity measurements indicate the solid volume occupied by the rock and are significant in the proper design of the stability of breakwaters.

Test for Absorption and Bulk Specific Gravity are designgated as ASTM C-127. (This test provides an indication of the rocks porosity and is a measure of resistance of stone to freezing and thawing. The specific gravity provides a measure of a fragment susceptibility to movement and indi-rectly relates to mechanical abrasion in the breakwater.) Typical acceptable values are 2.7 and up.

6. COMPRESSION STRENGTH

Strength tests are also important in evaluating the performance of the rock under the variable loading forces to which it will be subjected to.

Test for Compression Strength is designated as ASTM C-179 (This test provides an index of the general strength and toughness of the rock element and also is an index of durability.)

7. SHEAR STRENGTH

Type IV core stone may be tested to determine its shear strength. The testing consists of a series of consolidated drained triaxial tests. The 12-inch diameter triaxial cell of Geotesting, Inc. of San Rafael, California is currently being used to test various stone specimens for acceptability in breakwaters.

8. GRADATION

Gradation tests are useful in determining if crushed materials meet appropriate standards for the specific application and requirements.

The core material is the smallest size rock in the total breakwater system. It must be protected by filter layers gradually increasing in material size until the armor layers. A proper gradation will provide the structure with minimum cavities for entrapped air which will increase internal pressure. Figures (4.2 and 4.3) show a mix of quarried rock for breakwater construction. The gaps in gradation sizes is very well illustrated. To remedy this problem, specific sizes of rock have to be obtained which means an increase of cost.

Figure 4.2: Mix Of Quarried Rock At Sines Breakwater, Portugal

Figure 4.3: Mix Of Quarried Rock At Gijon Breakwater, Spain

9. PRESENCE OF FINES

Another important test is a washing procedure used to determine the amount of 200 mesh material present in crushed stone. For breakwater construction, an excess of fines is undesirable, as it will be washed away during placement. It should be noted that the very act of dumping the rock through the water tends to improve its strength, while at the same time reducing its impermeability.

10. SHAPE OF ROCKS

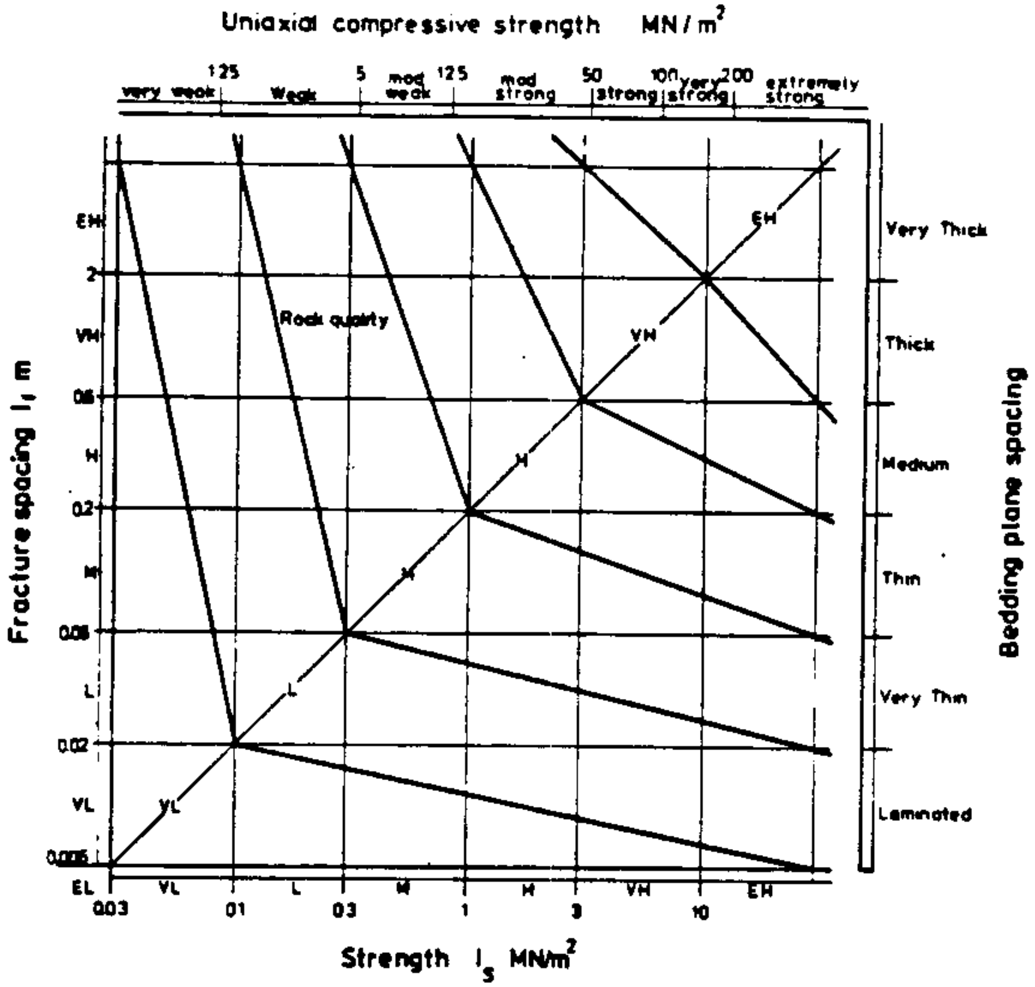
Shape of crushed particles is another key property that must be looked at when the quality of the rock in a prospective quarry deposit is being assessed.

A large quantity of flat or elongated particles are undesirable because they tend to prevent consolidation and produce voids. This will lead to a reduction in density and the overall strength. Flaky particles break easier than the equidimensional ones. Also flaky materials increase segregation during placement. Round rocks such as cobbles are also undesirable since they tend to roll and have a very low angle of internal friction.

11. SURFACE ROUGHNESS

Surface roughness is very important in the core rock material chosen for construction. A rough surface will increase friction between the particles and increases stability. Water polished or glacially-polished rock will have a lower angle of internal friction.

D'Angremond, Weide and Woestenenk (1970) presented a method for rock classification, based on field experience. A graphic illustration, of the classification of strength and fracture spacing of rock, used in each component of a rubble mound breakwater is demonstrated in the following figure (4.4) (Wakeling,1977).



EH EXTREMELY HIGH
VH VERY HIGH
H HIGH
M MEDIUM
L LOW
VL VERY LOW
EL EXTREMELY LOW

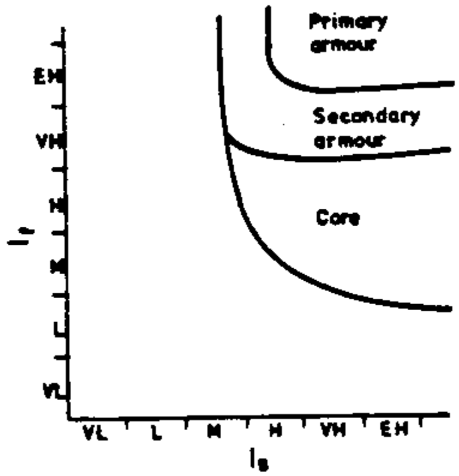


Figure 4.4: Classification Diagram Showing The Different Classes Of Rock Material

Wakeling,(1977) presented "a typical specification of rock quality used by his firm in the selection of material:

(i). Apparent specific gravity not less than 2.6 (Test Method in BS812).

(ii). Water absorption not greater than 3% gain by weight. (Test Method in BS812).

(iii). Aggregate Impact Value 30% for standard test fraction (Test Method in BS812).

(iv). "Ten Per Cent Fines" value not less than 10 tonnes (Test Method in BS812).

(v). Soundness($MgSO_4$) not greater than 18% loss by ASTM Method C88 Test.

(vi). Prismoïdal shape, maximum dimension less than or equal to twice minimum dimension."

TYPES OF ROCKS FOR THE CORE

The types of rock most suitable for constructing a breakwater are:

A. Those which have a source favorably located for ecomonically producing fragments of rubble of the required sizes and quantities.

B. Those types of rock sufficiently tough to withstand both the short-term processes associated with quarrying, transportation and placing and the longer term processes associated with the life cycle of the breakwater.

The following paragraphs are parts of a study on the properties of rock used in the construction of rubble mound breakwater, (A.G.S., P.S.A.R., Amendment 1, April 17, 1974). This study was prepared for the Diablo Canyon breakwater, California. Appendix C represents more information on these specifications.

"Geologically, the most suitable materials are derived from massive, fine-grained igneous rock such as diabase or basalt; the massive metamorphic rocks such as greenstone and quartzite and the massive sedimentary rocks such as hard sandstone and limestone. Coarser-grained igneous and metamorphic rocks such as granite and granitic gneiss are also suitable. Lithologic suitability applies especially to the larger size ranges of rubble, since for the most fine-grained igneous and sedimentary rocks, their inherently closer joint spacing or bedding tends to limit the maximum size."

" Highly foliated rocks such as schist, micaceous gneises and some amphibolites, as well as thinly bedded, sheared, and excessively jointed rocks are not suitable, since these are likely to shatter on blasting, producing a large number of flat elongated particles that would leave voids in between them and would require more fines to fill the gaps." If these voids are not filled, the whole structure will be excessively permeable and may have low shear resistance."

"The importance of the characteristics of natural discontinuities (i.e. jointing, bedding, etc.) in the rock mass cannot be overstressed, since these control not only the ability of the rock to break into angular roughly equidimensional fragments showing suitable surface roughness; but they control also the distribution of fragment sizes (gradation) in the production blast. Such properties, if favorable, contribute significantly to a relatively dense rockfill with a relatively high angle of internal friction."

This is a very important point in the construction of breakwaters. The importance of these characteristics of the core rock is illustrated in the following schematic figures (4.5 and 4.6).

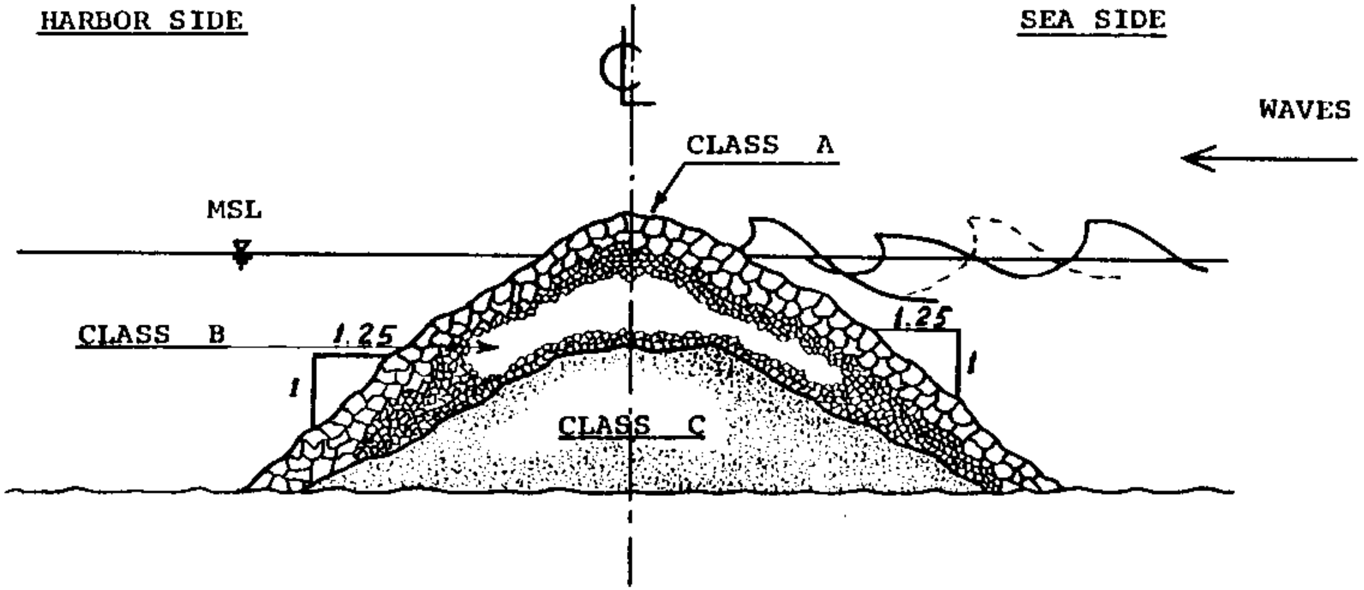


Figure 4.5: Schematic Of A Rubble Mound Breakwater

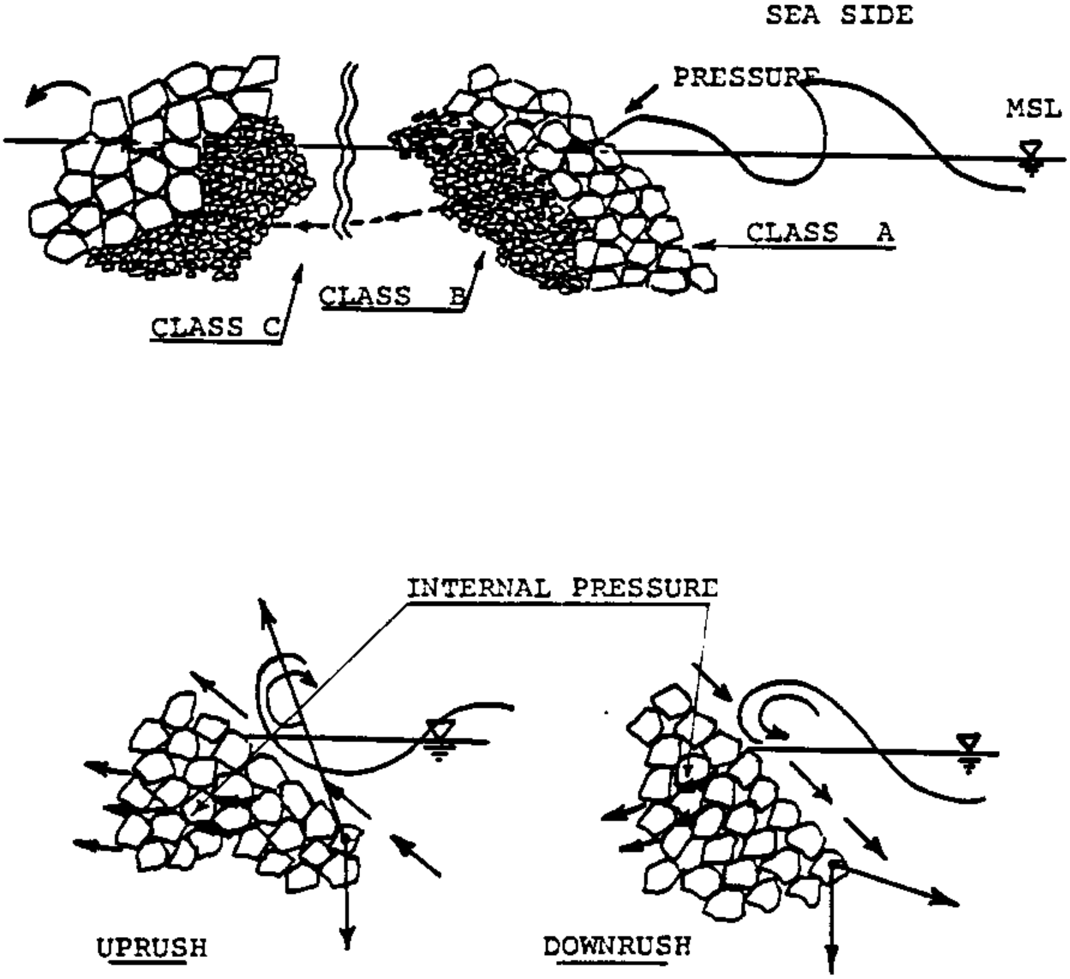


Figure 4.6: Forces Acting On The Outer Slope And Effecting the Core

As a result of this wave attack, hydrostatic pressure (P) will travel between the rocks whenever there is a free passage and may end up underneath one of the cover stones, so as to dislodge, roll or eject it. The shock wave of breaking waves can reach instantaneous values as high as 30 $^{T}/_{M}{}^{2}$ (ref. Det Norske Veritas, "Rules for the Design, Construction and Inspection of Fixed Offshore Structures", 1977) Once the armor rock is displaced, progressive failures may take place. The armor rocks have lost their interlocking aspects thus creating more voids and openings so that the core can be easily washed out.

However, if there is a proper distribution of fragment sizes, these particles wil fill the interstices and create internal friction that will attenuate the forces created by the impact of waves through the rocks. This will help prevent the displacement of the cover layer.

"Suitable breakwater stone is further geologically characterized by being resistant to physical and chemical weathering. Rock with a minimum of internal voids is usually least prone to frost action. Chemical weathering involves rock disintegration accompanied by marked changes in chemical and mineralogical composition. Over a geological time span, the complex silicates that make up the bulk of igneous and metamorphic rocks commonly change into hydrous silicated hydrous oxides, and carbonates. Some materials, such as carbonates, are lost by solution. With the exception of carbonates, the susceptibility of a rock to chemical weathering is related to the temperature at which the rock formed. Higher temperature rocks (igneous and metamorphic) are more susceptible than lower temperature rocks (sedimentary). Except under extreme climatic conditions (sub-Artic or Tropical), weathering is a very long-term process and the possibility of weathering adversely influencing any sound rock placed in the breakwater during its 40 year life cycle is extremely remote." (A.G.S./P.S.A.R. 1974).

"Favorable geological rock types relate to materials with favorable engineering index properties. In other words, properties such as mass per unit volume, compressive strength and durability in terms of mechanical and chemical stability, are a function of the mineral composition, texture, and macro structure of the rock element. Favorable properties are generally associated with such features as fine grain size, a lack of micro-fracture or well-developed cleavage, hard stable minerals, isotropic mineral arrangement, and good crystal interlock. Conversely, less well suited rocks are associated with coarser grain size, the presence of micro-fractures, unstable soft minerals, an abundance of minerals with well-developed cleavage, a well defined anisotropic mineral arrangement, and a clastic texture." (A.G.S. 1974).

The susceptibility of a rock type such as diabase, granite or granitic gneiss to weathering or break-down in a salt water environment is almost non-existent. Most chemical weathering phenomena occur in an acid environment ($pH < 7.0$) under conditions favoring oxidation, where precipitation containing dissolved CO_2 acts as the solvent. In the ocean, however, the pH is always in the neighborhood of 3.0 as the combination of several solutes (most notably the $CO_2 - H_2 - CO_3 - HCO_3 - CO_3$ system) serve as an effective buffer. This inhibits the rapid fluctuation of pH and is probably responsible for the relatively benign conditions which have prevailed in the oceans over a long geologic time period.

While the dissolution of minerals within diabase, granite or granitic gneiss in a salt water environment is feasible thermodynamically, it is kinetically improbable. The concentrations of anions and cations already present in sea water (and common to those in the subject crystalline rocks) can approach saturation at certain temperatures and pressures. Such a condition would make it highly unlikely that solution of any of these minerals would take place. This is evidenced, in part, by several petrographic analyses of rock specimens from existing breakwaters. Clinton Point dolomite from the Jones Beach Breakwater (N.Y.) and granite from the Hyannis Port breakwater (Mass.) (granite from Stonington, Maine) were analyzed in thin sections and showed no mineral alteration after up to 40 years of exposure in a salt water environment.

Unfortunately, the majority of published detailed studies have been directed towards rock on the Atlantic coasts, and thus are not directly applicable to the Mid-East. Because of the geological history of the Mid-East with its lack of natural weathering, the rock qualities there require more examinations and study. There is limited experience available by which to evaluate the performance of rock from particular sources.

The properties of the rock may be evaluated in quantitative terms in accordance with standarized laboratory procedures, as discussed earlier in this chapters. Specific gravity is an important index property in this respect since as noted earlier, the specific gravity of a rock is directly related to its resistance to movement. An illustrative example is discussed by Treasher who compares two similar sized stones, one having 10% greater density in air than the other. The heavier stone, when immersed in water has 50% more resistance against movement by wave action than the lighter stone. A specific gravity of greater than 2.6 is desirable. Frequently, however, there will be an alternate source with rocks of heavier unit weight and the added cost of transportation may be offset by the better performance.

There have been relatively few studies on the actual forces induced by waves on individual units of a rubble mound breakwater. In design practice a type of integrated approach is used. The formula generally used to calculate the size of armor rock to use is a modification of the original Iribarren formula, and (Hudson, 1953 ,1959) where W is the weight of armor units (pounds), Y_r is the unit weight of the armor units (pounds per cubic foot), H_s is the design wave height (feet), usually taken as the significant wave height at the structure location with no structure present, K_D is a experimentally determined dimensionless coefficient that varies primarily with the shape of the armor units, roughness of the surface, sharpness of the edges and degree of interlocking obtained in placement, S_r is the specific gravity of the rock relative to the water in which the structure is placed (S_r = Y_r/Y_w' where Y_w is the unit weight of the water; Y_w = 64.0 pounds per cubic foot in the ocean), and θ is the angle of breakwater slope measured from the horizontal in degrees.

$$W = \frac{Y_r H_3}{K_D (S_r - 1)^3 \cot \theta}$$

It is suggested that the relationships given by this formula also have applicability to the coarser fractions of the core rock.

Design suitability of rock is evaluated in terms of the efficiency with which individual rock fragments recombine and act as a single rockfill mass. The critical design parameter in this respect is the angle of internal friction needed to prevent the shear failure within the core mass, this also determines the safe slopes, which defines the strength of the rockfill structure. The highest angle of internal friction is naturaly desirable. Determination of this parameter should be consistent with the confining stresses and density anticipated for the core material of the breakwater.

Some of the major breakwaters built in the last few years are suffering damage and experts in this field are pointing to the poor quality of rock used as being one of the reasons of failure. Based mainly on the economic aspect of the construction of the breakwater, the closest quarry is often chosen as a source for the core rock. Most of its rock is used, regardless of quality, and often it does not properly meet standards as to gradation and quality.

The crushed stone industry is considered an important industry all over the world. The use of crushed stone is tied in very closely with the construction industry, and thus the standards developed for its use in construction may contribute to improving the quality of the core of rubble mound breakwaters.

In order to better evaluate the production of rock for breakwaters, an indepth investigation was made into the procedures currently used for site selection and rock production in existing quarries.

GEOLOGIC INVESTIGATION OF QUARRY SITES

An important activity which must be carried out before beginning the operation of any crushed rock quarry is a geological investigation of the quarry site. It will assist in a better evaluation of the properties of rock used for the rubble mound breakwater, both for the armor rock and the core and filter layers.

The Purpose of a geologic Investigation is to evaluate the continued economic usefulness of an existing quarry deposit near the proposed construction site for the breakwater or to explore a new deposit to determine if it will be worth developing.

A PROPER GEOLOGIC INVESTIGATION SHOULD CONSIST OF SEVERAL PHASES, as described very well in Zaruba and Mencl (1976):

1. Office research.
2. Preliminary deposit reconnaissance.
3. Detailed exploration.
4. Selection of operational methods to be used in exploiting the deposit.

The investigation should start on a general basis and continue to more detailed studies if the initial research and reconnaissance indicates that further work is warranted.

FIRST PHASE:

In the first phase of an investigation of a quarry site, the initial step is to establish the type, size, and quality of material that is required. In addition to the characteristics of the rocks which have been mentioned previously in this chapter, it is important to specify the sizes (gradation) of rocks required for the different parts of the breakwater.

Once the material requirements have been established, prospective **sites should be selected.** The location of the site relative to the breakwater will dictate the methods of transporting the rock products which can be used. Transportation costs are a key factor in determing the deliverd costs of the core and armor rock and the overall economy of the project.

Geological information can be obtained from geologic maps. One drawback of many maps and publications, is that the rocks are described from a purely geological standpoint without any discussion of their engineering characteristics.

SECOND PHASE:

When office research on a quarry site has been completed, a preliminary reconnaissance of the deposit should be made **to see if it is of sufficient size,** and **the core rock of sufficient quality to meet the previously established material requirements,** as described in the beginning of this chapter.In addition to testing for rock quality, data from the preliminary field reconnaissance should be used to compute approximately the total volume of the deposit and make a rough estimate of the reserves available. These rough computations are important because they give a first indication of whether or not the deposit is large enough to be worth exploiting.

Samples of the rock should be taken for analysis to determine pertinent properties such as: **friability, toughness, cleavage, color, structure** and **weathering characteristics.** Several representative samples of the rock to be quarried should be collected, in order to obtain a characteristic range of material properties, since some variation is to be expected. **Petrographic** and **X-Ray diffraction techniques** can be used to ascertain the mineralogic content of the rock, as well as give additional clues to its durability and other properties.

Chemical analysis is helpful to assess the suitability of the rock to withstand the seawater attack. The quality of the rock to be quarried is usually evaluated by seeing how well it meets crushed stone specifications established by the various public agencies,(ASTM, AASHO ect..), and consultant firms, since their standards are the most widely used in the construction of breakwaters. Unfortunately this procedure is not always vigorously applied to specific projects, even though breakwaters are designed to last for long periods of time in an extrememly severe environment.

THIRD PHASE:

Detailed explorations of the quarry site. This phase should include detailed mapping of the topography and geology; photogrammetry may be helpful for these activities.

The comprehensive site survey should be undertaken from an engineering point of view, both in mapping the site and in the classification of the rocks, keeping in mind the functions that these rocks will be used for in the total system of the breakwater.

In mapping the site geology, the following items should be considered:

1. Major joint systems
2. Faults and Fractures
3. Extent of overburden
4. Outcrop locations
5. Strikes and dips of exposed beds
6. Location of all test, pits, trenches and boreholes used in explorations.

The volume of the overburden of soil and weathered rock should be determined. An excessive amount of overburden material may render this quarry site uneconomical for this project because of the expenses involved in the removal and disposal of the unuseable material.

The mapping of the quarry site should be carried out even after the mining operation begins in order to gather more detailed information. Opening a face (trial pits) can be very helpful in locating subsurface cleavage planes. This will provide the quarry engineer with the necessary information to determine if the site is appropriate for the project at hand.

FOURTH PHASE

At this stage the primary site has been assessed and determined to be economicaly feasible. The quarry engineer will then develop the best method to be used in the operating procedures. These activities should be planned in a very precise manner.

1. The development and sequence in which the rock will be removed.
2. The method in which the overburden can best be stripped, and wasted.
3. The selection of the blasting equipment.
4. The design of the blasting patterns.
5. The selection of the powder type, speed, loading ratio, and stemming.

6. The selection of the transportation equipment, to move the blasted rock from the quarry to the processing plant.
7. The layout of the material stockyard.
8. The means of delivering the product to the construction site of the breakwater: selection of equipment and methods.

These four phases represent the begining of the quarry operation intended for producing rock used in the complete breakwater. Follow-up surveys and investigations should be conducted on an ongoing basis, so as to improve the efficiency and productivity of the quarry.

The following table (IV.1) represents a summary of the four phases.

PHASE I	PHASE II	PHASE III	PHASE IV
OFFICE RESEARCH	**PRELIMINARY DEPOSIT RECONNAISSANCE**	**DETAILED SITE EXPLORATION**	**SELECTION OF QUARRING METHODS**
1. Determination of material to be produced.	1. Study of mode of deposition.	1. Topographic and geologic mapping.	1. General layout of the quarry pit, crushing plants and material stockyard.
2. Selection of prospective sites.	2. Rough estimates of deposit and overburden thickness.	2. Study of hydrogeological conditions.	2. Development of stripping and disposal plans for overburden.
3. Geological Research.	3. Analysis of rock samples - important properties, performance in standardized tests for the core material.	3. Refinement of estimates for deposit volume, available reserves and extent of overburden.	3. Selection of mining equipment.
	4. Assessment of how well rock meets specifications set by public agencies for crushed stone.		4. Design of crushing plant.
			5. Selection of blasting equipment and design of blasting program.
			6. Study of equipment and methods for delivering rock to the breakwater site.

SUMMARY OF THE FOUR PHASES REPRESENTING THE QUARRY OPERATION

TABLE IV.1

QUARRY OPERATIONS

REMOVAL OF THE OVERBURDEN

Stripping the ground cover layer to reach the rock that will constitute the rubble for the core material, as well as the filter layers, and armor units, is usually the first operation performed in most quarries. The depth and the area to be stripped vary from one site to another. This overburden **should not** be used in the core material unless it passes the tests previously mentioned. It is of course desirable to remove as little material as possible so as to lower the cost of the quarrying operation while still ensuring that the remaining material will meet the specifications.

FACTORS INVOLVED IN THE LAYOUT OF THE QUARRY OPERATIONS

While extension of an existing quarry operation has the advantages of facilitating the geological investigation and experience as to fragmentation when shot, nevertheless it is often easier to open a new quarry for the production of rubble for the construction of breakwaters, than to readjust the existing quarry.

1. The geometry of the deposits and its orientation relative to the existing site topography and overburden plays an important role in the design of a quarry. If the exsisting quarry has been established for

the production of crushed rocks to be used in highway construction, the equipment and method of mining (drilling and blasting according to the geometry of the deposit) will differ from the equipment used to produce the wide variety of sizes of rocks needed for breakwater construction.

2. The geologic structure of the rock such as, bedding, joints, and folds will be a major factor in the stability of excavations made in the deposit. The quarry engineer should design his layout to minimize the possibility of the development of large unstable zones. This includes the hydrogeological conditions and the flow patterns of ground water seepage. Proper drainage should be observed to avoid such problems.

3. Since rubble mound breakwater construction uses a large volume of rocks, this makes the production requirement one of the most important factors influencing the planning and design of a quarry operation. It is necessary to select the type of mining operation as well as the layout of benches and ramps, and the drilling and blasting

procedures to produce the size and grade of rubble mound material that will meet the specifications for the different sections of the rubble mound breakwater.

DESIGN OF BENCHES

In designing the benches in a quarry, the geological characteristics and safety should be very thoroughly examined. Benches serve as a means of protecting personnel and equipment from slope failures and provide flat areas to spiral down from the top of the pit to the bottom, or to lay out horizontally with connecting ramps between them as a working area giving access to different sections of the quarry (**Appendix D** provides consideration for berm selection and bench design.). The geological characteristics of the material will control the maximum slope and height that can be safely cut into the rock. These characteristics include:

1. The strength and density of rock.
2. The orientation and strength of discontinuities such as bedding, joints, shear zones and folds.
3. The drainage conditions at the site.
4. The size and type of equipment that is used in the quarrying operation.

Once the sizes and types of mining equipment have been selected, the benches can be designed to provide adequate room for safe and efficient maneuvering of the excavators and haul trucks. The following figure (4.7) is a photograph of the quarry used for core and underlayer material for Sines Breakwater, Portugal. It is also showing the benches and quarrying operation.

Figure 4.7: Quarry Of Sines Breakwater, Portugal

DRILLING AND BLASTING

After preparing the quarry by stripping away the overburden, the next step is to break the rock deposits into the sizes of rocks needed for the breakwater construction. This is the most important step in the entire quarrying operation. If it is carried out with careful planning, minimum material will be wasted, and the cost of the entire project will be reduced. The major attention should be directed to the Class A rock, since this is usually the scarcest size to obtain, and is the highest in price. This focus on Class A rock will normally dominate the decisions as to drilling and blasting operations. For this reason the blasting program should be tailored to give minimum fragmentation. Large, widely spaced drilled holes and low velocity explosives will produce large fragments of rock.

SELECTION OF DRILLING EQUIPMENT

Controlling the size of rock fragments depends on the design and layout of the drilled holes and the equipment used. The factors involved are:

1. Production requirements
2. Volume of rock that must be blasted.
3. The amount of drilling that must be done.
4. The hardness of rock and the degree to which it is fractured.
5. The terrain at the site.

6. The mining operation.

TWO OF THE MOST WIDELY USED TYPES OF DOWN-HOLE RIGS are:

1. Rotary drills use tri-cone roller bits that could be made either with steel teeth or tungsten carbide teeth depending on the type and hardness of the rock to be drilled.
2. Percussion drills use cross bits which have four tungsten inserts set at right angles to each other or X-bit with the same inserts, but set at angles forming an X pattern. These drills can be either surface- mounted or down-hole. The major difference between the down-hole percussion drills and the surface-mounted drill is that the down- hole drill gives greater penetration rates in deep holes.

Each type of equipment and bits have both advantages and disadvantages. There is a great variety of drilling equipment on the market and the quarry engineer should select the proper type of drilling equipment depending on the factors discussed previously.(Morrell and Unger, 1973)

SELECTION OF THE EXPLOSIVES

The procedures and equipment used in quarry operations must be determined with full regard to the geological character of the quarry and the sizes of rock required for a specific structure. This is especially important where large quantities of armor rock and even greater quantities of core material are required.

The two types of explosives most widely used in quarry operations are:

1. ANFO (ammonium nitrate with fuel oil) which has several advantages:
    a. Low cost
    b. Equal performance to dynamite (if it is carefully controlled when used.)
    c. Safe because of its low sensivity (Gregory, 1979)

   The major disadvantage of this blasting agent is the fact that it is not water resistant, i.e. it is soluble in water. The use of SBA (slurry blasting agent) will be more suited for these wet conditions, because of its high velocity of detonation.

2. WATER GELS (which are highly explosive materials). Their main advantages are:
    a. High loading density and performance.

b. Low sensitivity to impact.

c. Water resistant.

d. Absence of headache-causing ingredients.

A third type of explosive material used in special cases is dynamite (nitroglycerin). This explosive is very dangerous and causes headaches known as NG Headaches. Extreme caution should be maintained in its use.

The initiating devices used with the ANFO and the slurry blasting agents should be a high energy priming charge, because of their low sensitivity.

There are many types of detonators. The milli-second delay electric detonator should be used when quarrying for armor rock material. This is due to the fact that individual charges when fired milli-seconds apart produce overlapping shock waves which will give a much better fragmentation as well as a lesser amount of fly-rock and a smaller amount of over break(Gregory 1979). To produce the core rock material, an opposite procedure could be applied.

According to Gregory (1979), the most effective stemming material is sandy clay. However, it often is not readily available, and thus other materials are used as substitutes.

Each blast is designed with a pattern which is selected to achieve the most effective results. The most important factors in the determination of these patterns are:

1. The properties of the rock(Its strength, the distribution and orientation of discontinuities etc..).
2. The geometry of the bench section being blasted (specifically the availability of free faces, since rock breaks more readily to a free face.).
3. The sizes of rock to be developed.
4. The amount of rock to be required.
5. Water content in the holes.
6. Safety precautions and regulations.
7. The weather conditions at the time of the shot.
8. The restrictions concerning air concussion and ground vibration.

The parameters involved in the design of the blasting patterns vary according to the size of rock to be fragmented. These variables should be specifically determined to produce the wide variety of rubble for the construction of breakwaters. They are :

1. The spacing of the holes.
2. The burden

3. The hole depth and diameter.
4. The sub-drill depth.
5. The number of holes.
6. The number of rows.
7. The size of each individual charge.
8. The detonation sequence or delay pattern.
9. The amount of stemming for each hole.

These nine parameters do not necessarily represent all the variables that should be considered but they are the major factors.(**Appendix E** illustrates different blasting patterns.)

Once the blasting operation has been accomplished, the large rocks (Class A and B) are accumulated in stock piles in accordance with the planned lay out of the quarry. Sometimes the remaining rock can be all employed in the core, although it is desirable to put it through a grizzly (spaced bars) so as to exclude oversized fragments. In other cases, where gradations are erratic, it may be necessary to screen the rock into several sizes and then reblend them during loading. As actualy practiced, the reblending operation is often very crude and subjective, depending on the judgement of the scoop loader operator as to proportions and reblending sequence. It would appear desirable to exercise more engineering control over the reblending, since the extra costs due to this control would be nominal when related to the overall costs of production.

In extreme cases, some of the rock may have to be put through a crusher in order to produce rock of sufficiently fine gradation. This is a relatively costly operation and it may be preferable to adjust the blasting program so as to obviate its necessity.

It is important to note here that very fine material should be wasted at the quarry and not be transported to the construction site for use as core material. The following figures (4.8, 4.9 and 4.10) show the quarry operation at Gijon breakwater, Spain.

Appendix C represents a copy of specifications from contract documents used by the Army Corps of Engineers in the construction of rubble mound breakwaters.

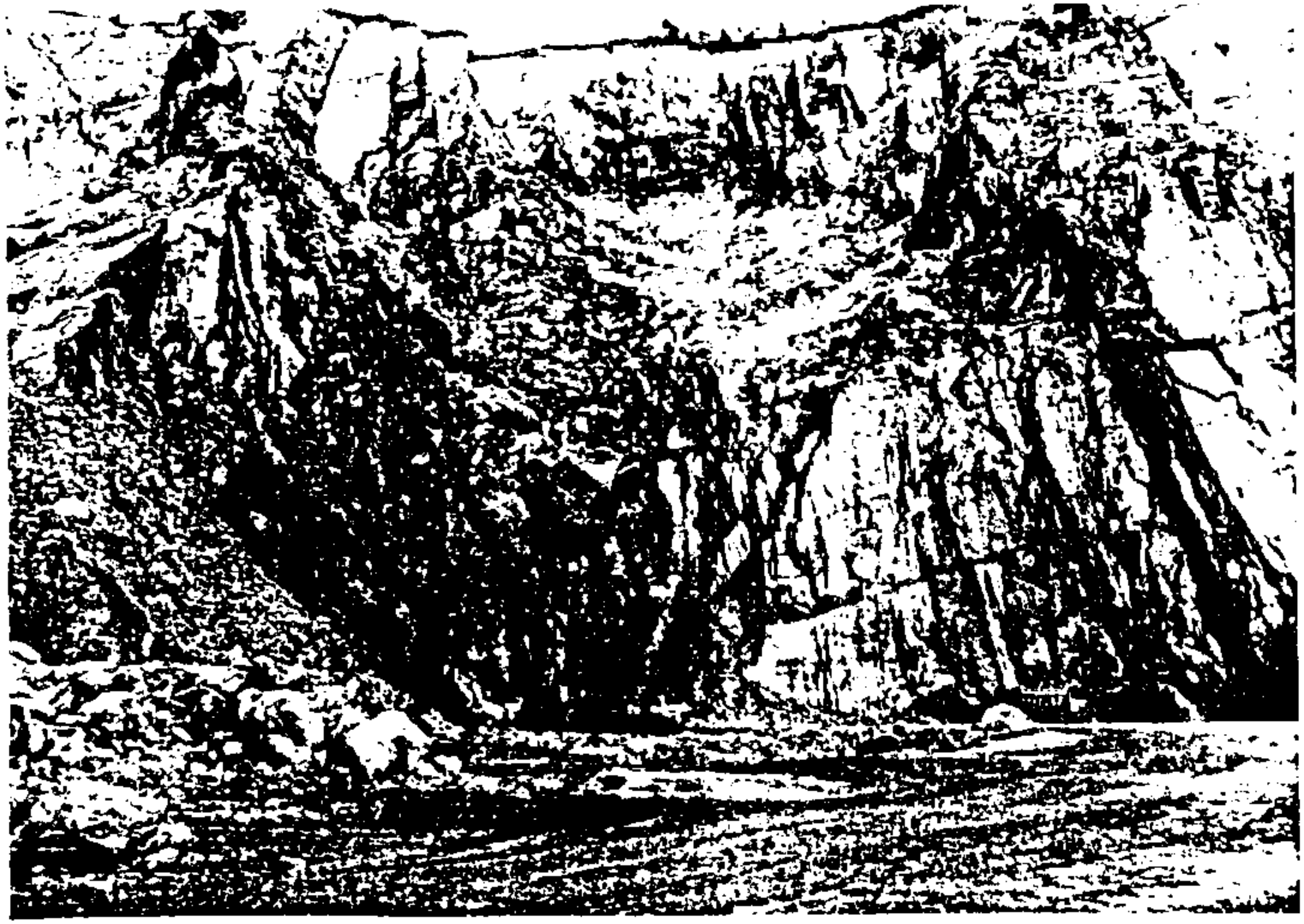


Figure 4.8: Quarry Of Gijon Breakwater, Spain

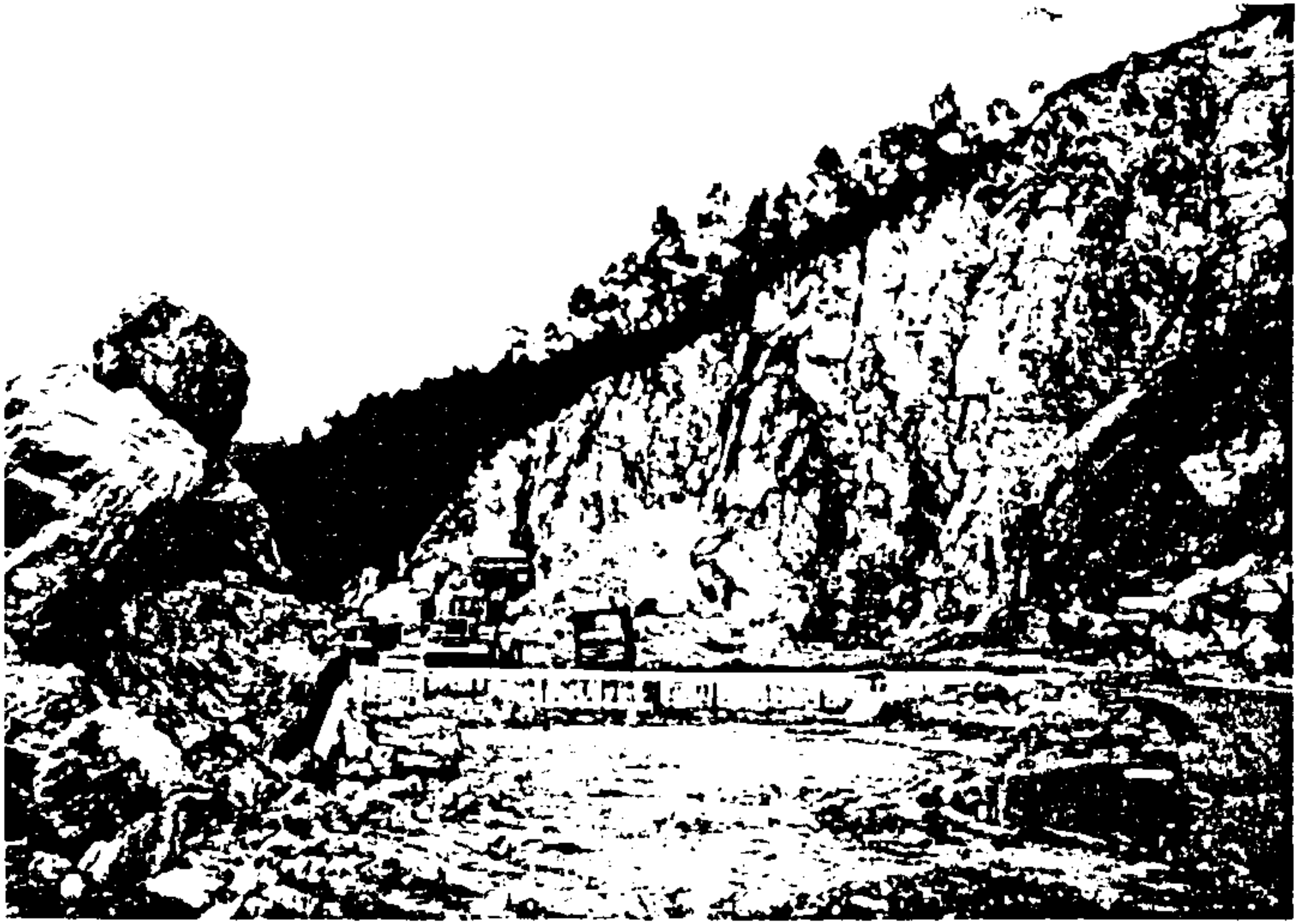


Figure 4.9: Quarry Of Gijon Breakwater, Spain

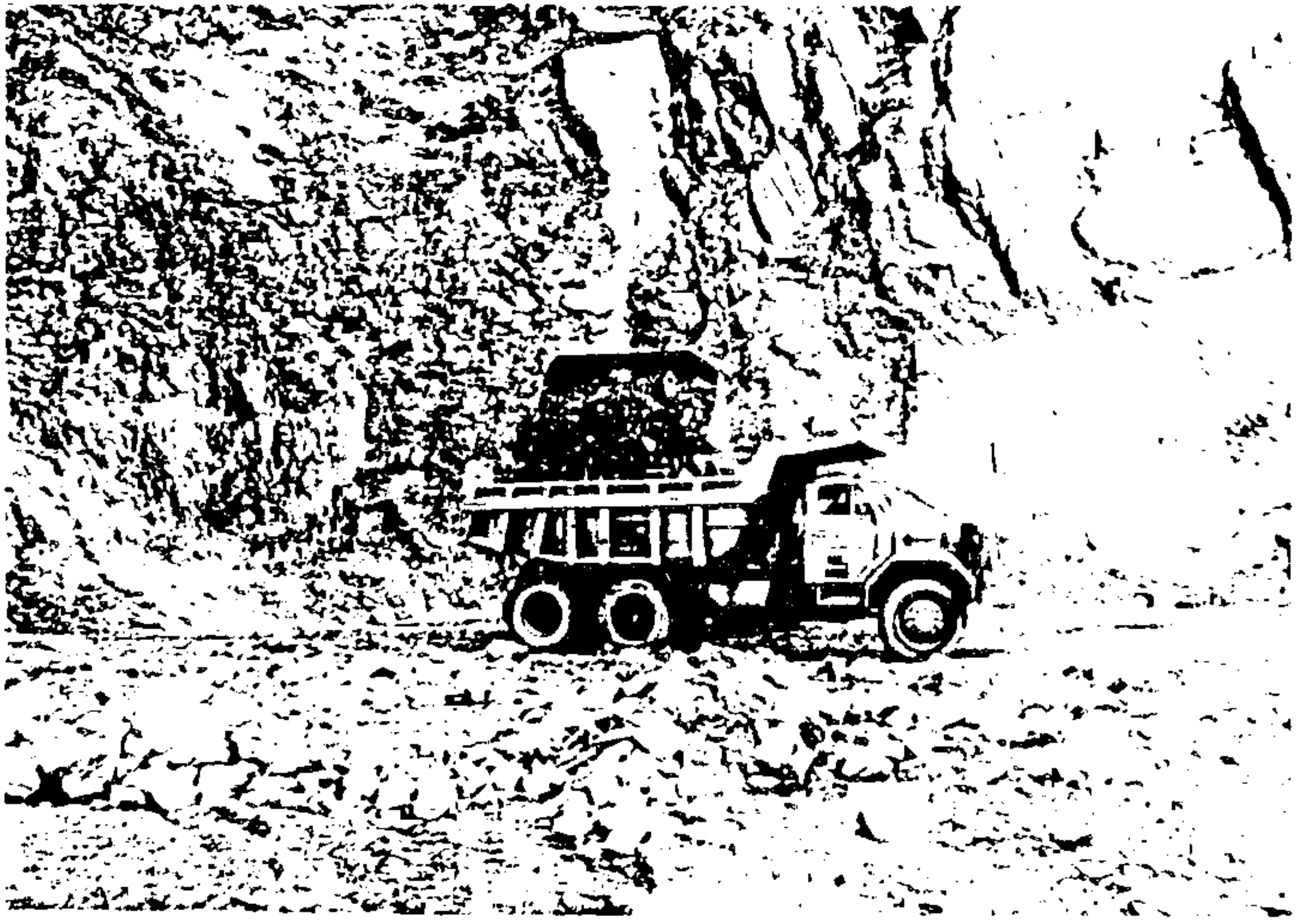


Figure 4.10: Loading Operation At The Quarry Of [illegible], Spain

FILTER LAYERS

After the construction of the core, which is class C rock or quarry run, the filter layers will follow. They are a layer of class B material, which is heavier and larger than the core material. The thickness varies from one structure to another depending on the design criteria and the wave conditions. Filter layers are very important to protect the core material from leaching out through the armor layers. Undoubtedly, several failures were caused by inadequate placement of a well graded filter layer over the core. The design must consider practical aspects of construction and supervision as well as engineering requirements of these filter layers. The control of thin filter layers is complicated and made more difficult by the fact that to a large extent they are carried out underwater and subjected to wave and surf action like the core. Internal erosion, where the materials are removed by percolating water, such as that due to water waves, surface runoff, and tidal flow will cause severe damage to the structure.

The size of stones in these filters is of primary importance. It should be graded in such a manner that the first layer next to the core will be the next size larger. The second layer should be larger in stone size than the first one. This process is continued until filter material size is large enough to resist washing away through the voids formed by the armor layer.

The criteria for filter design (Sowers & Sowers, 1970) is given as

$$\frac{D_{15}\ (\text{OVER})}{D_{85}\ (\text{UNDER})} \leq 5$$

Where D_{85} (under) is the diameter exceeded by the coarsest 15 percent of the underlayer and D_{15} (cover) is the diameter exceeded by the coarsest 85 percent of the layer immediately above the underlayer. The design principles of filter layers are presented by the U.S. Army, Corps of Engineers, Coastal Engineering Research Center (1966). Failure in the proper design and construction of these filter layers will result in an excessive amount of armor material which will escalate the cost of the entire project. Proper surveying and quality control (discussed in Chapter VI) will minimize this problem.

CHAPTER V

CONSTRUCTION PROCEDURE AND EQUIPMENT

As soon as a contractor has been awarded the project, the design team and the contractor should collaborate to determine if any changes could be made to provide a better structure and to facilitate a more efficient method of construction. The design of a breakwater core must consider practical aspects of construction and supervision as well as engineering requirements. The control of slopes and relatively thin filter layers is complicated and made more so by the fact that to a large extent they are carried out underwater and subjected to wave and surf action.

Site conditions will be the determinant of the construction method and must be considered in the design and specifications. Factors which cannot be ignored include (Miller,D.X):

1. Proximity to available quarries.
2. Transportation methods
3. Prevailing weather
4. Tides and currents
5. Seafloor (foundation soil) and bathymetry.
6. Time available for completion
7. Shipping and boating activity
8. Availability of construction areas near the water.
9. Adjacent on-shore areas for storage.

10. Whether moorage or space is available for construction equipment and plant.
11. Ecological considerations.

The objectives of a good construction method are:

1. **Rapid placement of material** during the good weather so construction will not be interrupted until the cover (armor) layers are placed and the core is protected.

2. **Minimum segregation**

3. **Economical**

4. **Not excessively sensitive to weather conditions during construction**

5. **Forms a well consolidated structure.**

6. **Maintaining acceptable tolerances**

Recent developments of heavy construction equipment have made it easier to overcome the difficulty of placing a large volume of heavy rocks. The availability of large offshore floating equipment (dump barges, large cranes, etc.) plus the Jack-up platforms (for placement of the heavier stones or armor units) has made the work and placement of rock in the surf zone more practicable.

As recently as 1967 two distinct types of construction were prevalent in the area of rubble mound breakwaters:

1. A rock mound in which the core material extends above water level and is covered with an envelope of armor rock sometimes separated from the core material by one or more intermediate layers.
2. The core fill is stopped at a considerable depth below water level and covered with a medium-weight rock, which forms the base for the heavy armor capping.

In the first type of construction the core is extended out from shore by end or side dumping from trucks which operate on top of the core as it is brought up above water level. This requires:

1. The top of the core to be above the highest water level.

2. The material out of which it is constructed to be broken rock of sufficient size with a minimum amount of fines so that it will not be washed away by the waves during the period of construction.

The disadvantages of this method are:

1. The top may have to be made somewhat wider than otherwise required by the breakwater design, in order to provide space for the trucks to operate.

2. The upper surface of the core, to form a roadway, will contain a considerable amount of fines, which will become compacted from the travel of the trucks. Before placing the capping armor rock, the top surface of the core will have to be removed or the fines washed out, so as to make a more pervious bed directly under the armor rock.

3. Unless placing the armor rock follows immediately that of the core, a considerable amount of storm damage may occur and the top of the recently placed core material may be washed away, by several feet, below water level.

The advantages of this method are:

Possible economies in construction (maximum use of smaller size fragments), especially if the quarry rock blasted has an excess of small fragments suitable for the core.

The second type of construction is based on the core being dumped or placed from floating barges and scows. The top of the core is at a considerable distance below water level and the core is covered with a medium-weight rock to a level about equal to the height of the usually expected wave below mean sea level, where it forms a base on which to place the heavier armor rock extending to the top of the breakwater.

The advantages of this method are:

1. The breakwater is less susceptible to storm damage during construction because the core is stopped well below sea level.

2. The breakwater can be constructed to optimum design configuration.

The disadvantages of this method are:

1. The amount of Class B or medium-weight rock is considerably increased over that required in the other method . This may be a problem if the quarry is deficient in this size of rock.

2. The top armor rock will have to be placed by floating crane barge. Installation with floating equipment may result in considerable loss of time because work

cannot be done from water-borne equipment operating so close to the rock breakwater except in relatively calm sea states.

In an article in the Dock and Harbor Authority Magazine, Bertlin (1967) discusses these two methods of construction in detail.

In the earlier days, the best method in breakwater construction was accomplished from "floating plants". The heavy facing stones and the topping were placed by derrick and fitted closely together, forming a structure which offers the maximum of resistance to attack by the waves. The core is built with bottom-dump scows. This type of construction is practicable only where the sea is comparatively smooth most of the time and storms of great severity are infrequent. Another method is the use of a trestle. In this case the material will take a rough natural slope as it is dumped from the cars only to be flattened out and compacted by the action of the waves. In the normal use of this method an elevated double-track tramway is constructed on the axis of the proposed breakwater. The rock is carried out on the tramway by means of specially constructed flat cars, capable of being dumped to either side. The core is dumped between the tracks and the heavy facing stone is dumped outboard on each side. This method was adopted in the construction at Humbolt Bay, California in 1899.

After a storm in 1912 the structure was maintained by a crane operating on a double track, 17 feet out to out. The crane had a capacity of 20 tons at 35 feet radius. To protect the undermining of the structure, an apron of rip-rap was placed along the toe of the slope. In 1970, another rehabilitation for Humbolt Bay took place, 116,000 tons of stone for the core were barged to the site. A Manitowac 4600 Vicon Ringer crane placed 48,000 dolosse of 42 tons each. The following figures (5.1 thru 5.3) shows an example of a crane operating on a double track, the Manitowac 4600 at Humbolt Bay, California and the storage yard for the protective units.

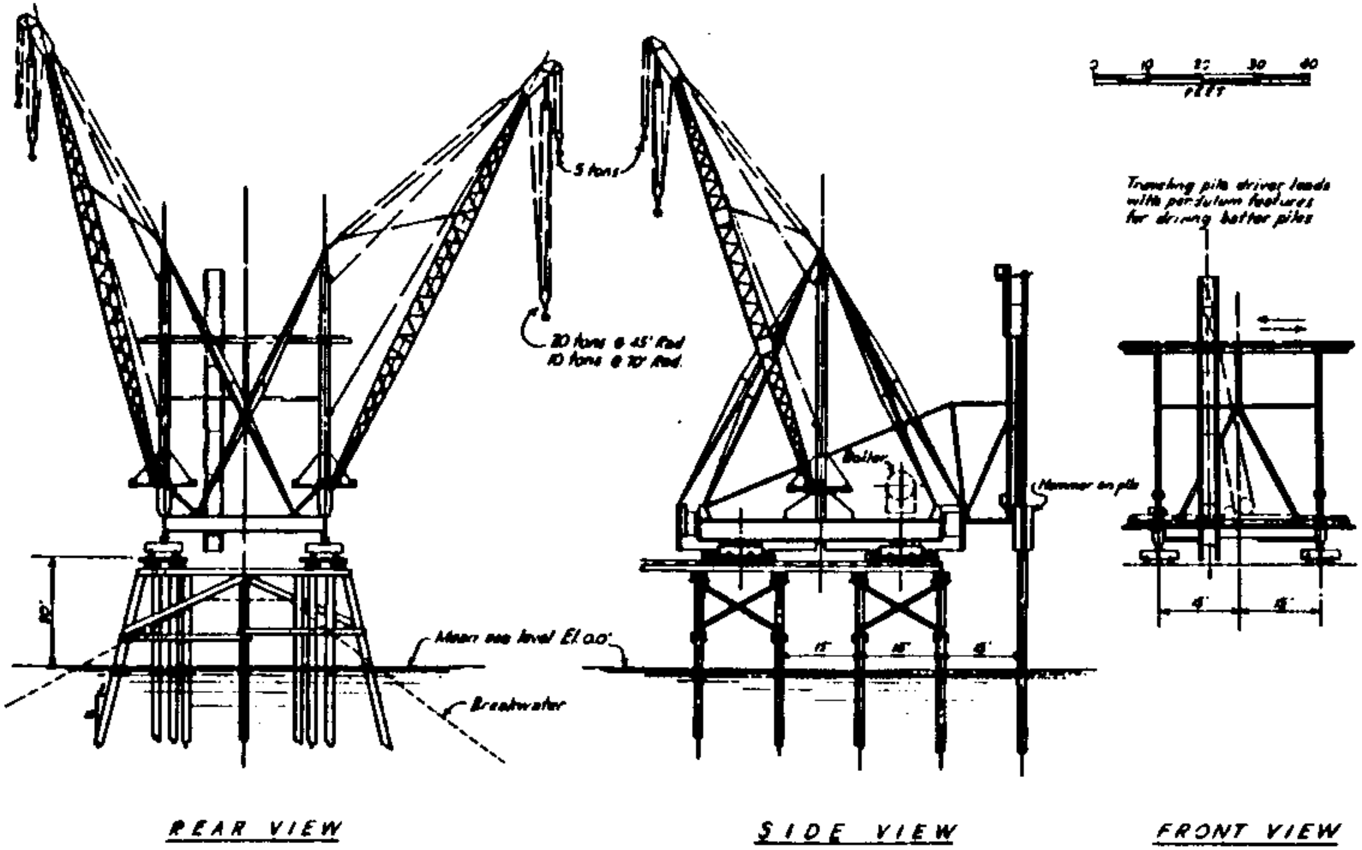


Figure 5.1: Typical Cross Section Of Trestle And Travler Used For Construction Of Coco Solo Breakwater (Quinn,1972)

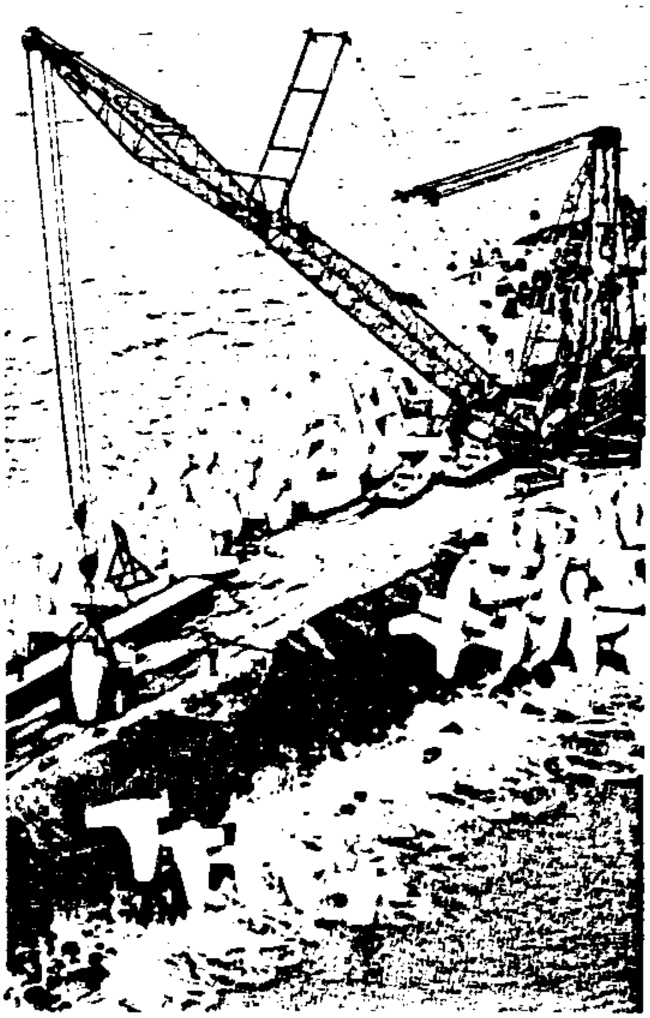


Figure 5.2: Manitowac 4600 Vicon Ringer Crane Placing A 42Ton Dolos At Humboldt Bay, California

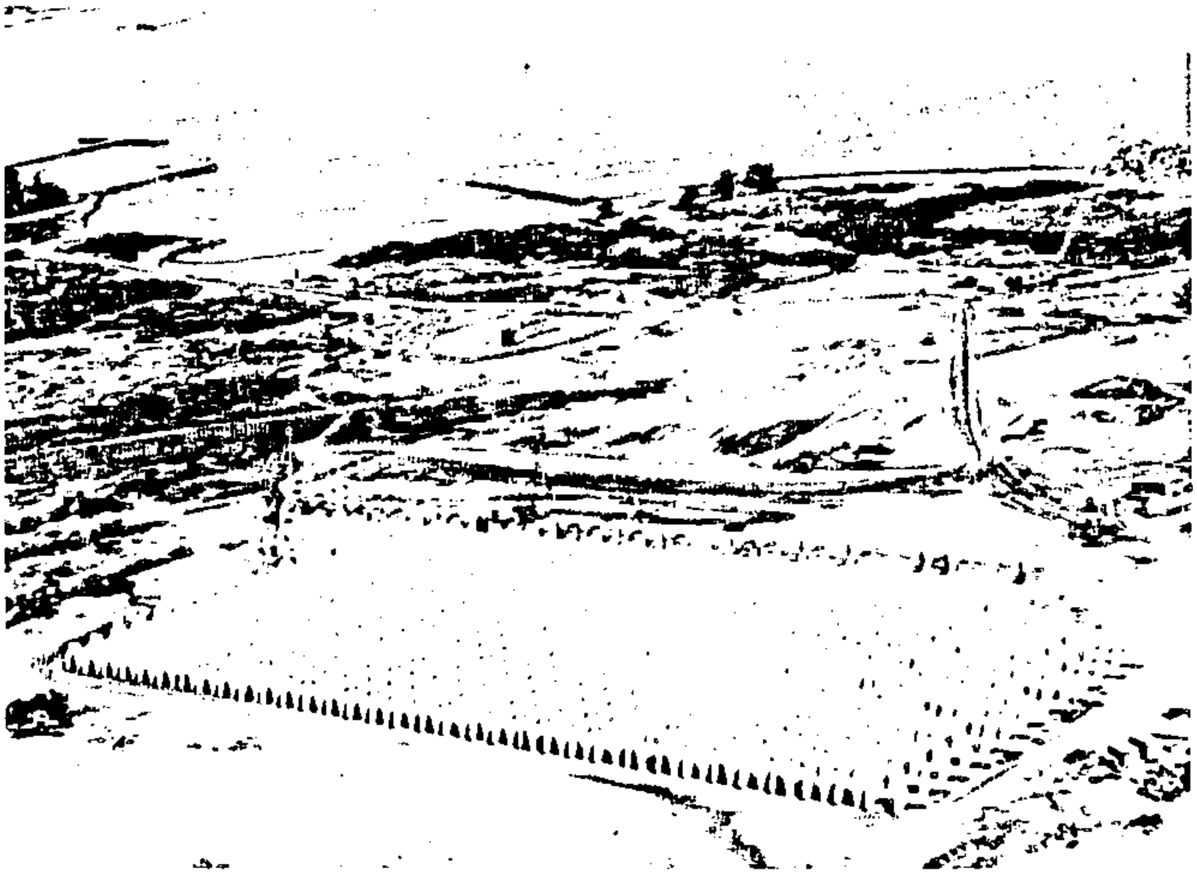


Figure 5.3: The Storage Yard For The Protective Huts.

In the early years of the seventies, the offshore oil industry increased tremendously and construction of the facilities has progressed to deeper water and more exposed locations. New modern and sophisticated equipment was developed for offshore operations, much of which is applicable to breakwater construction.

A number of major breakwaters were constructed off the coast of Southern California in the 1960 - 1970 decade, with the majority of the work being carried out by floating equipment. Initial core rock placement was by bottomdump barges, extending as close to sea level as these barges could operate (about-8'). Then the remaining core and armor rock was placed by a long boom derrick barge, using a large clamshell bucket for the remaining core rock and Class B rock, and rock tongs or slings for the Class A rock. The derrick barge was positioned in the lee of the already constructed breakwater dike, so as to enable work to continue even in moderate sea states.

Since the construction of breakwaters involves placing large quantities of core material, a large fleet of high-powered tug boats and self un-loading barges are required. The Dutch, in particular, have developed such vehicles which they call "omnibarges". They are center hinged, hydraulically controlled, self-dumping barges which carry up to about 900 cubic yards of material with a loaded draft of about 11 feet. The next figures(5.4 and 5.5) show a fleet of barges and tug boats heading towards the construction site while another barge is being loaded at the shore.

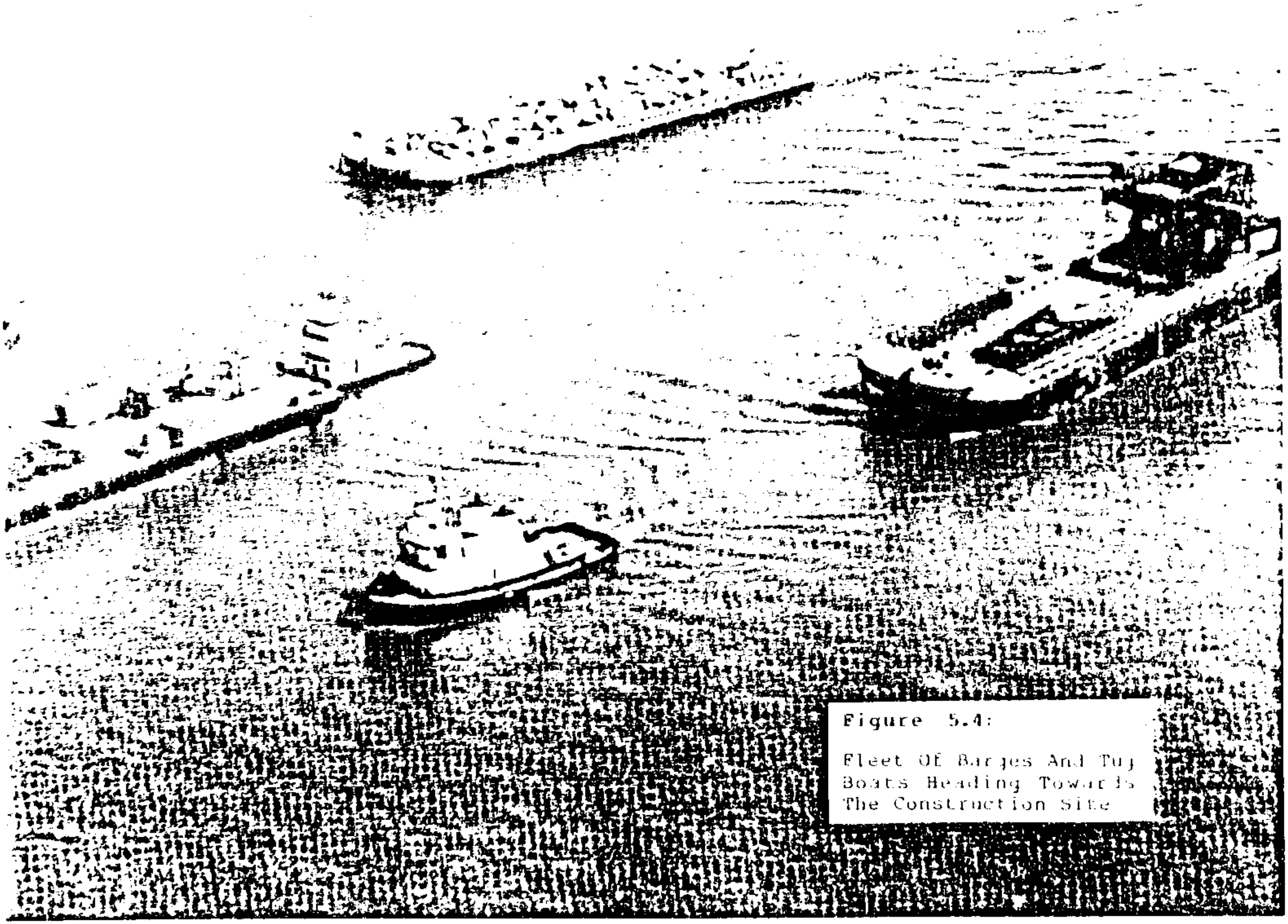


Figure 5.4:

Fleet Of Barges And Tug Boats Heading Towards The Construction Site

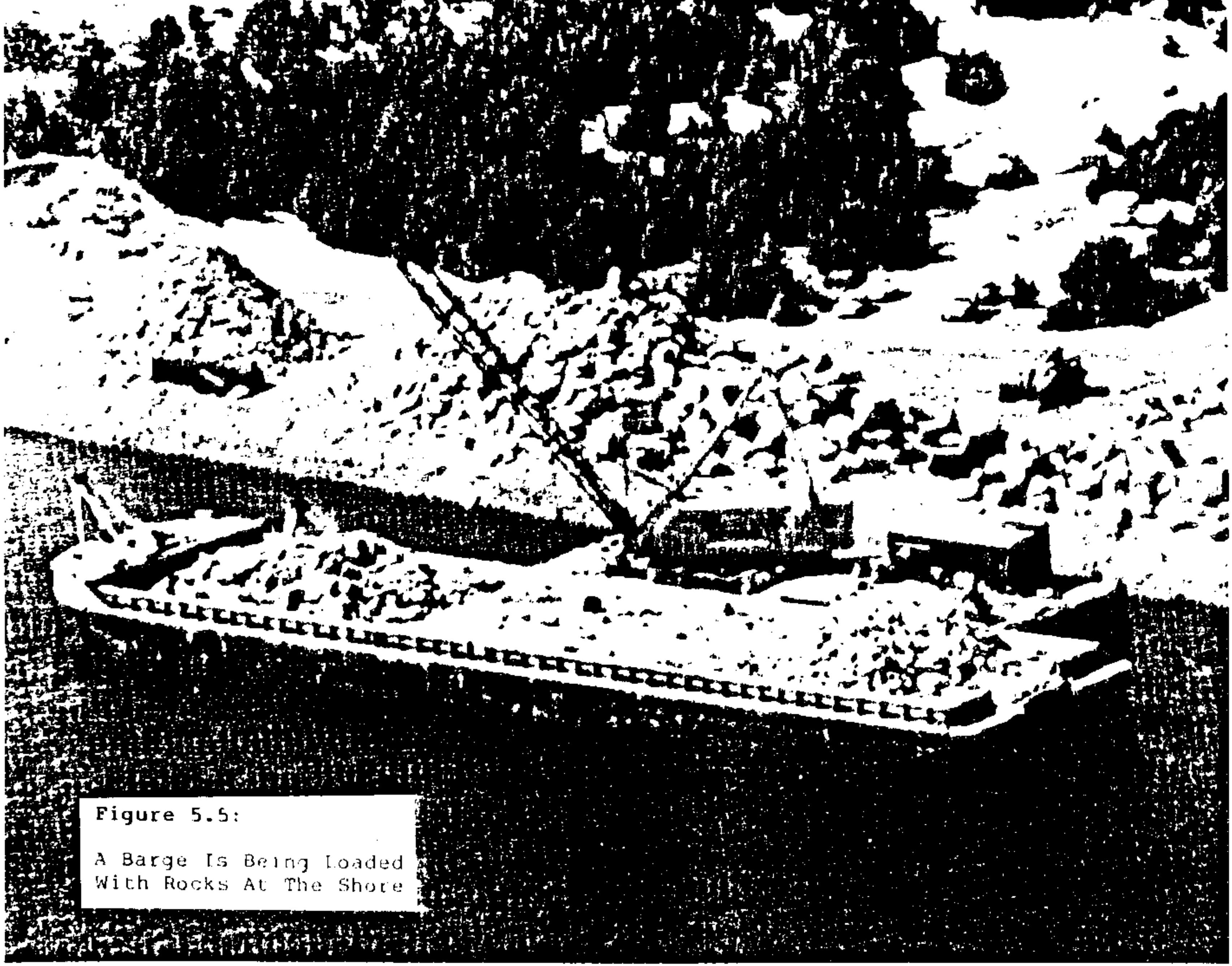


Figure 5.5:

A Barge Is Being Loaded With Rocks At The Shore

Using a clamshell bucket or skip to place the core material in deep water is a classical way to minimize segregtion and to confine the material within a fairly limited area. The disadvantage of the method is the high cost and the time consumed during the construction operations. Equipment cycle time is a problem in deep water. The clamshell or skip must be lowered and raised the full distance, from the barge on the surface to the core on the bottom, each time a load is deposited, and depending on cable reel speed, operator proficiency, and water depth, cycle times of several minutes per load may arise. Since construction operations in a marine environment are always more expensive than similar operations on land, anything which lengthens the time spent on location is undesirable.

For breakwaters extended out from the shore, Method No. 1, using trucks to deliver the rock out over the partially-completed breakwater is often the most practicable and economical means of construction. Core rock is placed by end-dumping trucks (Figure 5.6 and 5.7). In order to provide sufficient width for truck operation, the crest may be temporarily lowered to a few feet above high tide, where the cross-sectional width is greater. Small size fragments are placed to form a road surface. If necessary the core cross-section may be widened as well. Trucks run out on the completed core, headed seaward, then must turn around to dump. So a turn-table may be installed at intervals.

Widened passing zones may be required.

The Class B rock is also delivered by truck and often placed by a crane using a skip or clamshell bucket. Then the Class A rock is delivered and placed by crane, usually using slings or drilled-in lifting bolts and a single sling (Figure 5.8 and 5.9).

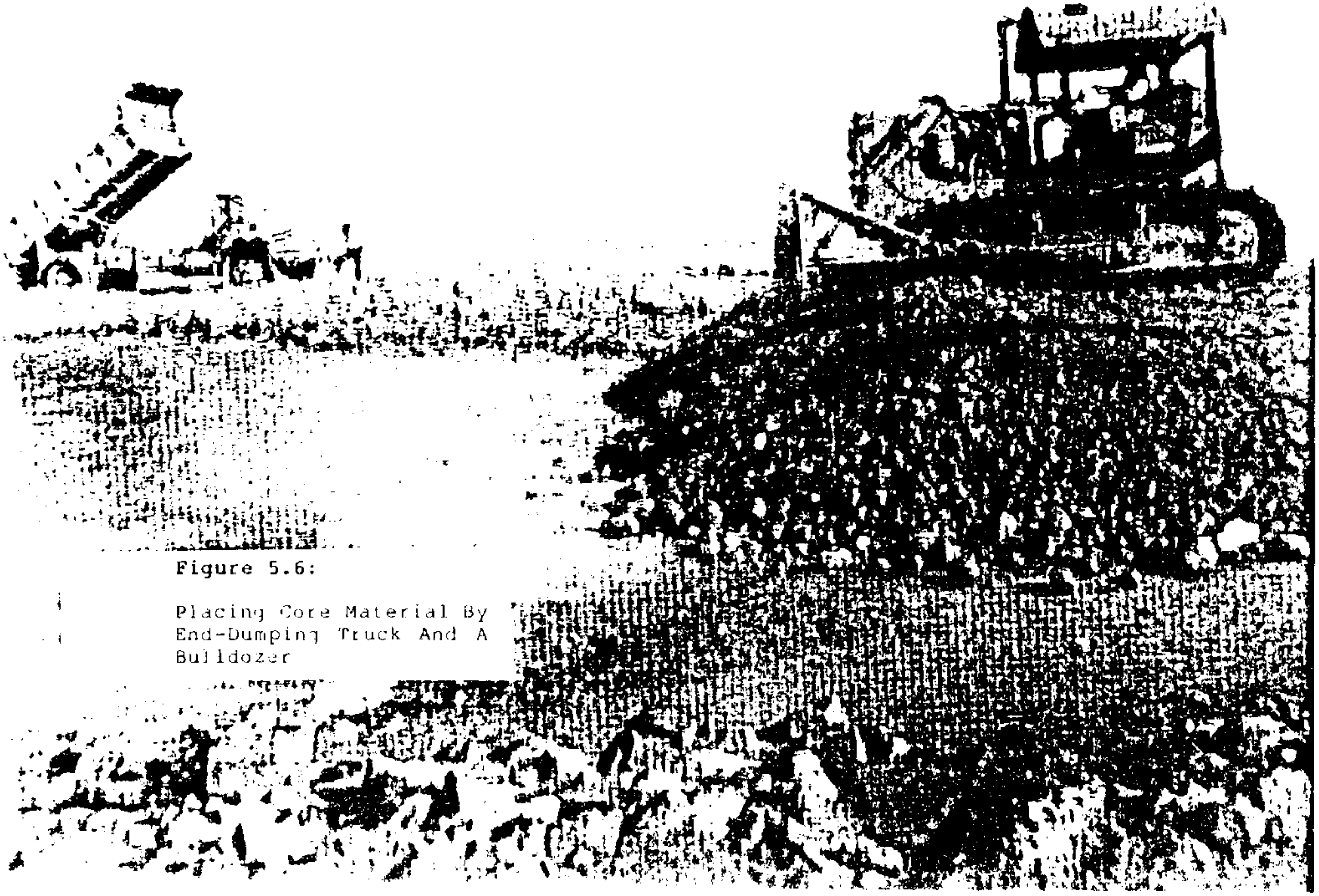


Figure 5.6:

Placing Core Material By End-Dumping Truck And A Bulldozer

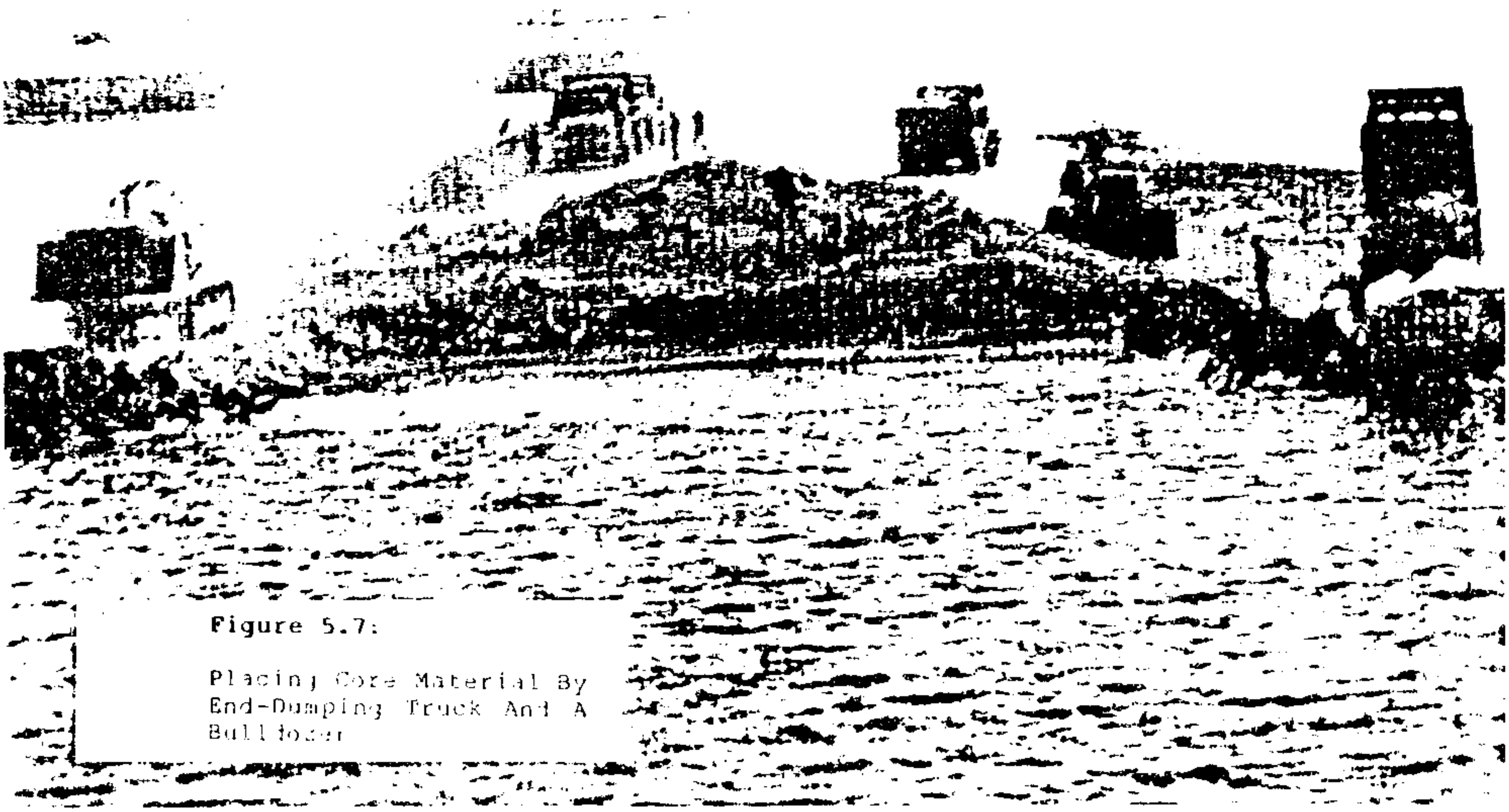


Figure 5.7:

Placing Core Material By End-Dumping Truck And A Bulldozer

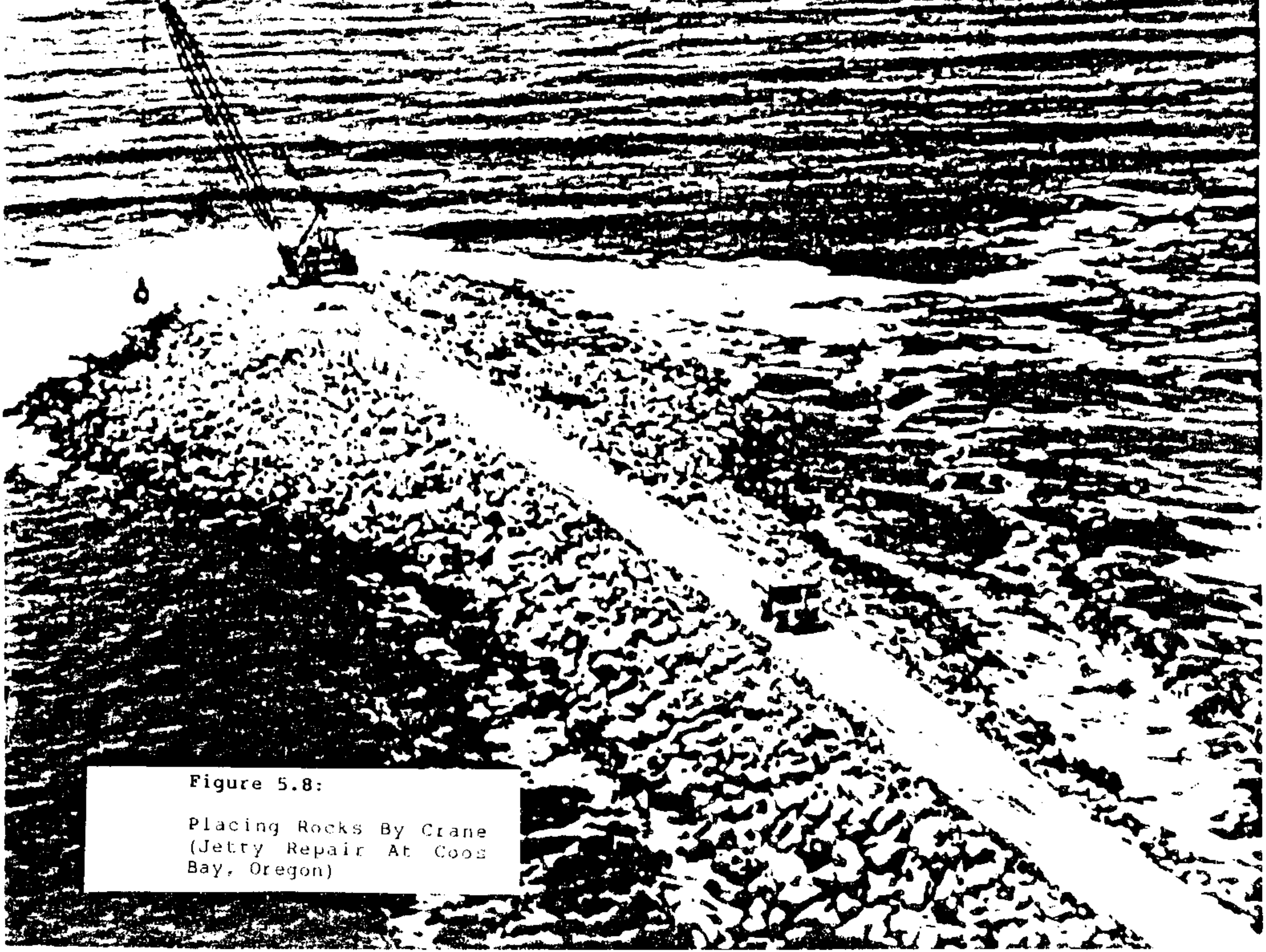


Figure 5.8:

Placing Rocks By Crane (Jetty Repair At Coos Bay, Oregon)

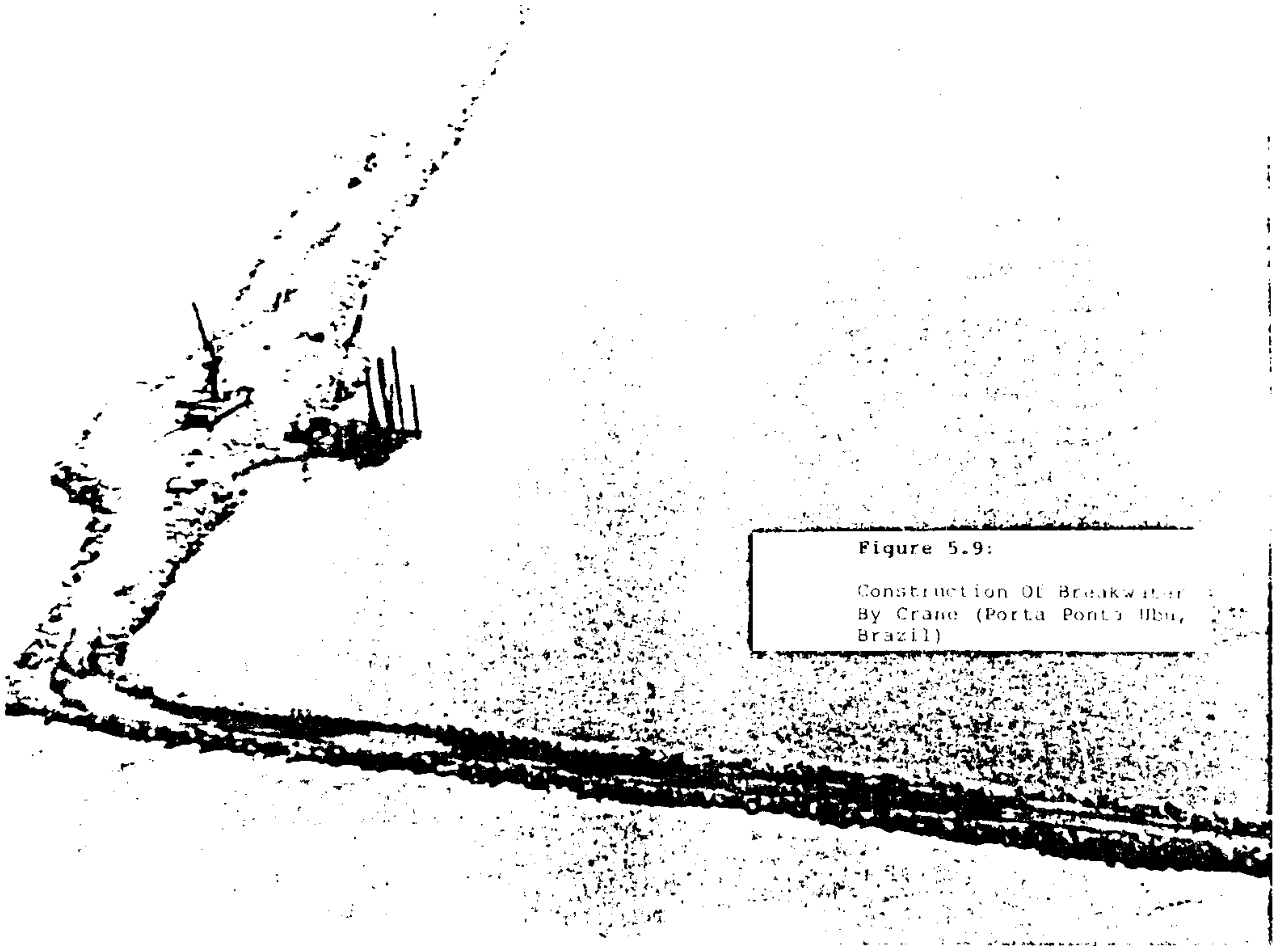


Figure 5.9:

Construction Of Breakwater By Crane (Porta Ponta Ubu, Brazil)

To gain the necessary capacity and reach, the crane must be very large: typically a Manitowoc Ringer Crane (nominal 200 ton capacities) is used, which enables it to place 20 ton rock at a radius of 60 to 80 feet. By this means the breakwater is constructed to full cross-section, except for that portion above high tide (**Appendix F** represents a schematic of Manitowoc Ringer Crane showing its range and capacity).

Specifications and good practice usually require the Class B rock on the seaward face to be not more than 100-200 feet behind the core at any time, and Class C rock not more than 300 feet behind the Class B.

When the entire length of breakwater has been so constructed, then the top crest is progressively completed, working from the outer end towards shore, with the large crane now mounted on crawlers, backing up as each portion is topped out.

MODEL TESTING OF THE PLACEMENT OF THE CORE OF RUBBLE MOUND BREAKWATERS

In order to fully understand the mechanism of the placement of rock in the water model tests were carried out of the dumping of rock in a tank at a scale of 1:20.

The objective of this model testing is to present an understanding of the results obtained in placing the core of rubble mound breakwaters. This will enable an evaluation of these methods to determine the best possible way to place the core material with minimum segregation and waste.

A review of the considerable literature concerning model testing on Breakwaters reveals that model studies fall into two broad categories:

1. The hydrodynamic model to study wave action and energy dissipation.

2. The structural model, where the focus is on the strength of the individual unit.

Although great progress has been made during the last decade in these two categories there have been very few attempts to study the actual physical placement of these rocks. This is due to the complexity of the matter and the difficulty to simulate the construction procedure in a laboratory. However, similar tests have been carried out by Dutch engineers during their planning for specific projects of the Delta Plan.

In practice, the core rock material are transported either by:

1. Trucks, (onshore operation) and placed by end or side dumping.
2. Being placed or dumped from floating scows.

The use of floating craft (in calm water) in construction has the advantages of flexibility but its use may frequently be interrupted by adverse weather. Cranes mounted on Jack-up barges are useful in deep water but are expensive. Also there is the danger of damaging the Jack-up legs due to rock falling against them.

The construction procedure of placing the core rock material in practice is like trying to reach a target blind-folded. The water is very cloudy and murky and the contractor is working in a very difficult environment. Studying the behaviour and pattern of movement of rocks in a calm water (like a lake) is easy to model. But in most cases, real breakwaters are built in water with currents that are changing directions and speed with time and also with waves and swell that move in a multidirectional pattern and with different heights and intensities. These problems combine to make the contractor's work very difficult. Needless to say that the placement of the core is very important in the construction of the breakwater since the core is the foundation of the overall structure.

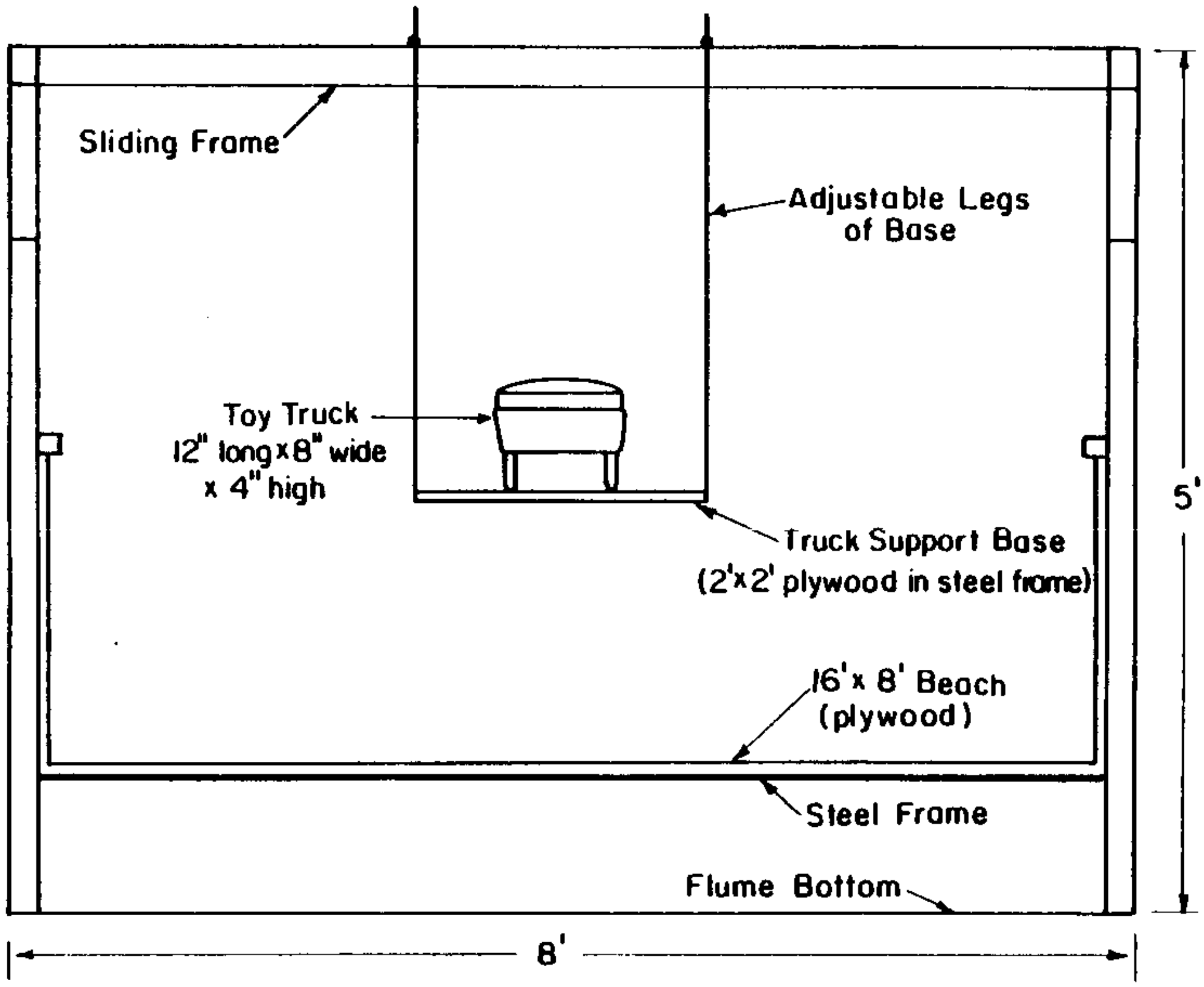
Important effects can occur due to model construction procedures and scale effects. Not all these could be overcome in the model tests, due to practicable constraints. This study will attempt to clarify some of the important patterns that occur during the placement of the core of breakwater. The model scale chosen was based on several factors:

a. The available facilities

b. A greater Reynolds number than 10^5 will be maintained to avoid any viscous scale effects. According to J. Ploeg (1981) rubble mound breakwater model tests on a scale of 1:20 or less present no viscous effects of importance.

c. The purpose of these tests are mainly to give an indication or comparison of the pattern and behaviour of rocks placed in different conditions.

The model scale chosen for this test is 1:20. The flume used is 160 feet long by 8 feet wide and 5 feet in depth. A simulated beach is made out of plywood, resting on a steel frame to permit varying the depth. A mechanical wave generator and a pump to produce current at different speeds are used in these tests. To simulate the actual placement of rocks, a frame of steel was constructed to provide a platform on which a toy dump truck (figure 5.10)scaled to 1:20 was operated.



CROSS-SECTION OF THE 160'x8'x5' FLUME

Figure 5.10

Serious scale effects can be avoided by distorting the shapes and/or the size of the core material. A Reynolds Number can be calculated for the flow in the core:

$$Re = \frac{g^{1/2} \quad H^{1/2} \quad L}{V}$$

g = gravity

H = wave height

V = viscosity

L = length parameter

$L = K \quad (^{W}/_{Y})^{1/3}$

K = Shape coefficient (usually 1)

W = Weight of armor units

Y = Specific weight of armor units

V Kinametic viscosity of water at 20°C = $1.003 \times 10^{-6}\ m^{2}/_{s}$

g Acceleration of gravity = $9.81\ m/s^2$

H .0762 m

$$L = K \quad \frac{W}{V}^{1/3}$$

$$= 1 \quad \frac{(20.1)gm^{1/3}}{.00001639\ m^3} = (1226357.54)^{1/3} = 107.04\ Kg/m^3$$

$$Re = \frac{(9.81)^{1/2}(.0762)^{1/2}(107.04)}{1.003 \times 10^{-6}} = \frac{3.13 \times .276 \times 107.04}{1.003 \times 10^{-6}}$$

$= 92.25 \times 10^6$ which is greater to 10^5

TEST PROCEDURE

The back of the toy truck is tied with a rope that will lift the back by pulling on it. Each load is 10 lb. in the model which will be equivalent to :

$$\text{Specific Weight} = \frac{\text{Weight}}{\text{Volume}} \quad \text{in the model}$$

$$= \frac{10 \text{ lb}}{12"\text{x}8"\text{x}4"} = \frac{10 \text{ lb}}{384 \text{ Cu. In.}}$$

$$\text{Specific Weight} = \frac{\text{Weight}}{\text{Volume}} \quad \text{in the Prototype}$$

$$= \frac{W}{384(20)^3} = .026$$

$$W = 39.936 \text{ or } 40 \text{ tons}$$

$$1 \text{ ton} = 2000 \text{ lb (1 short ton)}$$

SIEVE AND WASH ANALYSIS

U.S. STANDARD SIEVE NO.	WEIGHT RETAINED	CUMULATIVE WT. RETAINED	PERCENT RETAINED	PERCENT PASSING
1 1/2 in.	841.63	841.63	3.75	96.25
1 in.	1120.13	1961.76	8.74	91.26
3/4 in.	5443.11	7404.87	32.98	67.02
1/2 in.	5702.98	13107.85	58.38	41.62
3/8 in.	5062.41	18170.26	80.93	19.07
4	2393.31	20563.57	91.59	8.41
8	962.67	21526.24	95.88	4.12
28	517.48	22043.72	98.18	1.82
35	371.31	22415.03	99.84	.16
100	36.73	224151.76	100.00	0.0

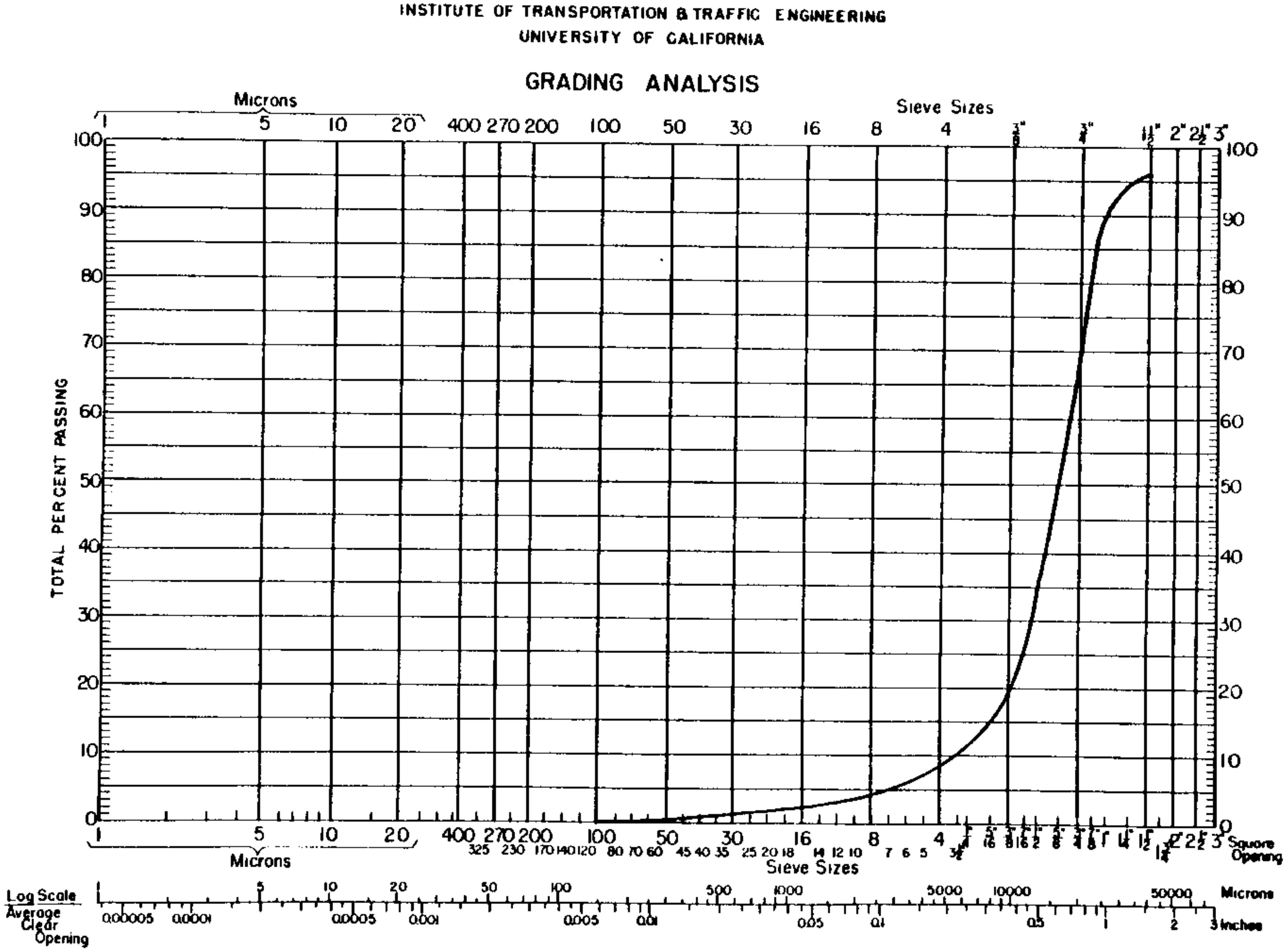


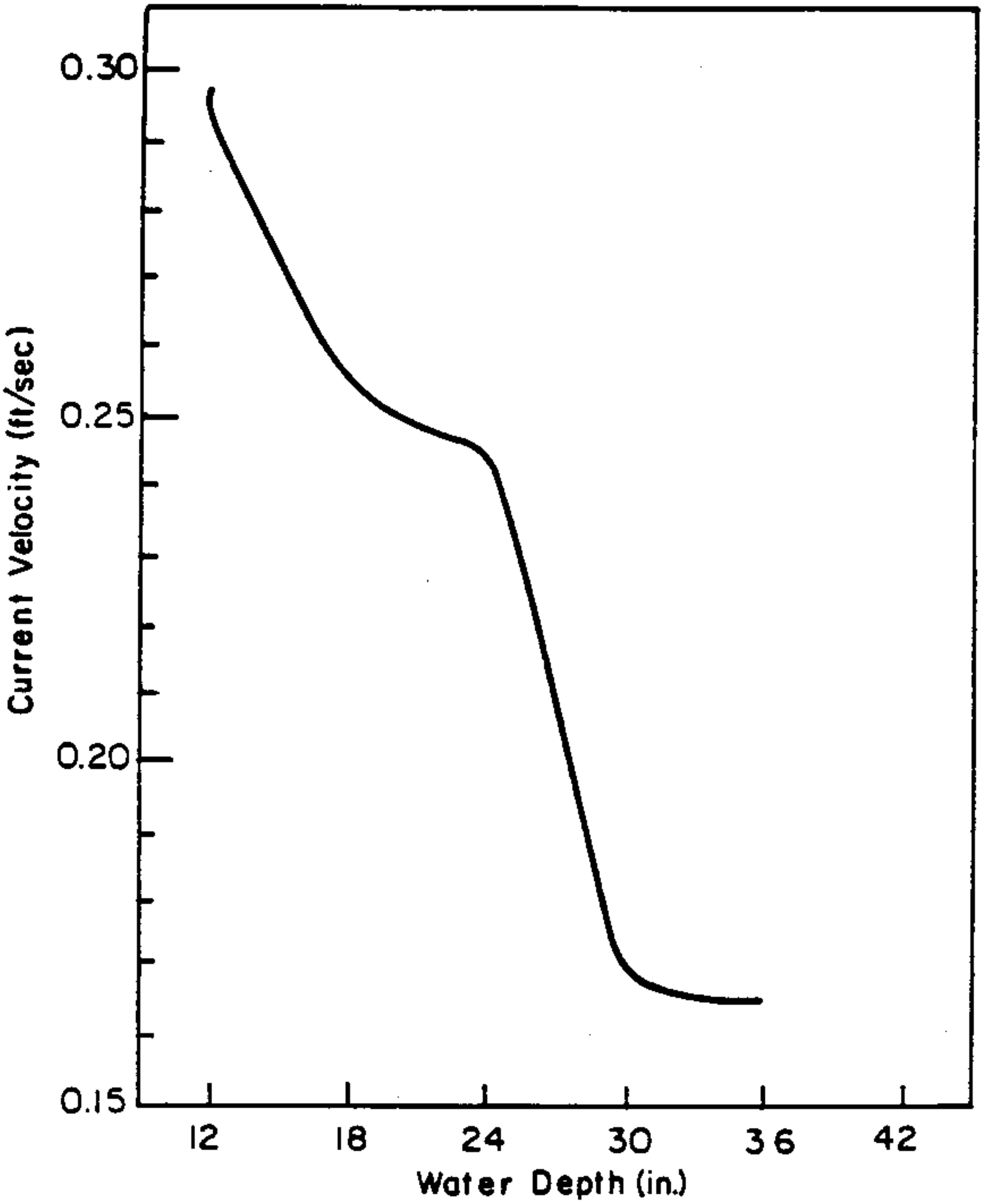
Figure 5.11: Grading Analysis For The Sample Used In The Test

CURRENT VELOCITY IN THE FLUME

WATER DEPTH	DEPTH OF PROBE FROM BOTTOM	frequency f	SPEED S=Ø.1 + Ø.Ø18f
12"	7 1/2	9.7	
		13.7	
		11.Ø	Av. f = 1Ø.97
		1Ø.Ø	
		8.9	S = Ø.1+Ø.Ø18 X 1Ø.97
		1Ø.3	= .2974 ft/s
		12.4	
		9.9	
		11.2	
		12.6	
18"	11 1/4	6.3	
		8.6	Av. f = 8.62
		9.9	
		8.5	S = Ø.1+Ø.Ø18 x 8.62
		9.3	= .25516 ft/s
		9.5	
		7.9	
		7.8	
		9.3	
		9.1	
24"	15"	6.9	
		2.5	Av. f = 8.Ø2
		6.Ø	
		13.Ø	S = Ø.1+Ø.Ø18 x 8.Ø2
		8.9	= .24436 ft/s
		7.9	
		1Ø.3	
		12.4	
		6.2	
		6.1	

CURRENT VELOCITY IN THE FLUME

WATER DEPTH	DEPTH OF PROBE FROM BOTTOM	frequency f	SPEED S=Ø.1 + Ø.Ø18f
3Ø"	18 3/4	5.6	
		2.5	Av. f = 3.86
		5.4	
		3.5	S = Ø.1+Ø.Ø18 f
		4.Ø	= .16948 ft/s
		3.Ø	
		4.8	
		3.Ø	
		Ø.5	
		6.3	
36"	22 1/2	4.Ø	
		2.8	
		2.6	AV f = 3.71
		3.7	
		4.3	S = Ø.1+Ø.Ø18 f
		3.7	= .16678 ft/s
		5.2	
		2.5	
		4.1	
		4.2	



AVERAGE CURRENT VELOCITY IN DIFFERENT DEPTHS.
THE CURRENT IS TURBULENT FLOW.

Figure 5.12

These series of tests are conducted in an attempt to simulate the different conditions occuring when the core rock material is placed. The variables in these experiments are as follows:

1. **Different water depths.** The depth of the water in the model varies from 12 inches to 36 inches which is equivalent to 20 feet to 60 feet, which are average water depths for breakwater construction. Currently there are breakwaters constructed in as deep as 130 and 150 feet of water (Sines breakwater, Portugal).

2. **Different saturation conditions.** A dry sample is compared to a wet sample of rock before it is placed in the water. It was postulated that prefilling of the voids with water instead of air might decrease segregation.

3. **Different water conditions.** The samples are placed in calm water (no movement) versus the placement in water where current and mechanical waves are induced. This comparison will illustrate the difference in patterns achieved in each state.

4. **Different seafloor conditions.** Two different types of seafloor were used in the model. A hard surface for the first one is demonstrated by a hardwood plank and a sand box for the second to represent a softer seafloor condition.

The first series of tests shows the different patterns and lay-outs of the rock dumped in 18 inches of water depth (equivalent to 30 feet in the prototype). Three different conditions were used:

1. Dry rock was dumped in calm water
2. Wet rock (sprayed with water), was also dumped in calm water.
3. Dry rock was dumped in water with current of an average speed of 0.25516 feet/second.

RESULTS

1. In the dry sample the pattern was a circle with a diameter of 2 feet, and the fines were scattered outside the circle.

2. In the wet sample, no apparent change from the dry sample except there were less fines in the pattern.

3. In the moving water the shape became different. Instead of the circle an

elliptical pattern was formed and the sample was moved $^{3}/_{4}$ of a foot away from the datum in the direction of the current flow. The dimensions of the ellipse were 2 $^{1}/_{2}$ feet by 2 feet wide. The fines were easily washed away in the direction of the current.

The second set of figures in this series show the same results after dumping another load of 10 lb. in each of the different conditions. In the inside of the circles and elliptical patterns the rocks start to fill up the pattern and few rocks as deposited on the outer perimeter.

The same results occur when the third load of rocks is dumped. This is illustrated in the third series of tests.

The second series of test was conducted in the same pattern as the first series except the depth of water is changed from 18 inches to 24 inches. The same results were observed.

The third and the fourth series of tests were conducted at 30 inches and 36 inches of water depth respectively.

The fifth series of tests was executed at a depth of 36 inches of water. This one represents a comparison between the formation of a mound in a still calm water versus another one in water with current and mechanical waves with an average speed of water at 22 $^1/_2$ inches from the seafloor equal to 0.3 feet/second. The loads were dumped until a mound was formed. The total amount of rock used in each case was 300 lbs.

RESULTS:

1. The shape of the mound formed in the calm water was circular, but the shape of the mound formed in the moving water was an ellipse with the major axis in the direction of the current.

2. The diameter of the circle was 3 $^1/_2$ feet while the dimentions of the ellipse were 2 $^1/_2$ feet by 4 $^3/_4$ feet.

3. The slope angles differ drastically. In the case of calm water the angle is from 35^o to 42^o and in the case of moving water the slope angle is between 15^o to 24^o.

The sixth series of tests was carried out to show the difference between the pattern formed when dumped on soft material versus hard seafloor. The first sample of rock was dumped onto a hard wood " beach" at a depth of 24 inches in calm water. A second sample was placed from the same height, but into a sand box, having a 2" thick layer of sand.

RESULTS

There was no difference in the pattern formed in either case. The same circle with the same diameter was observed.

The last series of tests was enacted to compare the method of dumping and the rate at which the operator unloads the rocks. The first sample was dumped with one action, while the second sample was dumped with a very controlled slow speed.

RESULTS:

The pattern formed by the rocks dumped in the second sample (slow lifting of the bed of the truck) was a circle with a much smaller diameter.

OBSERVATIONS AND CONCLUSIONS

1. The pattern formed when dumping rocks from scows or trucks is usually a circular ring regardless of seafloor characteristics. If there are waves and currents the circular pattern is elongated into an elliptical form in the direction of the current.

2. This pattern is not affected by the water depth. The variation of the water depths in the model were from 1 foot to 3 feet. This

did not lead to any appreciable difference in the size of the circle or ellipse.

3. There is no apparent difference in the pattern formed when dumping the rocks whether the rocks are dry or wet. It has been reported in field observations that bottom-dump barges in which the rock was pre-saturated (which is not the same as the case in this experiment) will place the rocks in a smaller circular ring than dry loads, due to the adverse effect of the entrapped air in the dry case. Air bubbles were noted in these experiments in both conditions. Further study should be carried out in this area.

4. Continuous dumping to form a mound has proven to be affected by the sea condition. This was demonstrated by the flatter slopes formed when there was a current induced in the water. Many methods were suggested to place the rocks with a steep slope in order to achieve greater economy in material and construction. The use of a clamshell or a dragline to redress the slopes after placement of the rock is another method for obtaining steeper slopes and might prove to

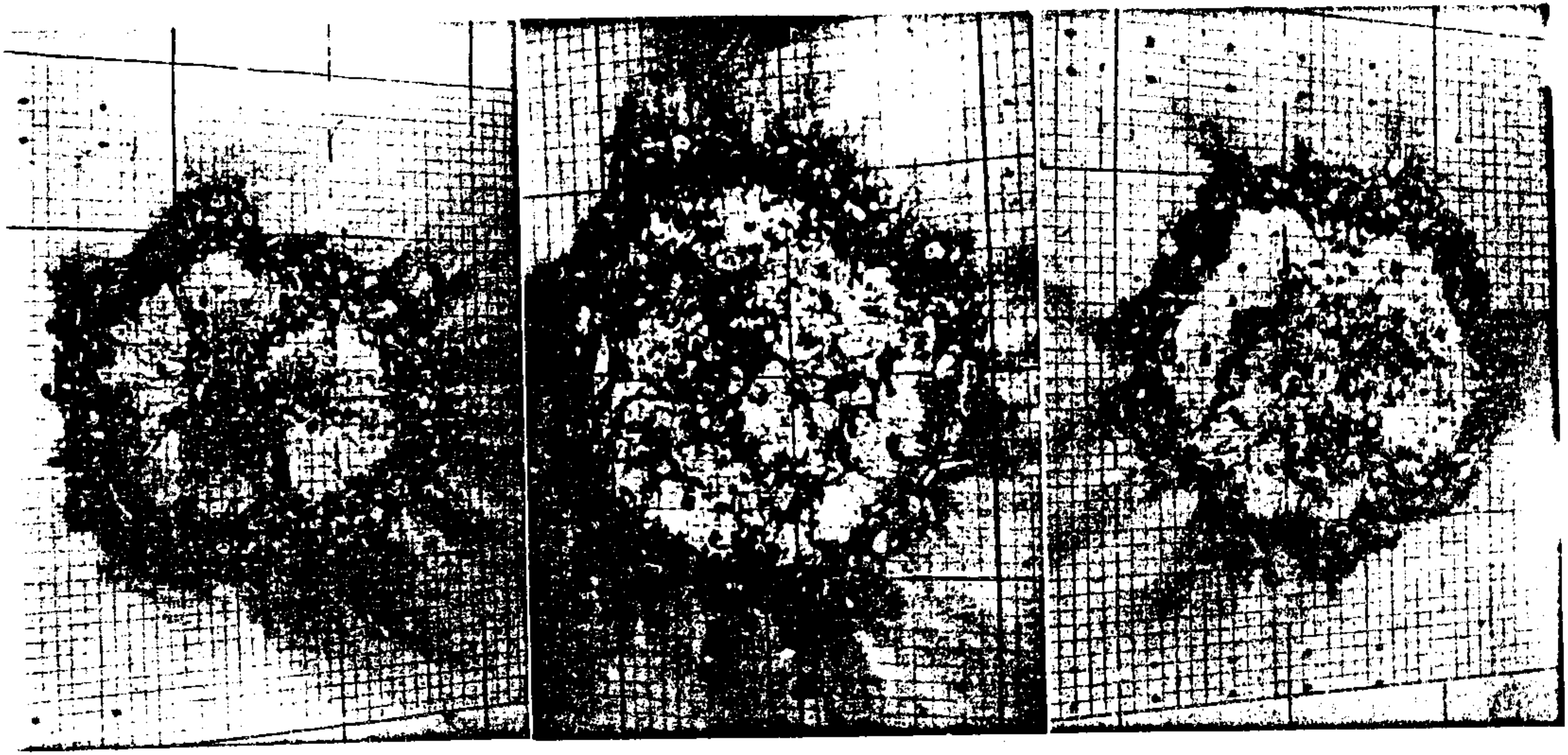
be more economical.

5. Natural segregation is formed when dumping continues, with the coarser material being deposited on the outside. This becomes a natural filter.

6. Extreme fines wash out and disperse radially.

7. Slow dumping (as compared to sudden dumping) has proven more effective in achieving a smaller spread. This is due to the fact that sudden dumping creates a full load as one mass moving in the water, and generating an appreciable added mass of water moving with it. This mass will push the water ahead of it gaining speed until it impacts the seafloor. It's kinetic energy then causes it to spread into a circular ring, the diameter of which depends on the friction of the rocks on the seafloor as well as the drag force in the water until it stops.

The next figures (5.13 thru 5.23) illustrate the patterns formed in these series of experiments.

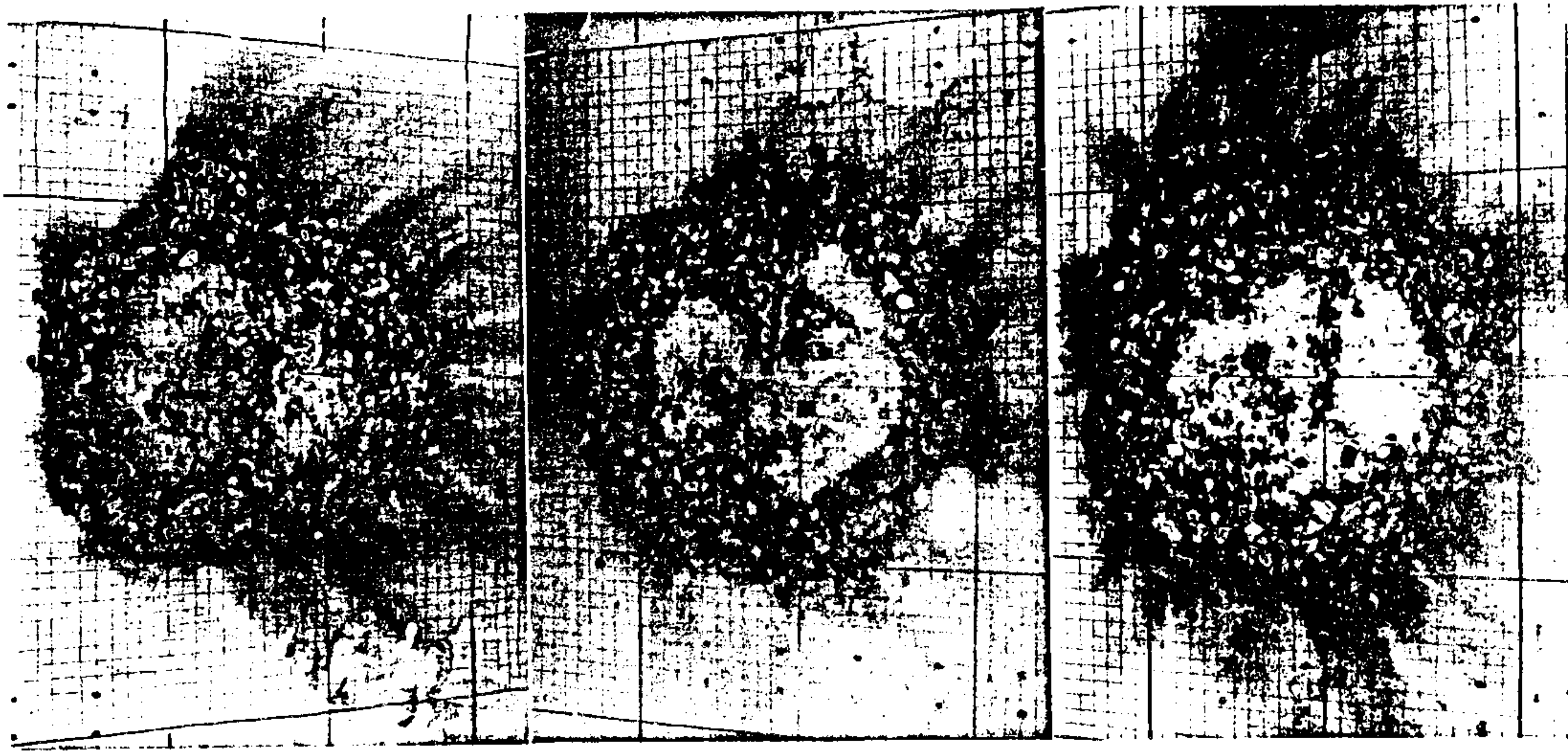


WITH CURRENT   WET SAMPLE   DRY SAMPLE

DEPTH OF WATER 18 IN.   FIRST LOAD DUMPED

THE FIRST SERIES OF TESTS

FIGURE 5.13

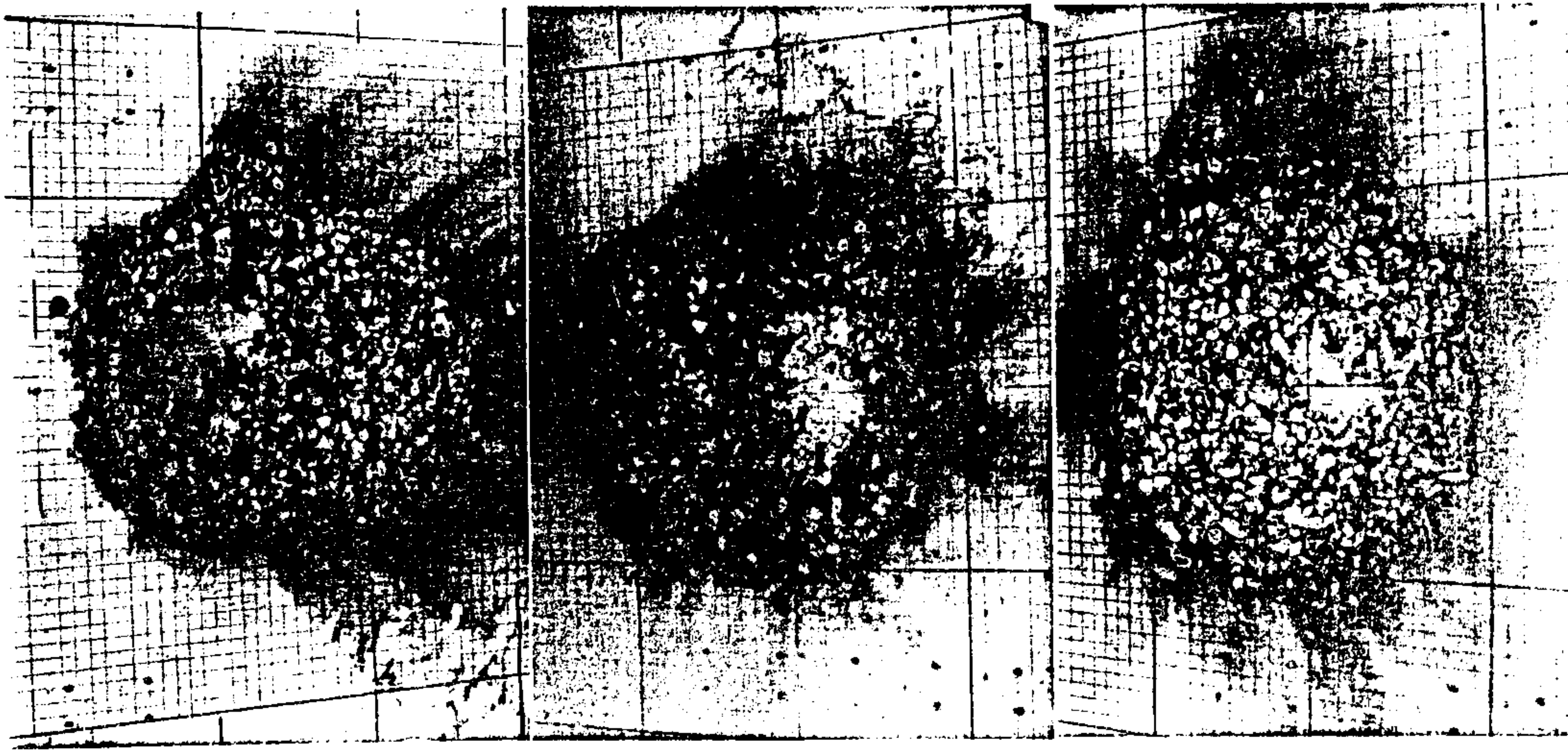


WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 18 IN. SECOND LOAD DUMPED

<u>THE FIRST SERIES OF TESTS</u>

FIGURE 5.14

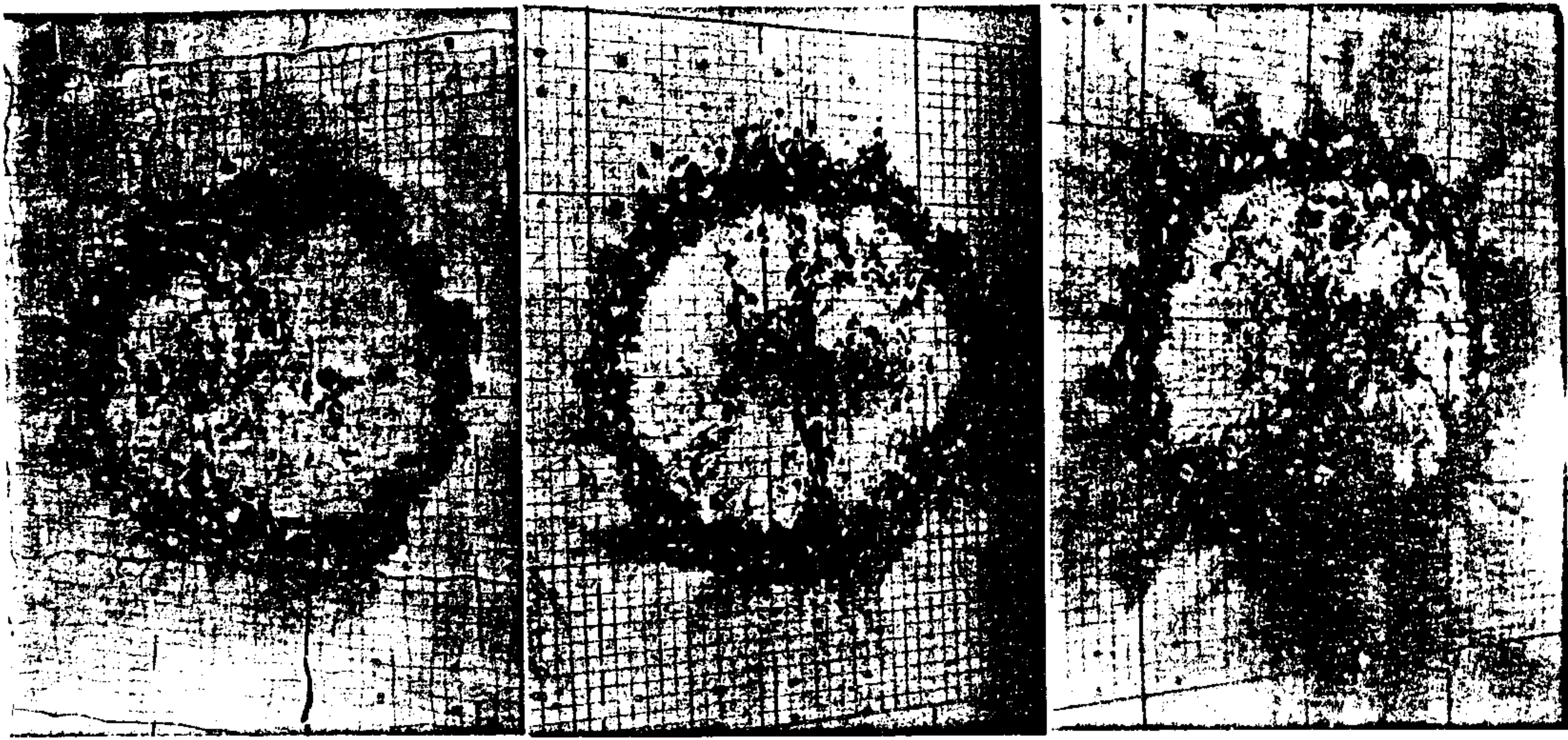


WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 18 IN. THIRD LOAD DUMPED

THE FIRST SERIES OF TESTS

FIGURE 5.15

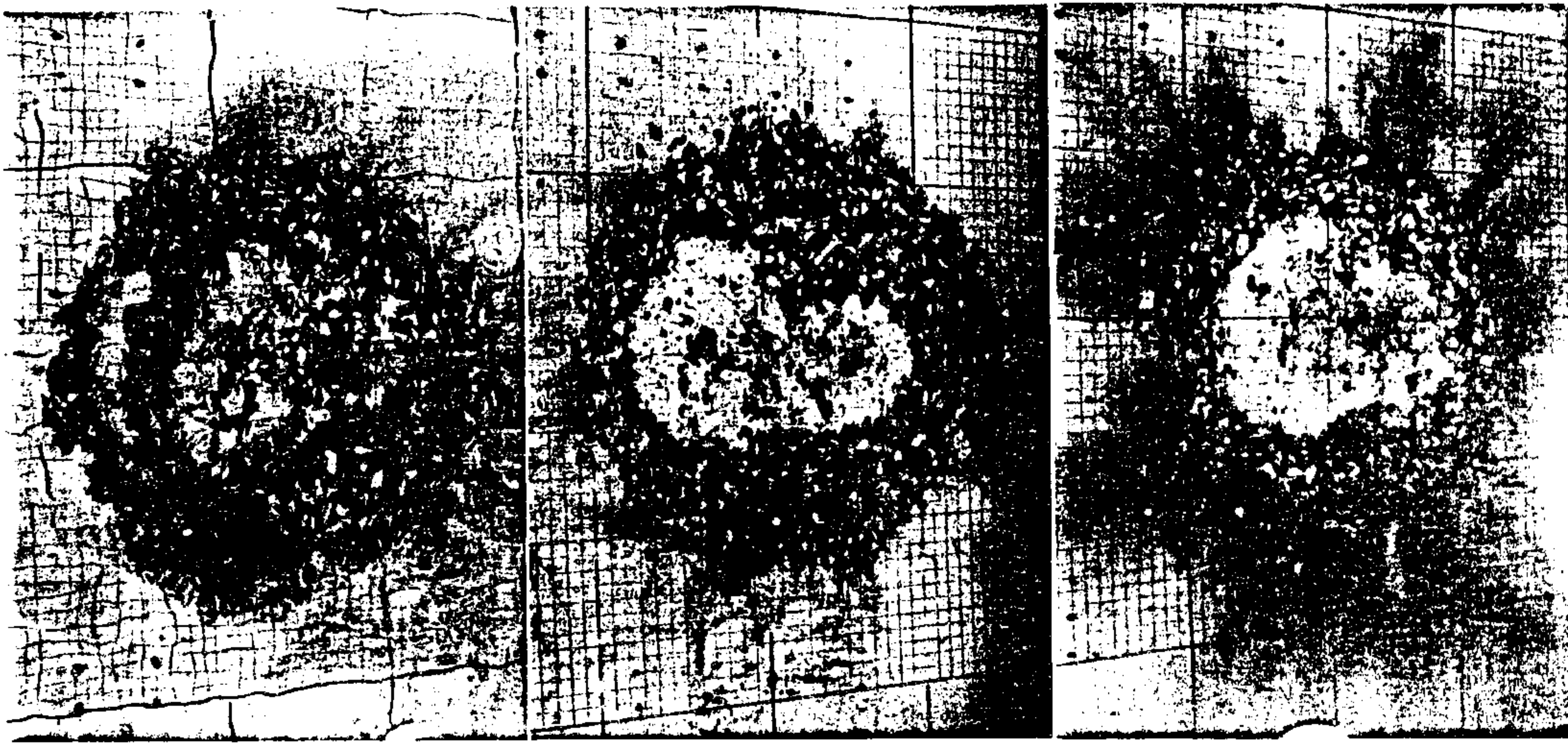


WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 24 IN. FIRST LOAD DUMPED

<u>THE SECOND SERIES OF TESTS</u>

FIGURE 5.16

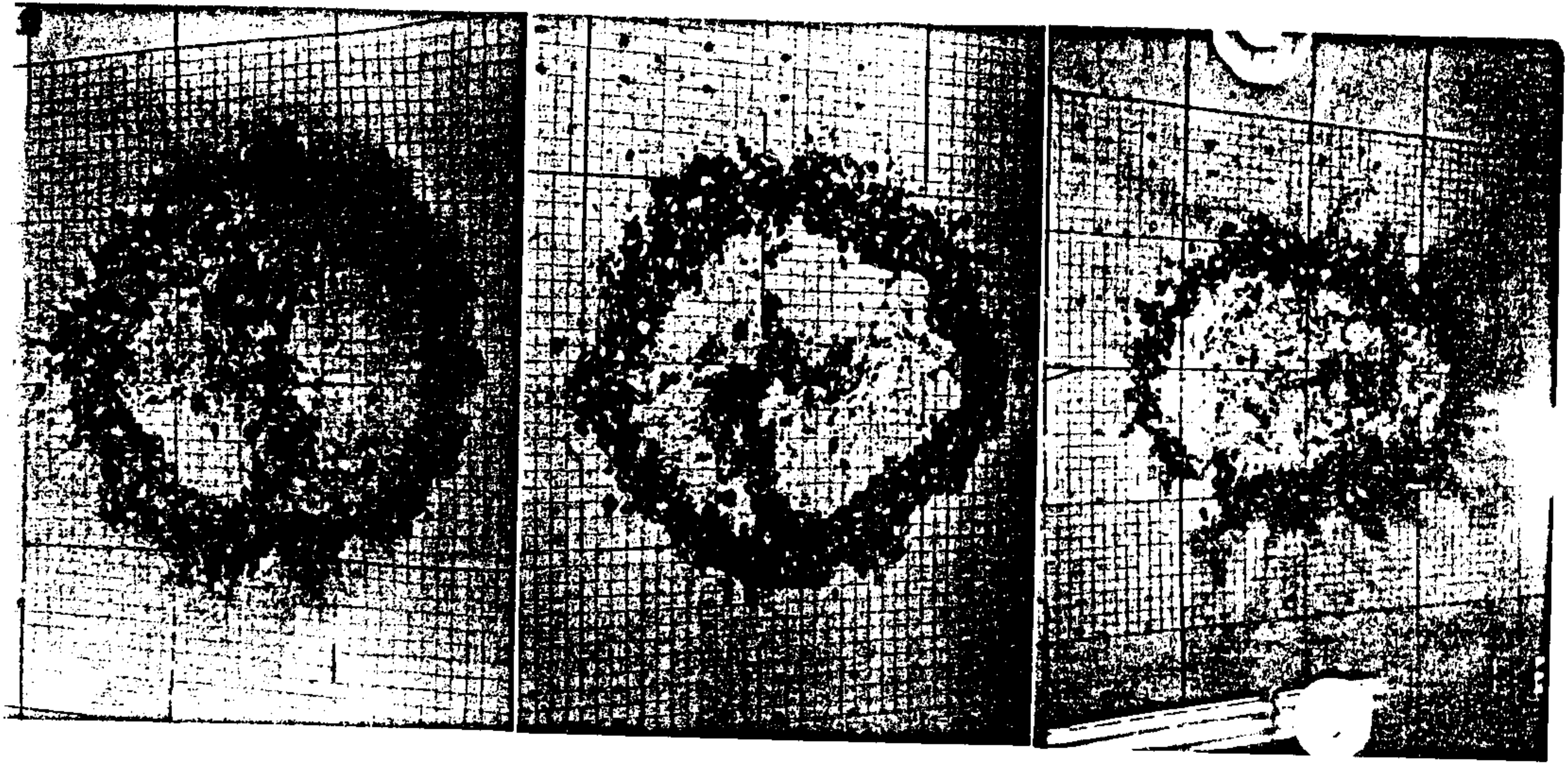


WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 24 IN. SECOND LOAD DUMPED

<u>THE SECOND SERIES OF TESTS</u>

FIGURE 5.17

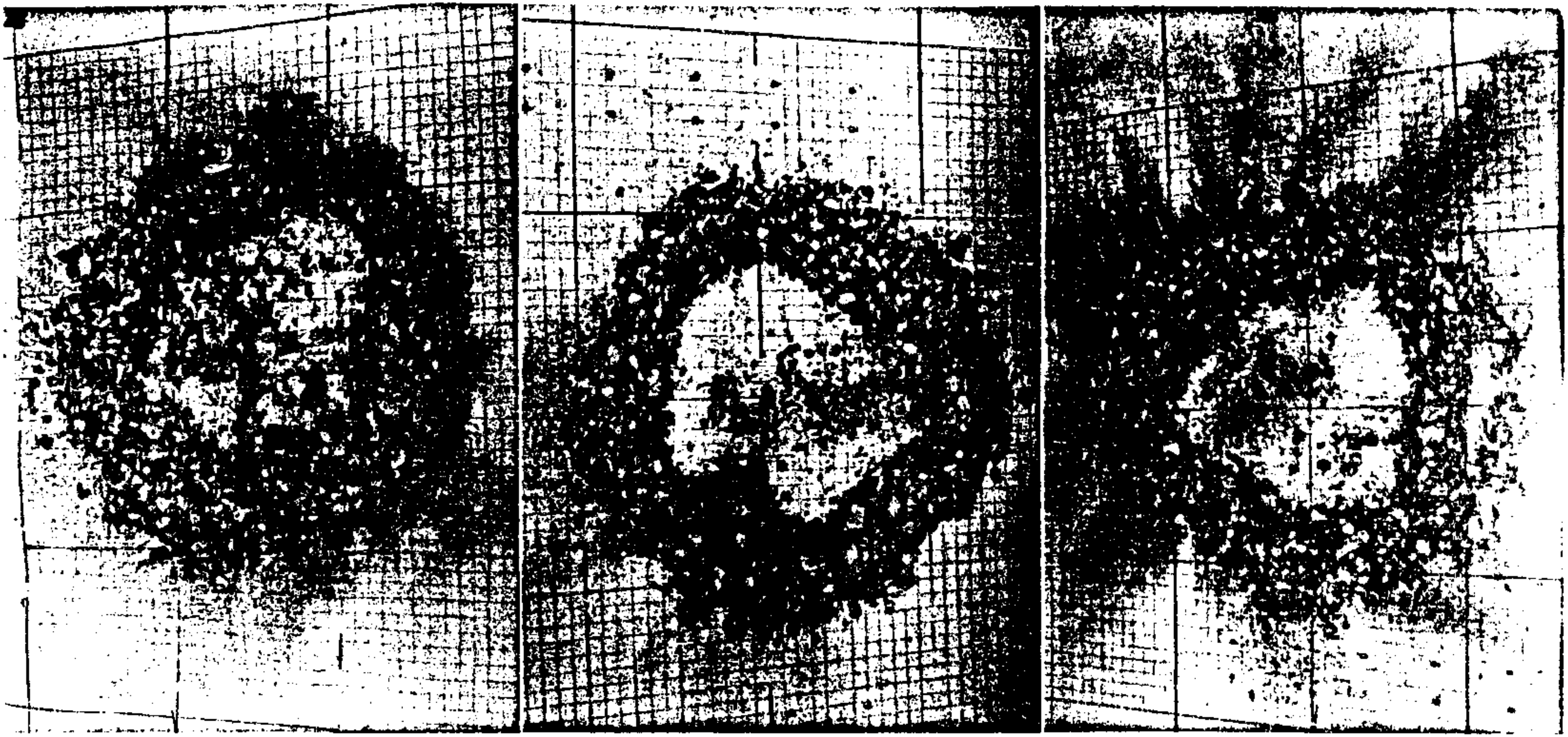


WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 30 IN. FIRST LOAD DUMPED

<u>THE</u> <u>THIRD</u> <u>SERIES</u> <u>OF</u> <u>TESTS</u>

FIGURE 5.18

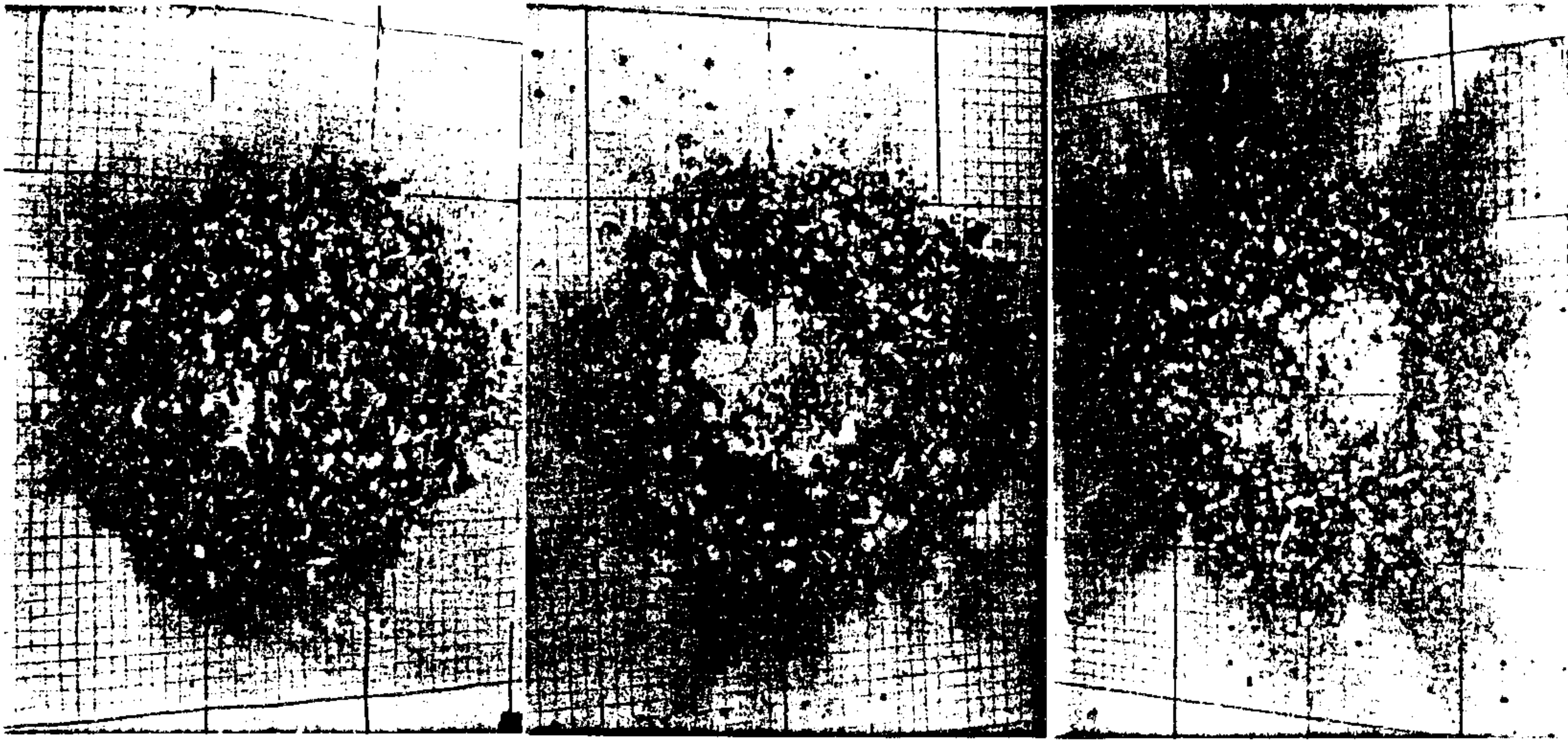


WITH CURRENT　　　　WET SAMPLE　　　　DRY SAMPLE

DEPTH OF WATER 30 IN.　　　　SECOND LOAD DUMPED

THE THIRD SERIES OF TESTS

FIGURE 5.19



WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 30 IN. THIRD LOAD DUMPED

<u>THE THIRD SERIES OF TESTS</u>

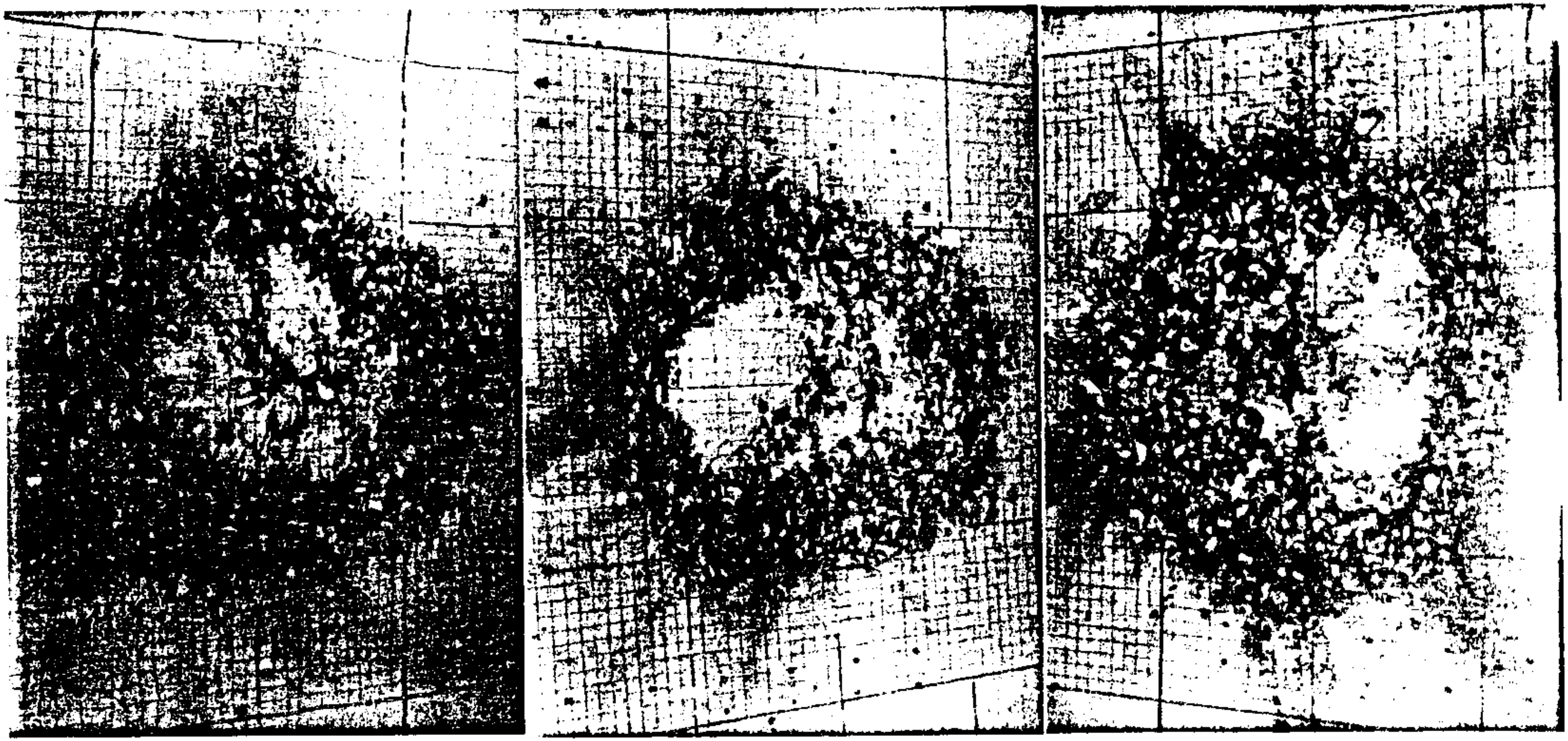
FIGURE 5.20

WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 36 IN. FIRST LOAD DUMPED

<u>THE FOURTH SERIES OF TESTS</u>

FIGURE 5.21

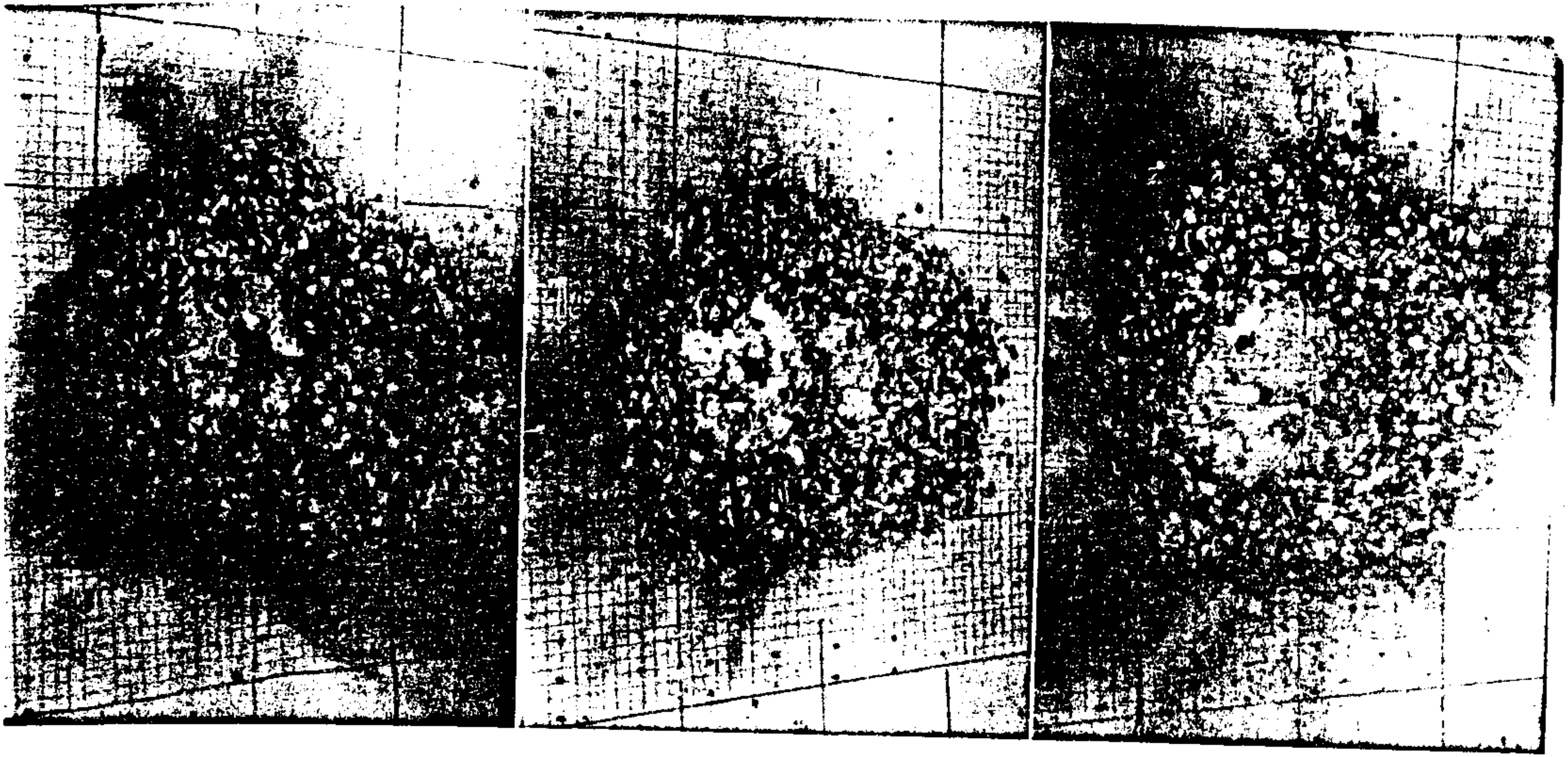


WITH CURRENT — WET SAMPLE — DRY SAMPLE

DEPTH OF WATER 36 IN. — SECOND LOAD DUMPED

<u>THE</u> <u>FOURTH</u> <u>SERIES</u> <u>OF</u> <u>TESTS</u>

FIGURE 5.22



WITH CURRENT WET SAMPLE DRY SAMPLE

DEPTH OF WATER 36 IN. THIRD LOAD DUMPED

<u>THE FOURTH SERIES OF TESTS</u>

FIGURE 5.23

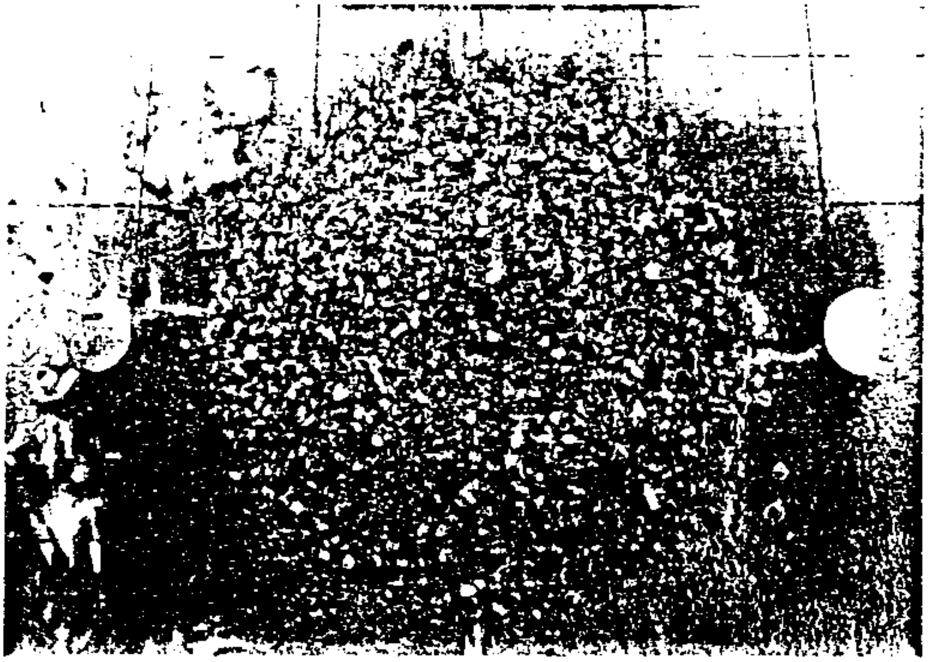


FIGURE 5.24 A MOUND OF CORE ROCK MATERIAL PLACED IN CALM WATER

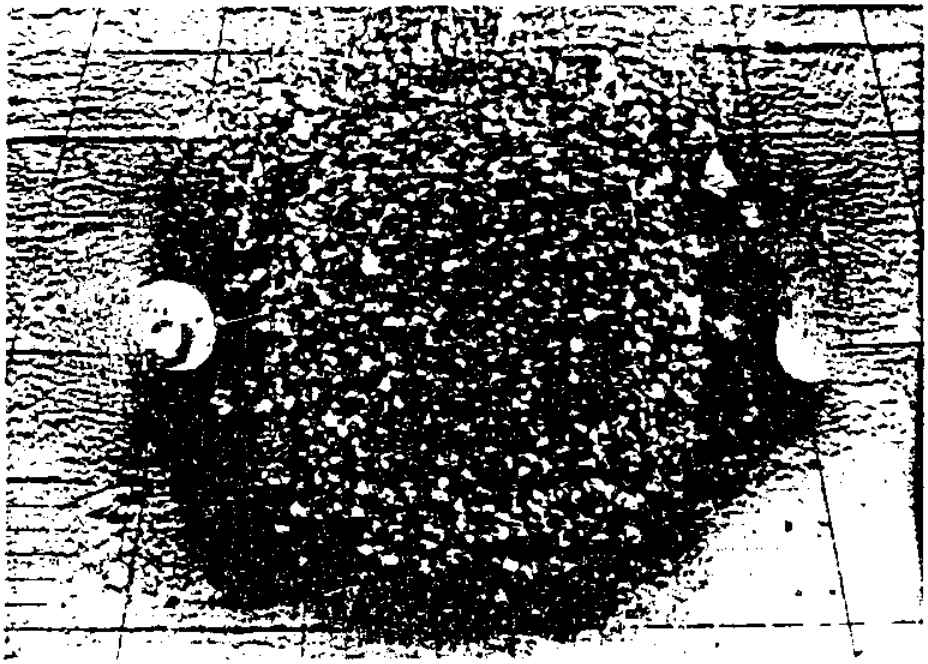


FIGURE 5.25 A MOUND OF CORE ROCK MATERIAL PLACED IN WATER WITH CURRENT AND MECHANICAL WAVES

THE FIFTH SERIES OF TESTS

A PLAN VIEW OF THE MOUNDS FORMED IN CALM WATER AND WATER WITH CURRENT AND MECHANICAL WAVES

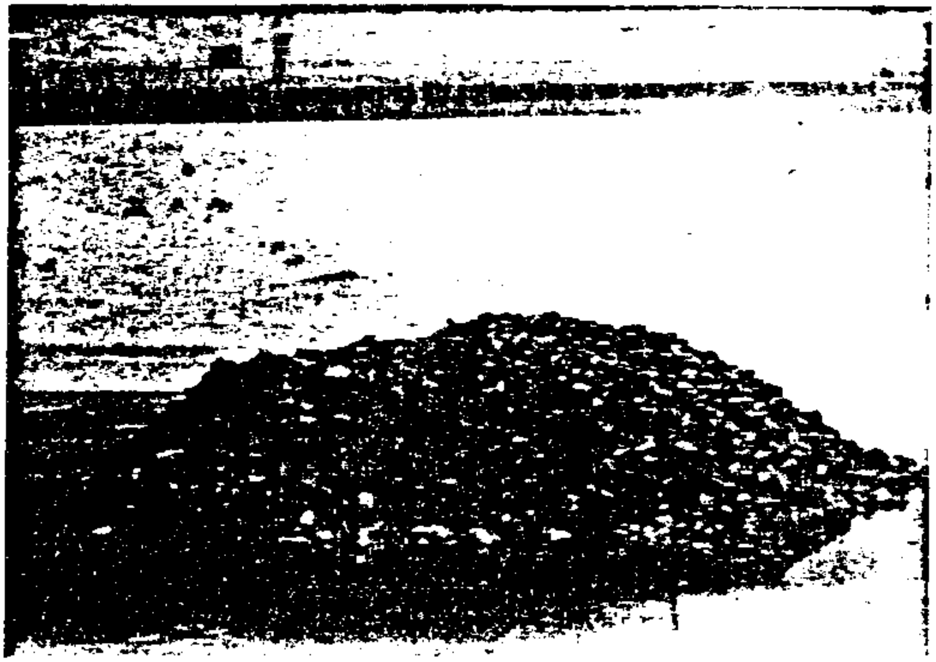


FIGURE 5.26 A MOUND OF CORE ROCK MATERIAL PLACED IN CALM WATER

FIGURE 5.27 A MOUND OF CORE ROCK MATERIAL PLACED IN WATER WITH CURRENT AND MECHANICAL WAVES

THE FIFTH SERIES OF TESTS

A SIDE VIEW OF THE MOUNDS FORMED IN CALM WATER AND WATER WITH CURRENT AND MECHANICAL WAVES

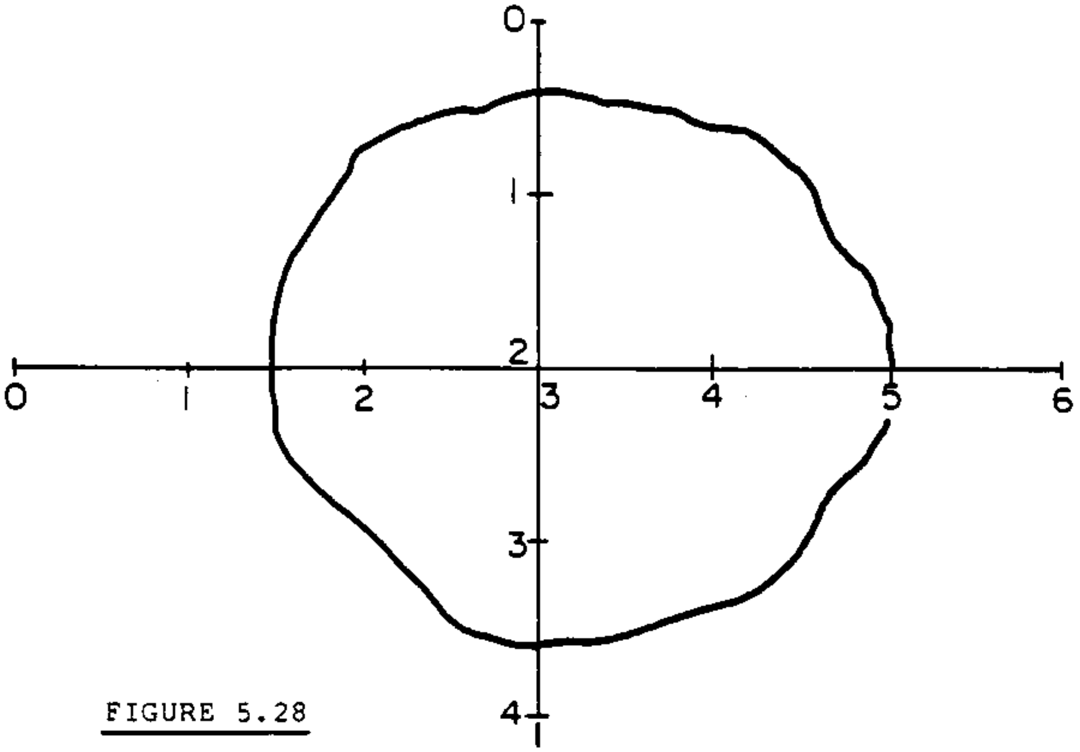


FIGURE 5.28

Layout of mound formed in calm water after dumping 30 - 10 lb loads of rock

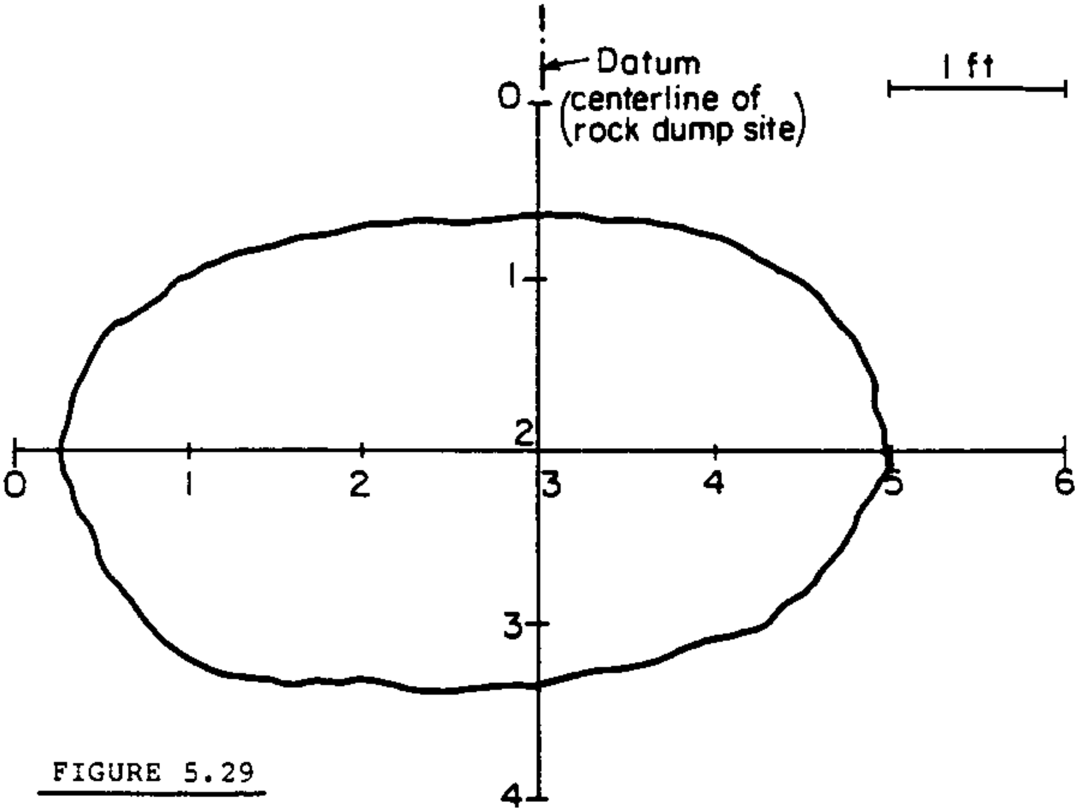
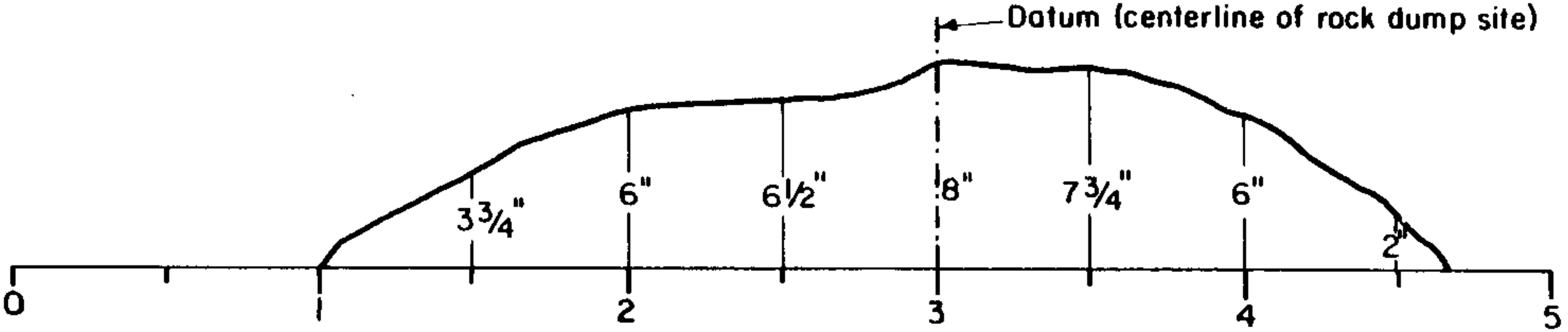


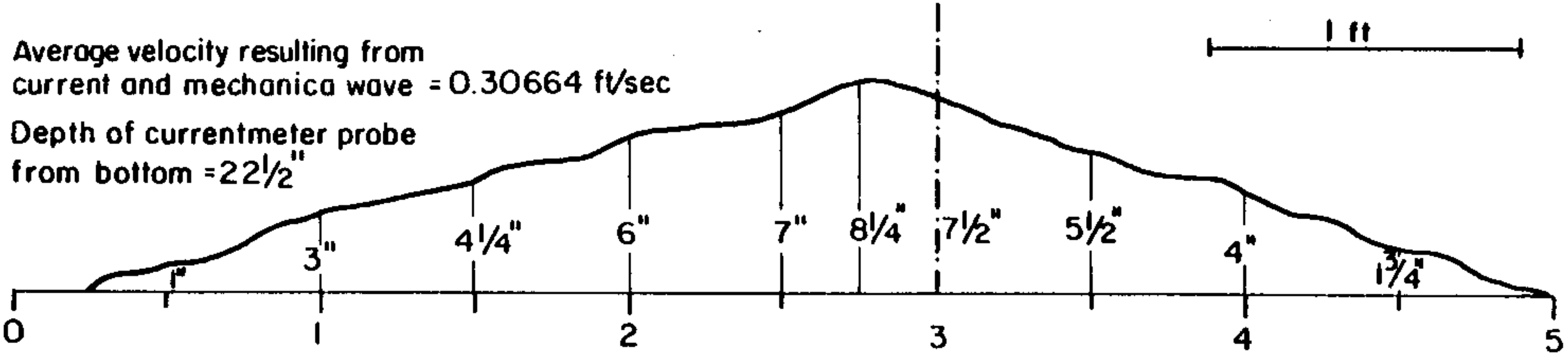
FIGURE 5.29

Layout of mound formed in water with current and mechanical waves after dumping 30 - 10 lb loads of rock



Mound formed in still, calm water

FIGURE 5.30



Mound formed in water with current and mechanical waves

FIGURE 5.31

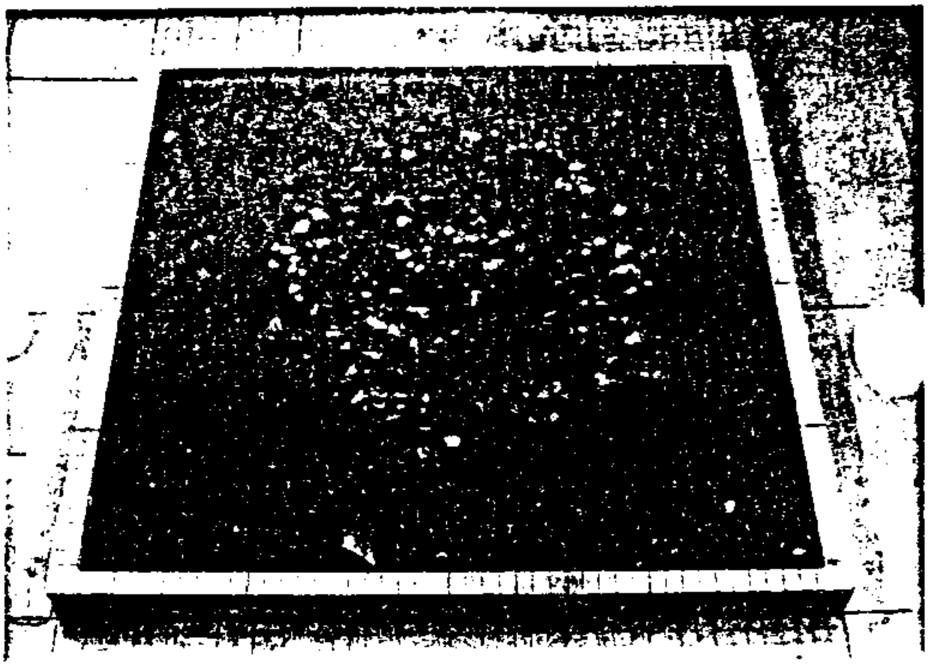


FIGURE 5.32: A LOAD OF ROCK DUMPED ON SOFT SEAFLOOR

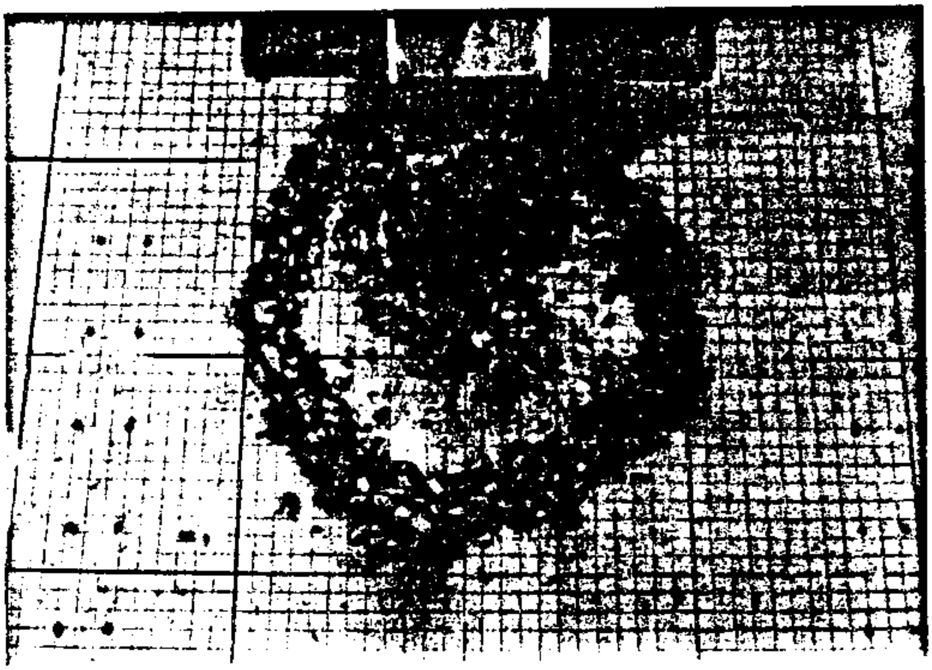


FIGURE 5.33: A LOAD OF ROCK DUMPED ON HARD SEAFLOOR

THE SIXTH SERIES OF TESTS

A COMPARISON OF PATTERNS FORMED ON SOFT SEAFLOOR VERSUS HARD SEAFLOOR (NO DIFFERENCE IN THE SIZE OF THE LAYOUT OF ROCK)

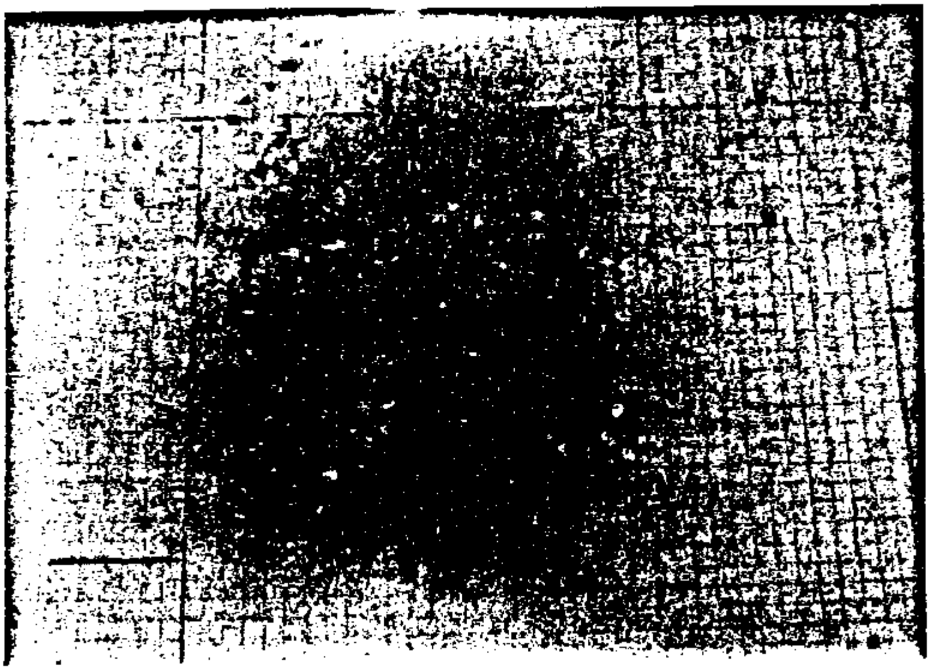


FIGURE 5.34 : A LOAD OF ROCK IS DUMPED SLOWLY (CONTROLLING THE SPEED OF TRUCK BED MOVEMENT)

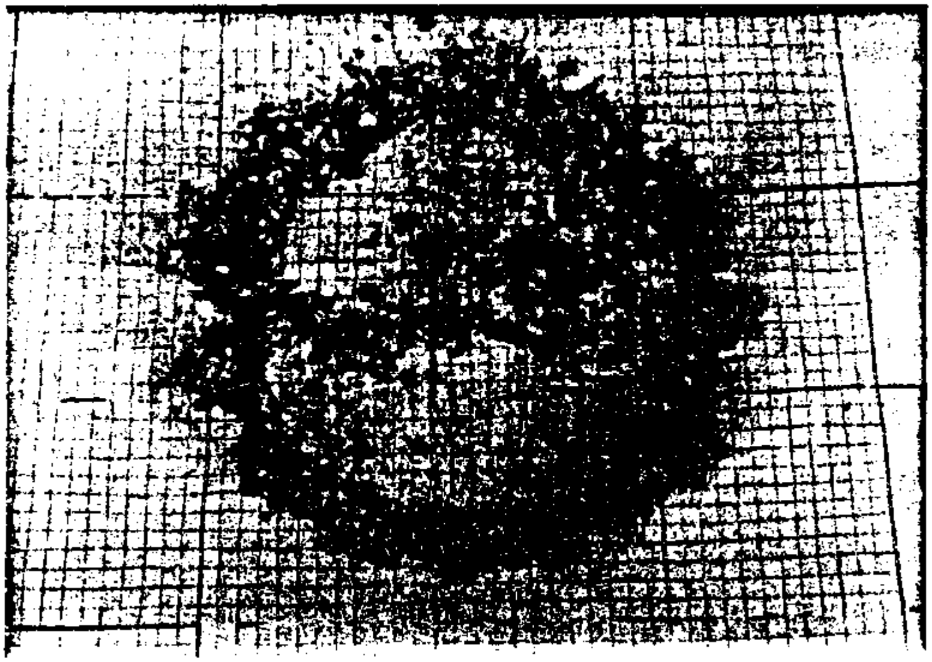
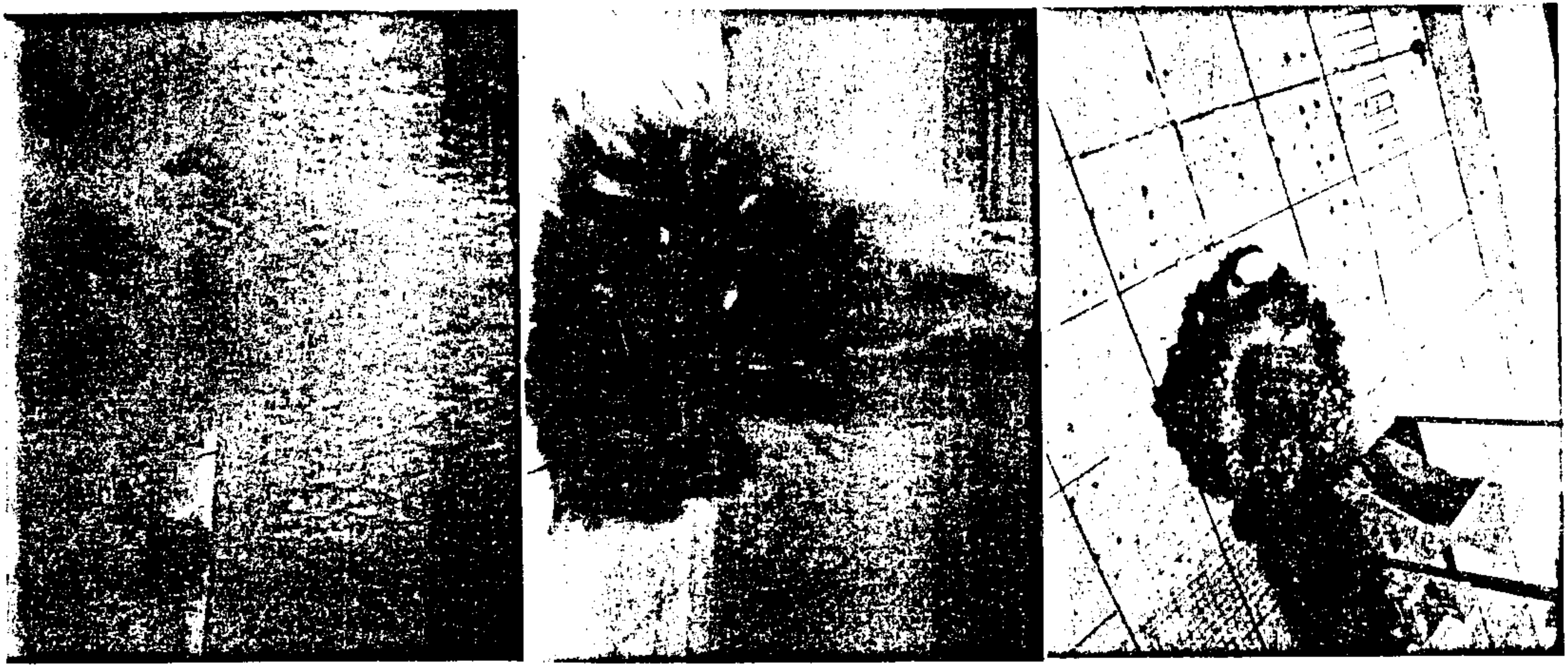


FIGURE 5.35: A LOAD OF ROCK IS DUMPED WITH SUDDEN LIFTING ACTION OF TRUCK BED

THE SEVENTH SERIES OF TESTS

COMPARISON IN PATTERN FORMED BY DUMPING ROCKS WITH TWO DIFFERENT RATES OF UNLOADING THE TRUCK



SEQUENCE OF THE DUMPING OPERATION FROM TRUCK BED TO SEAFLOOR.
(NOTE: AIR BUBBLES RISING TO THE SURFACE)
WHILE FINE MATERIAL IS SETTLING IN A LARGER PATTERN

FIGURE 5.36

A field experiment was conducted by the U.S. Army, Corps. of Engineers, (Pequegnat,1978), to assess the potential impact of dredged material disposal in the open ocean. Even though there is a physical difference in composition, between dredged material and core rock material, the results of this experiment and the model previously described show a definite similarity.

Eleven "process variables" that will effect the disposal operation:

1. Mechanical properties of dredge material
2. Insertion speed
3. Volume released
4. Solids fraction in material released
5. Current
6. Density gradients
7. Depth
8. Impact properties of clods and bottom
9. Bottom erosion strength
10. Bottom slope
11. Bottom roughness

The dispersion pattern results gathered from different sites studied resemble to a great extent the layout pattern of the core rock material dumped in the model. This complete investigation is documented in several volumes prepared for the Office, Chief of Engineers, U.S. Army, Washington, D.C. (Pequegnat,1978).

On the twelfth and thirteenth of October, 1979 a Norwegian construction company, Norwegian Contractors, conducted a test in the field for different methods of placing rock in deep water. Even though there is a great difference in the water depths between the previous experiments in the laboratory and this field experiment, their results are worth consideration.

Three different methods of placing the rock into a limited area at a depth of 24 meters were tested:

1. Guide pipe dumping

2. Direct dumping

3. Split barge dumping

GUIDE PIPE DUMPING

Rock was placed trough a 32" diameter tremie pipe, 24m in length, into a large test box.

TEST RESULTS

1. The dispersion of the rock was twice the pipe diameter. There were no pipe blockage problems. The falling velocity of the rock inside the guide pipe was approximately 3m/sec.

2. The rock seemed to pile up, with mainly

coarse material in the center surrounded by a wide thin layer of mainly fines.

DIRECT DUMPING

Dumping directly into the sea by means of a hydraulic excavator with a 2 cubic meter bucket.

TEST RESULTS

1. The lateral dispersion of the rock was relatively small, about 5m x 5m (in 24m water depth). The falling rock seems to rapidly spread out to approximately 5m X 5m, then it kept fairly well gathered together while falling the remaining distance. The falling rock creates a downward current, that washes out most of the fines.

2. With this filling method, there is considerable segregation of fine and coarser material, with a substantial amount of fines in the outskirts of the touch down area. However, the segregation is less with direct dumping than by use of guide pipe. By use of direct dumping, the segregating effect will be somewhat neutralized by the ability to constantly change the drop area.

3. The angle of friction developed is approximately 40^{o}- 45^{o}.

SPLIT BARGE DUMPING

This test was carried out by dumping with $150m^3$ selmer split barge.

TEST RESULTS

The second barge dumping test was inspected by a diver. The visibility was too poor to give any usable video recording, however, the diver could observe a minor spreading. The rock seemed to be kept fairly gathered together while falling. The rock upon landing spread out over an 8 x 8 meter pattern. The drop area had a very even surface.

There were more fines in the outskirts of the touchdown area than in the center. However, the segregation was much less significant than by using the guide pipe or direct dumping. (See figures 5.37 thru 5.42)

CONCLUSION

All three filling methods are feasible. Split barge dumping will give the least segregation, whereas guide pipe dumping will give the greatest segregation. Split barge dumping is expected to give the best mixing of fine and coarse agregates, and the most even ballast surface. Due to a somewhat uncertain touch down area for the direct dumping method, the location of the barge when dumping must be very accurately maintained.

It was recommended that the rock dumping operation be based on the use of split barges, with the possibility of "adjustment dumping" by means of direct bucket dumping and/or guide pipe dumping.

One must assume, that no matter which rock dumping method is chosen, a thin powder-like layer of fines will always settle out on the top of the ballast.

FIGURE 5.37: ROCK DROPPED THROUGH GUIDE PIPE SHOWING
SEVERE SEGREGATION AND ANGLE OF REPOSE

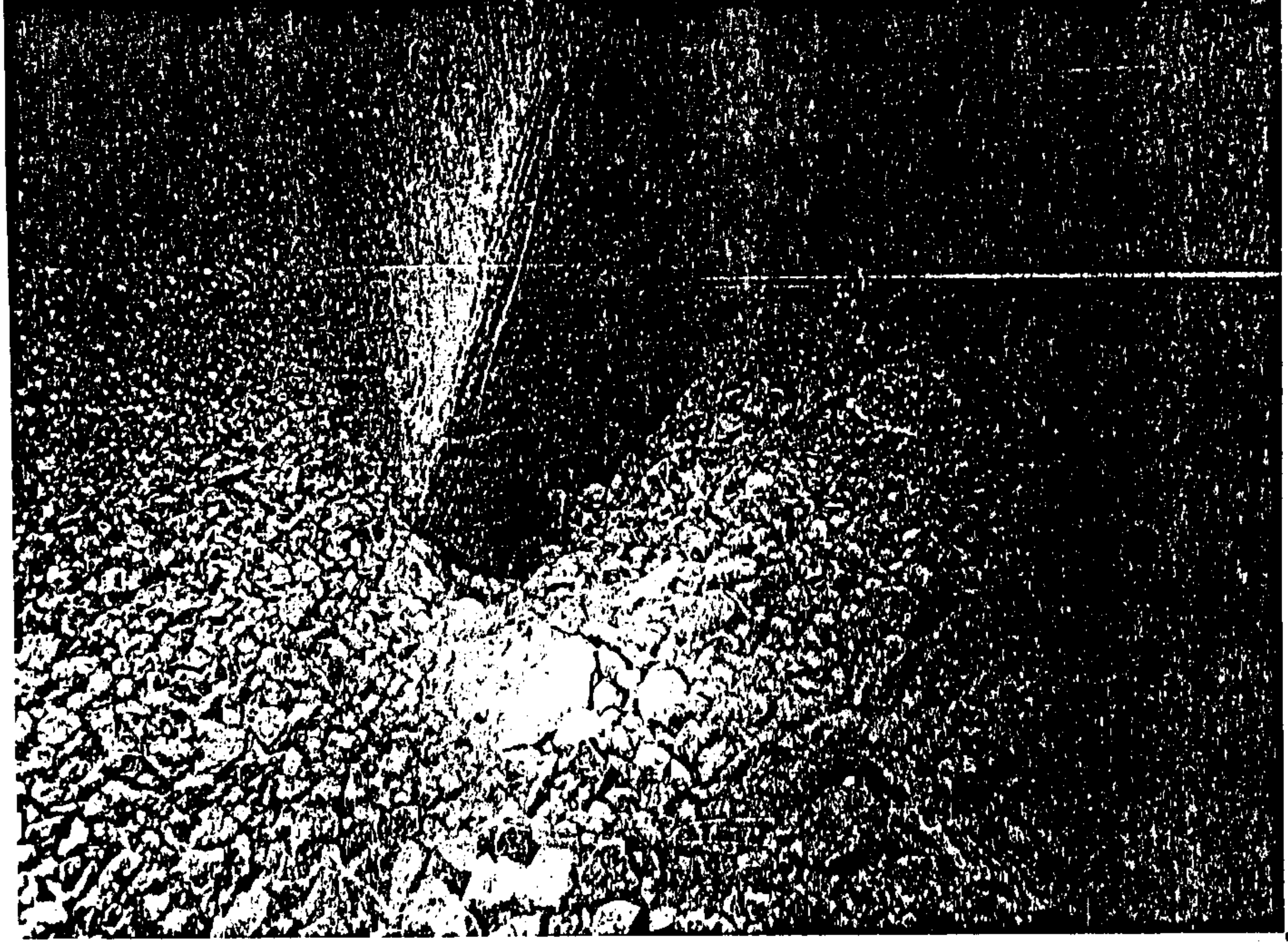


FIGURE 5.38: ROCK DROPPED THROUGH GUIDE PIPE SHOWING SEGREGATION AND ANGLE OF REPOSE

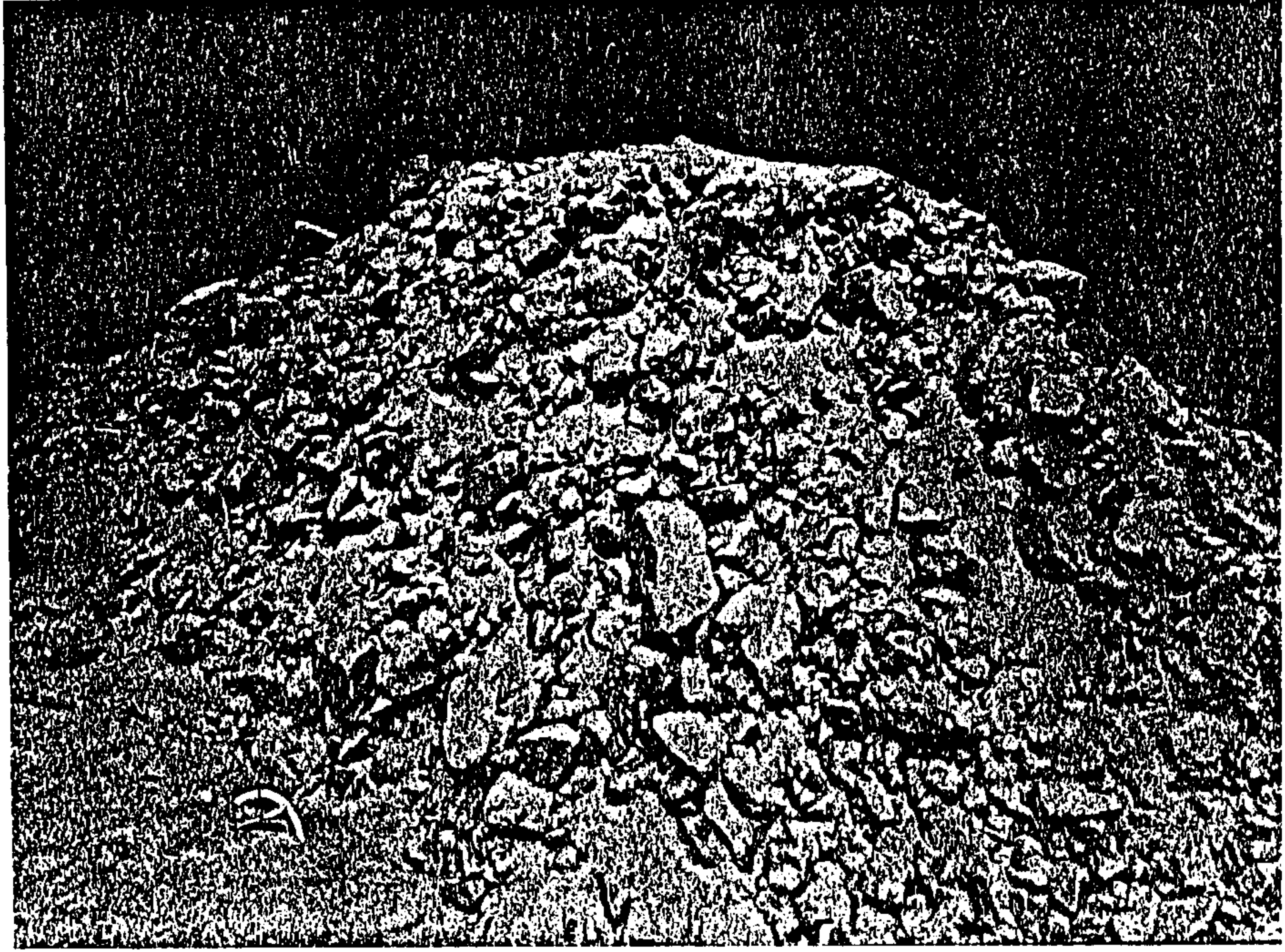


FIGURE 5.39: DIRECT DUMPED ROCK SHOWING
SEGREGATION AND ANGLE OF REPOSE

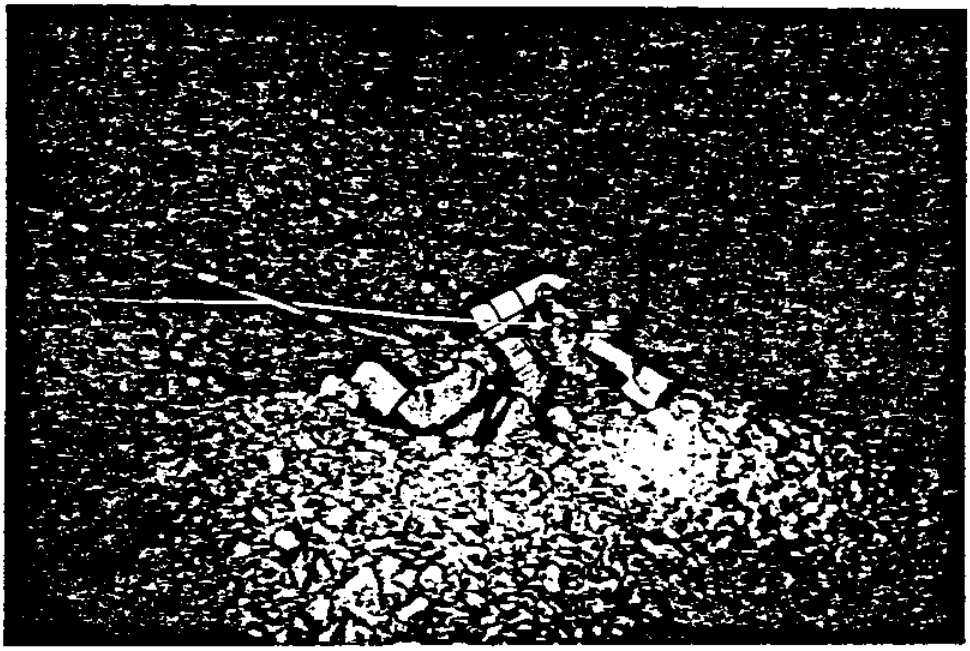


FIGURE 5.40: ROCK DUMPED BY SPLIT BARGE SHOWING EVENESS OF THE BALLAST SURFACE AND MIXING OF FINE AND COARSE AGGREGATES

FIGURE 5.41: ROCK DUMPED BY SPLIT BARGE CLOSE UP VIEW
SHOWING MIXING OF FINE AND COARSE AGGREGATES

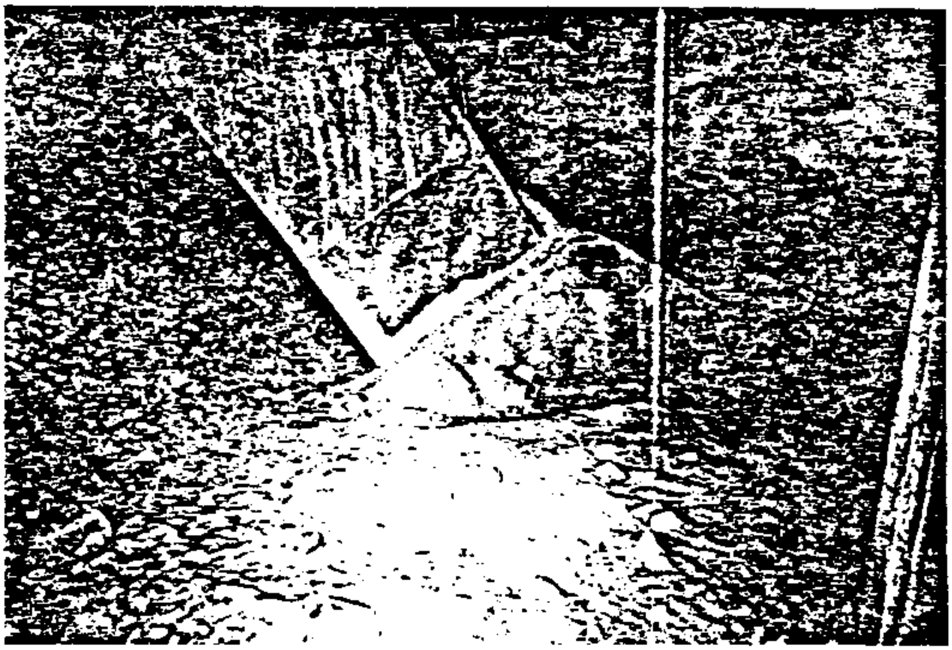


FIGURE 5.42: ROCK DUMPED BY SPLIT BARGE SHOWING BUILT UP OF FINES IN THE OUTSKIRTS OF THE TOUCH DOWN AREA

STABILITY OF MATERIALS DURING CONSTRUCTION STAGES

During the construction process, the partially-completed breakwater is exposed to damage from storm waves and currents. The core material can be washed away, if these big waves or strong currents occur, while there is no armor to protect it. For this reason, J.R. Ayers, Head of Waterfront Structures of Bureau of Yards and Docks, Washington, D.C. reported a testing program at the Waterways Experiment Station, Vicksburg, Mississippi, which addressed the stability of component materials during various stages of construction.(Ayers,1950)

The first tests were performed on models of partially completed breakwater sections representative of the various stages of construction on a prototype breakwater. Each tested condition of the model was subjected to wave attack until stability of erosion and displacement has been reached. These tests were limited to the water depth prevailing at the location of the proposed prototype, namely 58 ft. Specifically, it was desired that the model should yield information of value on the following points:

1. The height to which the Class C material could be constructed without being displaced by wave action before the protective covering (Class B) was placed.

2. The advantages to be gained by placing the Class B stone concurrently with the placing of the Class C core material.

3. The amount of covering stone (Class B) necessary to protect the core material (Class C).

4. The general stability of the completed breakwater section.

The following pages are a copy of the results of this experiment taken from The Proceedings of First Conference on Coastal Engineering, Long Beach, California, October, 1950. The title of the paper is Seawalls and Breakwaters, presented in Chapter 22, PP.192 to 204.

The results, of these, tests demonstrate the fact that placement of Class B material on both sides of Class C material concurrently will provide a higher degree of safety for the core material to withstand the wave and current action after placement.

Class C Material -- unprotected. In testing the stability of the Class C material, four different partial cross-sections representative of four stages of construction in the prototype, were used. These test sections had top elevations of - 49 ft., - 38 ft., - 29 ft., and - 24 ft., all referred to mean sea level. The model breakwater was subjected to waves of four sizes as follows:

HEIGHT	**LENGTH**	**L/H RATIO**
7.5 ft.	210 ft.	28.0
10.5 ft.	210 ft.	20.0
15.0 ft	270 ft.	18.0
21.0 ft.	300 ft.	14.3

Figure 5.43 is typical for the tests of the C material without enrockment, with top elevation at - 24 ft., and indicates the outline of the damage to the mound by waves 10.5 ft., 15.0 ft., and 21 ft. high. Figure 5.44 indicates the heights to which the Class C material may be placed in 58 ft. water depth, without displacement of the material outside the design limits, for various wave heights. It will be noted that the wave heights and corresponding maximum top elevations are as follows:

Wave height	**Maximum top elevation of unprotected Class C material**
7 to 8 ft.	- 20 ft., mean sea level
10 to 11 ft.	- 30 ft.
15 to 16 ft.	- 40 ft.
20 to 21 ft.	- 50 ft.

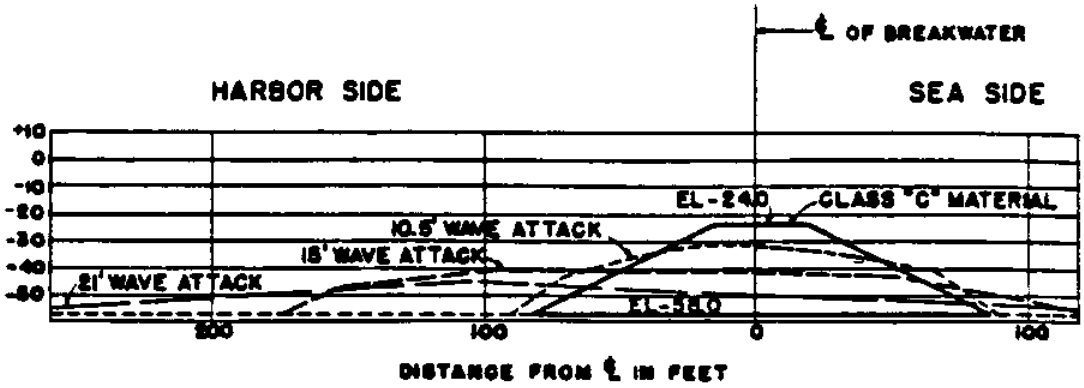


FIGURE 5.43: DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

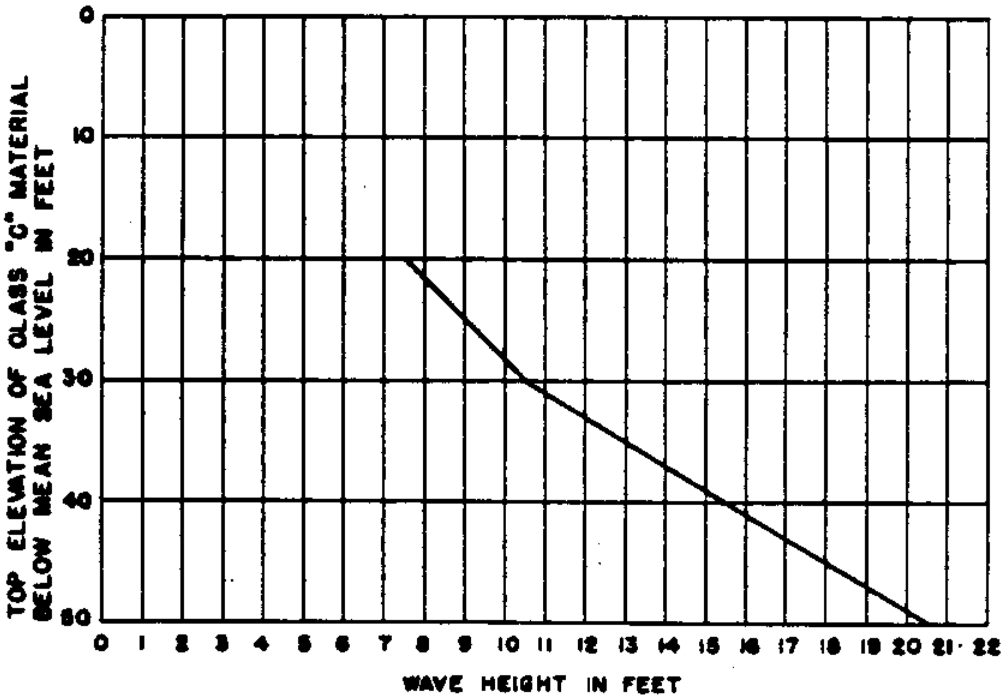


FIGURE 5.44: STABILITY OF CLASS C MATERIAL

Class C Material with Class B stone as toe protection on one side only.

Two series of tests on each of three partial breakwater sections having top elevations of - 38 ft., - 29 ft., - 24 ft., mean sea level, were made. One series had Class B stone protection on the harbor side only; the other series had protection on the seaward side only. A typical illustration of the results of the first series is shown on fig. 5.45. It appears that there is no particular advantage in adding toe protection on the harbor side only. The waves carried the unprotected Class C material over the Class B material to such an extent that no great saving could be realized by use of this method. Fig. 5.46 is typical for the results of second series, where Class B protection is provided on the seaward side only. The damage is very similar in type and extent to that of fig. 5.45 indicating no advantage over placing the Class B stone on the harbor side only and, for all practical purposes, no advantage over placing the Class C material without toe protection.

Class C material with Class B stone as toe protection on both sides shows typical results for tests of partially completed sections with toe protection on both harbor and seaward slopes. For sections of lower elevation, there was considerable displacement of Class C material due to the extensive area of this material exposed to the action of the waves. The resulting scour was concave in shape, with the deposition of the displaced material greatest on the harbor slope. As the top elevations of the sections were raised, the exposed area of the Class C material was decreased, and the displacement of material became progressively less. As a result, there was practically no displacement of the Class C material for the tests of the section with a top elevation of - 24 ft., even during the 21.0 ft. wave attack as shown on fig. 5.48.

From a study of these tests, it is concluded that the greatest degree of safety with respect to displacement of materials due to wave attack, is obtained by placing the Class B stone on both landward and seaward sides simultaneously. At lower elevations, the sections would be endangered in severe storms, but the damage would not be entirely detrimental, as the Class C material displaced would be washed over the Class B protection on the harbor slope where it would not interfere with the future placing of materials. A distinct advantage results from the fact that as the sections are raised in elevation, the area of the Class C material exposed to wave action becomes smaller, thus reducing the displacement.

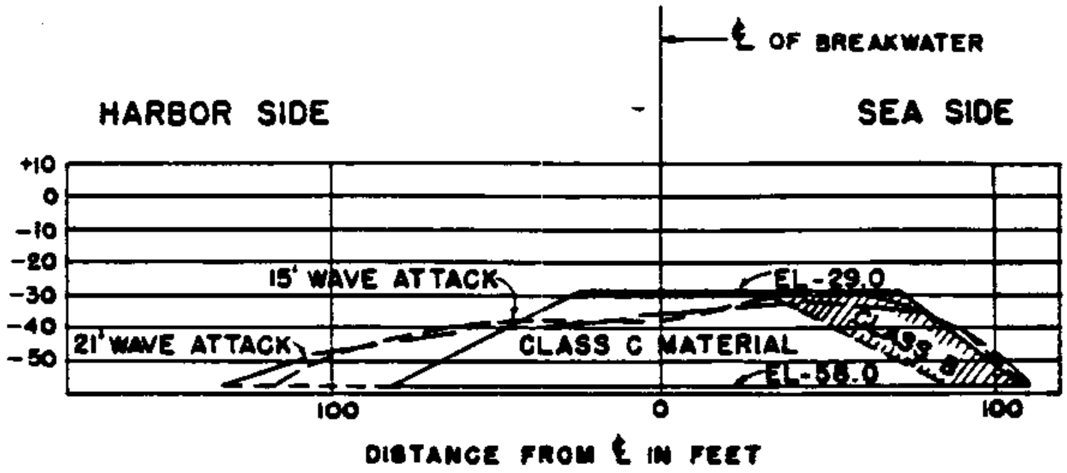


FIGURE 5.45: DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

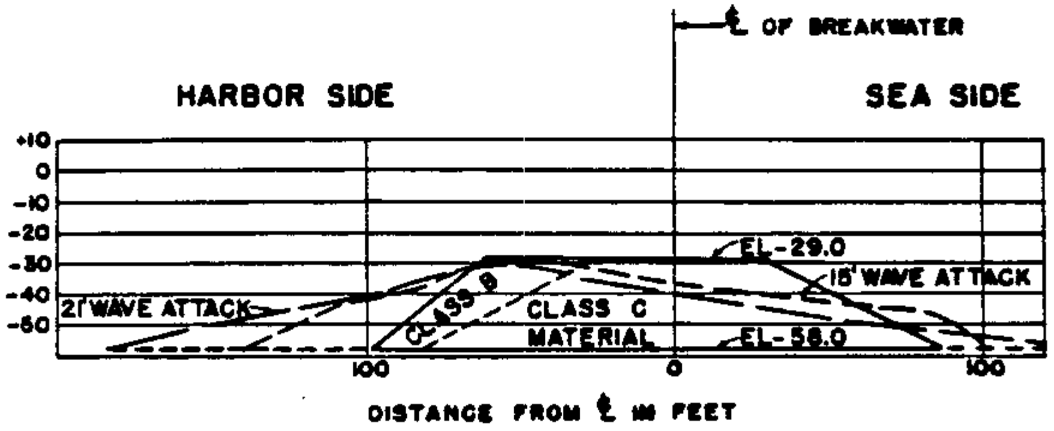


FIGURE 5.46: DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

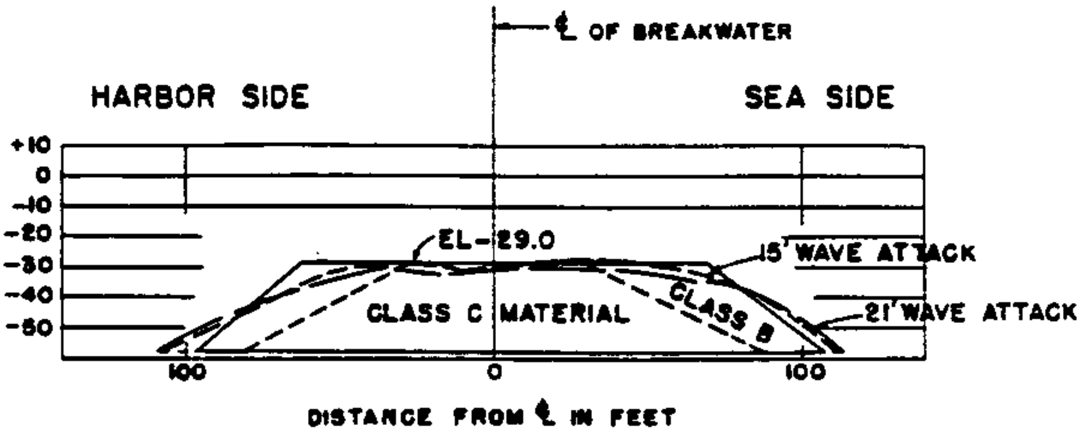


FIGURE 5.47: DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

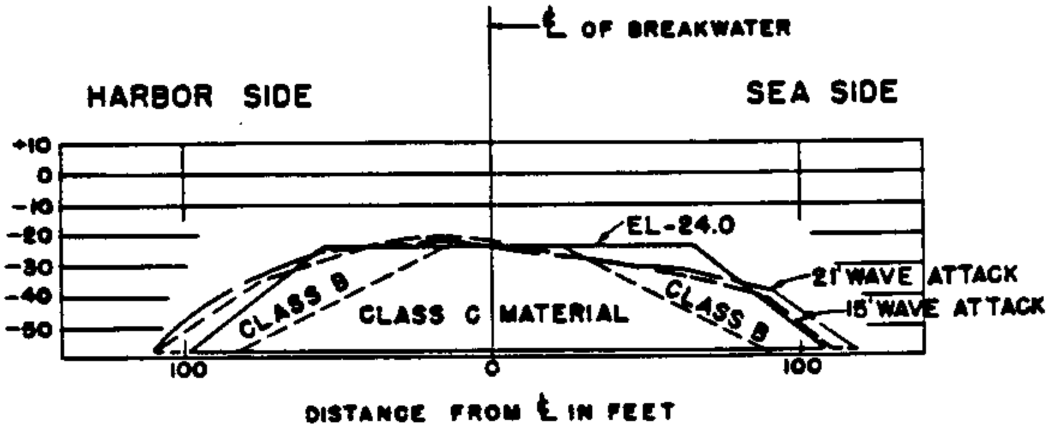


FIGURE 5.48: DISPLACEMENT OF BREAKWATER MATERIAL BY WAVE ACTION

THE USE OF BITUMEN IN BREAKWATER CONSTRUCTION

In closing this chapter it is important to discuss one of the latest applications in breakwater construction. It is a Bituminous mixture applied as a protective layer in hydraulic works. This was previously mentioned in Chapter II when this method was used to provide a protective sea bed layer. This part deals with the slopes of the core material. In order to save materials the construction height of the core requires steep slopes. This is very hard to obtain, as has been demonstrated by the previous tests. Therefore consideration has been given to the application of asphalt mixtures with slopes as steep as 1:1.75.

During construction of the core the application of sandasphalt, which is a mixture of sand and bitumen, appears to offer advantages, because this can also serve as a filter layer.

The sandasphalt mixture is applied in several ways

1. As asphalt grouting (so-called "pattern grouting") which means applying doing it in intermittent patches. The mixture would be placed through a pipe so as to penetrate the class B rock and at least the coarser class C, on the exterior face , so as to protect it during construction.

2. Pre-mixed class C stone asphalt as a core.

3. Sandasphalt as the complete core, to be later protected by B and A rock.

The application of this material in breakwater construction is fairly new, but a successful application has been proven in the construction of the breakwater at Hook of Holland (Europoort,Rotterdam).

One of the major fabricators and contractors of this product is Bitumarin Inc. which is a Dutch firm with subsidiary corporations worldwide. **Appendix G** presents a complete brochure of the product and its application in marine construction.

CHAPTER VI

SURVEY AND CONTROL

Two important phases of the breakwater construction merit particular attention. The first one is the accurate positioning of equipment to ensure that the rock will be dumped in the right spot. The second phase is an accurate control measurement to check the correct slopes and thickness of layers placed underwater to form the breakwater - the first is the Contractor's principal concern, the second can be used both by the contractor and the inspector.

In actual work being currently carried out, (as well as in previous breakwater construction), the contractor has typically used rather primitive and inaccurate methods of survey and control. These methods are summarized as follows:

1. **LEAD LINE:**

By the use of a line with a heavy weight attached to one end while the other end is attached to the boom of a crane or a helicopter. The contractor takes several measurements by placing the lead line in different locations and reading the depth by means of a level, sighting marks on the lead line. Needless to say that this method needs accurate horizontal control (by transit or electronic distance measuring as well. The major difficulty, in addition to being a slow laborious , and in the case of the helicopter, dangerous task, is that the lead line may sometimes penetrate crevices between large rocks, and other times land on a point.

2. **CATWALK:**

Another method that was used in the construction of the Humboldt Jetties is called the Catwalk which is a platform attached to a crane to allow the inspector or the contractor to walk out over the slope and sound the depth by direct lead line reading. Besides being inaccurate, this method is inherently dangerous.

3. **SEXTANT**

Sextant is an instrument that has been in use for many years in hydrographic surveys and positioning systems. This method is well within the accuracy required for the construction of breakwaters and is usually operated by two surveyors each with a sextant. Two angles are observed simultaneously to three onshore coordinated points, the center one of which is common to both observers. The resultant fix is known as the three point problem which is arduous to calculate but can be plotted graphically quickly and simply.

The best method of graphically plotting a sextant fix is with a three arm protractor. The best model is the brass type with a vernier or micrometer adjustment for the minutes. To plot with a three arm protracter, its arms are set to the value given by two simultaneous sextant angles. By maneuvering the arms over the positions of the observed points plotted on the chart the intersection of the arms gives the location from which the angles were taken, the observers' location should not fall on or near the circumference of a circle, otherwise the fix will not be unique (circular fix).

4. **RANGES**

Another method that has been used for the alignment and positioning of barges during construction is the use of ranges. Two ranges or lights are fixed on the center line of the breakwater, while the third one is fixed on the barge. An observer on shore gives signals to the operator aboard to move either right or left until the third range is lined up with the lights on the breakwater.

OPTICAL AND ELECTRONIC POSITIONING

There are many ways to determine position at sea. Taking optical bearings on landmarks can give accurate position but range is obviously limited and dependent on visibility conditions. The same applies for photography or photogrammetry (aerial photography). Electronic systems offer all weather capability and a variety of ranges but usually involve a trade-off between range and accuracy.

RANGING SYSTEMS

Most of these systems are used for horizontal control. They use "X" band frequency (RADAR) and can operate only on line of sight. This system involves a shipboard receiver for displaying ranges from two remote transponders situated onshore at points with known coordinates. A coded pulse is transmitted from the master to each transponder and counts the time of the return signal from each one. The shore units can be mounted on a theodolite tripod or on an iron pipe. The shipboard antenna is also mounted on an iron pipe.

HYPERBOLIC POSITIONING SYSTEM

The hyperbolic positioning system operates on the two MHz range, which is different than the previous system that depends on the line of sight. These systems can be used in a range/range or a hyperbolic mode. They vary in the configuration of the network, which consists usually of three stations, one master and two slaves.

Hyperbolic navigation systems are so called because the lines-of-position that they produce are hyperbolic in form. The shipboard receiver derives a hyperbolic position line by measuring the difference in the times at which the synchronized signals from the two slave stations are received. (Milne, 1980)

There are several systems marketed, but generally they have the same accuracies and are subject to the same variations and natural influences. They usually use low frequencies (Omega, Loran C, and Decca Navigator).

This system will give more accuracy to the barges in the positioning for dumping the rocks than the previous systems.

HYPERBOLIC RADIO POSITIONING SYSTEM

This classification of systems has even more accuracy than the preceeding system. This radio positioning apparatus uses a medium carrier frequency. They are basically hyperbolic but adaptable to circular operation and they are intended for radio position fixing as opposed to general navigation. Systems falling into this category include Raydist, Hi-Fix/6, C-Fix, Toran, Syledis and Pulse/8 (Milne, 1980).

The Raydist Ultraprecise Electronic Navigation System can pinpoint locations at sea with a sensitivity of 1.5 feet at ranges of up to 250 miles from base stations. Raydist is a proprietary radio navigation system designed and built by Teledyne Hastings-Raydist and is about 30 times more accurate than standard Loran C.

Raydist does not use pulses. Instead, two CW transmitters at separate locations each radiate a continuous radio signal. The frequencies of the two signals differ by several hundred cycles per second. These two transmitted signals interact to form a beat frequency equal to the difference of the two original frequencies. These two transmitters could be mounted separately on posts at the ends of the straight row of caissons comprising the closure breakwater or on the mainland.

If this third or beat frequency, as detected by a receiver at one transmitter site, is compared with that detected at the other, the phase relationship of the two signals can be determined. This phase relationship can then be used to detect very small changes in distance between the two stations. In practice, the signal received at the fixed transmitter could be sent back to a mobile transmitter mounted on the crane boom via a separate communication channel so that phase comparison can take place.

As the mobile station moves toward or away from the fixed station, it passes through a series of positions in which the two compared signals are exactly in phase. These in-phase locations actually form a series of concentric circular lines of position centered on the fixed station. The distance from one circle to the next is called a Raydist lane. The actual width of this lane varies according to the particular frequency being used, but in a typical case is about 150 feet. Raydist equipment is capable of discriminating the phase change accurately enough to determine position down to $^{1}/_{100}$th of a lane, or, in a 150 foot lane, a foot and a half.

Raydist is manufactured and serviced near the site and has been used in applications on Chesapeake Bay.

A more modern radio positioning system using UHF is Syledis. It has the advantage of being small, portable and easily assembled.

Most of these radio positioning systems suffer a loss in performance and reliability during the night-time. This is due to the interference with the wanted ground wave signal by the unwanted skywave component. This is not a major concern since most of the construction activities on a breakwater are completed during the day light hours, unless the breakwater is built in the northern North Sea where night can last up to 24 hours as far as skywave is concerned.

ELECTROMAGNETIC DISTANCE MEASURING (EDM)

This surveying equipment utilizes electromagnetic distance measurement system (EDM) by radio and other methods. They do not use the hyperbolic principle: instead they use the circular or range/range fixing which has the obvious and real advantage of requiring only two shore stations compared with three in the hyperbolic mode.(Milne, 1980)

PRECISE RADIO POSITIONING SYSTEMS

This group uses extremely high radio frequencies known as microwaves. They operate either on a phase comparison or a pulse timing. Instruments falling into this catagory include Autotape, Mini-Ranger, Ralog, Shoran, Trisponder and Tellurometer. The advantages of this type of system are:

1. High accuracy even at maximum range.

2. Continuous range display.

3. Remote stations require no special attention

except for power supplies.

4. Easy compatibility with track plotters and automated plotting equipment.

The Motorola Mini-Ranger III is a light-weight line of sight measurement system. Its range varies, depending on the relative height of the antenna mounted shipboard and the ones at the shore stations.

CLOSE-RANGE INSTRUMENTS

Ingham,(1975) summarized the direct methods of positioning used for close ranges as follows:

1. Sextant angle resection (5 km +).

2. Theodolite intersection (up to 5 km).

3. Substance ranging (up to 150m).

4. Distance line ranging (up to 100m).

5. Transit cut-offs (up to 100m).

Other close-range methods are described by Milne,(1980). These methods include the modern **LASER** and the **INFRA-RED E.D.M.** "One example is the AGA Geodimeter 700, which is a hellium-neon laser. It has been used offshore, suspended on gimbals, for precise position-fixing of distances up to 1 km with an accuracy of $\pm$ 5mm with an additional range-error margin of 1 mm^{-1}. In sea trials near the Swedish island of Sylt, distances of up to 6 km were measured accurately in good weather using a triple bank of prisms", (Milne, 1980).

Such a system can have advantages in the construction of rubble mound breakwaters.

PHOTOGRAMMETRY

The term Photogrammetry means the process of measuring images on a photograph. In a more comprehensive sense, photogrammetry includes (Moffitt and Mikhail,1980) :

a. Photographing an object.

b. Measuring the image of the object on the processed photographs.

c. Reducing the measurements to some useful form, such as topographic map.

Photogrammetry has been used in the past by archaeologists for the identification of submerged harbours and ruins, for example, by Frost (1969) at the Island of Arwad, off the Syrian coast, in the Mediterranean. An aerial survey was also carried out from a balloon to aid an underwater survey of the foundation wall of a Mycenean-age village lying underwater in the southern Peloponnese of Greece (Walton,1969).

Photogrammetry is an excellent tool for control of above water sections (stereoscopic) in the construction of breakwaters. Flying over the construction site once a month, using shore station controls, this method will provide an accurate verification of lines, grades, etc. down to about 5 feet below water surface. However, the development of infra-red and color aerial photography has allowed a deeper penetration, but still dependent on the clarity of the water. This method will reduce the time spent using other types of surveying. It also reduces the risk involved in the other procedures, which has always been notoriously dangerous and costly.

The success of the photographic mission is contingent upon optimal meteorological conditions. Low clouds, haze, smoke or fog will preclude operations. Light is usually sufficient for photographic work from one hour after sunrise to one hour before sunset.

ACOUSTIC SYSTEMS:

After the placement of the core material the contractor as well as the engineer needs to know the exact profile achieved during the construction operation. In the begining of this chapter several inaccurate methods were discussed. With the advent of sounding equipment this task could be performed more accurately and with greater safety.

Sounding is the term used to describe the operation of taking depth measurements over an area of the seabed.

ECHOSOUNDERS

Joseph Caldwell (1950) presented a description of the principles of operation of an echo sounder: " Briefly stated, the essential components of an echo sounder are a signal generator, a signal transmitter, an echo receiver, and an amplifier-recorder. The first and last items are frequently put in the same box and are referred to simply as the sounder. The other two items are usually mounted together or near each other, and the combination is called "the transceiver." The sequence of operations in obtaining a sounding is as follows:

a. The signal generator initiates, by electronic methods, a short, but strong, electrical impulse which is sent to the signal transmitter.

b. The signal transmitter, which is mounted underwater, converts the electrical impulse to a mechanical vibration or sound wave which is directed toward the bottom.

c. The sound wave reaches the bottom and is reflected as an echo.

d. This echo is detected by the signal receiver which is also mounted underwater. The receiver converts the echo into an electrical signal which is sent on to an electronic amplifier.

e. The electronic amplifier receives the signal and amplifies it sufficiently to make it discernible on a suitable indicator or recorder.

f. The indicator or recorder receives the signal from the amplifier, measures the time lapsed from the generation of the sound signal to the return of the echo, converts this time interval to equivalent depth of water, and furnishes an indication or record of this depth.

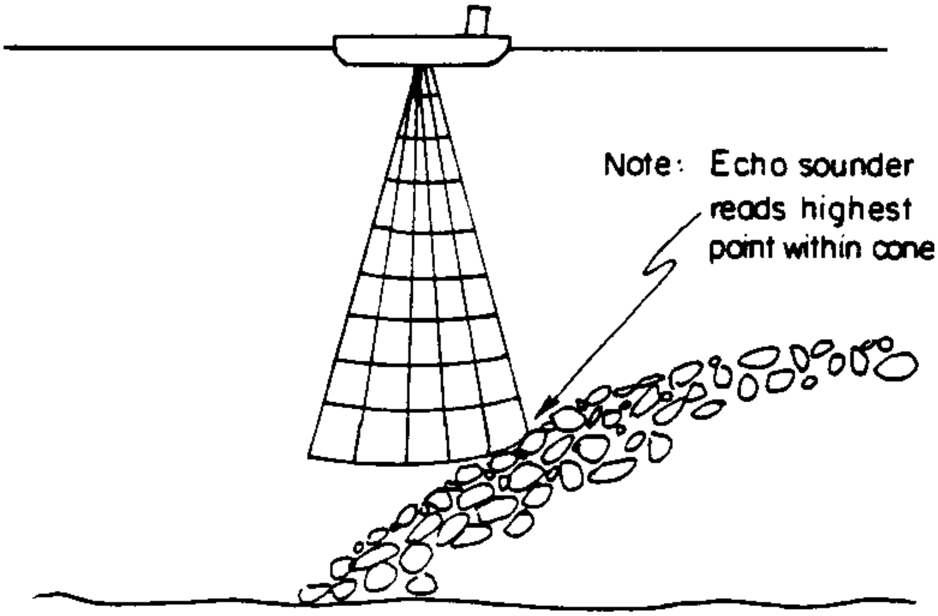
FATHOMETER

The fathometer has been in use for several decades. It uses ultrasonic pulses emitted by a source that is most often mounted on the hull of a boat. These pulses are directed vertically downward in a narrow beam and reflected off the seafloor by materials of different acoustic impedance. Reflections resulting from each separate sound pulse are detected and are automatically correlated and successively recorded on a moving-paper or chart recorder; hence, a cross section with constant scales is produced (fig. 6.2). The horizontal scale is determined by the speed of the boat, and the vertical scale is controlled by the travel time of the sound waves in the water and sediment on rock.

Site surveys are usually made by sonic depth finding equipment. Echo sounding equipment has a cone of transmission and thus fails to display sharp changes in elevation. Since it reads the highest elevation within a core, but shows it as being located on the axis of the core, it is relatively unsuitable for breakwater control during construction although it may be useful for preconstruction site surveys.

As an example with a breakwater slope of 1 vertical 2 horizontal and a 15° cone in 40 feet of water, the error will be over 2 feet, always reading higher than the true value. Refinement is possible by the use of narrower cones.

A schematic of an echo sounder in operation is in the next figure (6.1).



OPERATING PRINCIPLE OF A TYPICAL ACOUSTIC BOTTOM PROFILING

FIGURE 6.1

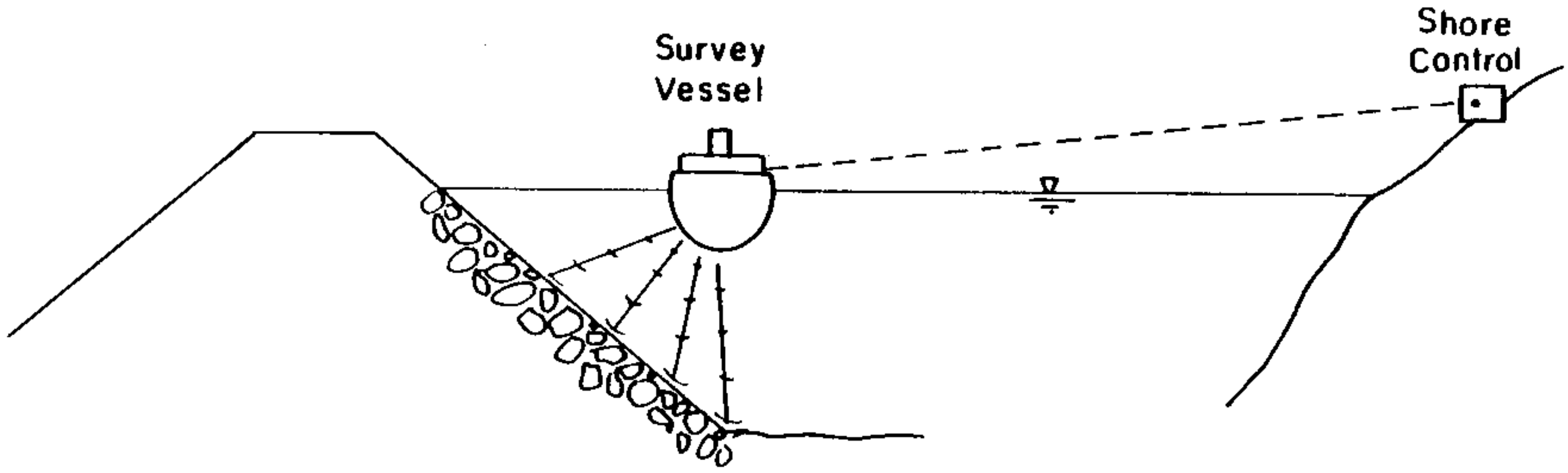
SCAN PROFILING SONAR

Scan Profiling Sonar is a system which, from a single location records a profile of water depths along a particular line of bearing. By manually changing the orientation of the device, profiles along several lines of bearing can be obtained to provide detailed information of bottom contours.

This instrument is potentially applicable to control of breakwaters during construction. The ability to gather bottom profile information from a single point greatly speeds surveys and drastically reduces the amount of navigation required during a survey.

By integrating this system with an electronic position and level control on shore, continuous correction can be automatically made for roll, pitch, heave and tide, giving an accurate plot of cross-sections below sea level at the time of survey.

A sketch of a surveying vessel coordinating its position with a shore control station, and plotting a profile of the slope of the breakwater, is presented in the next figure (6.2). A complete description of some equipment used in the bottom scan profiling sonar is presented in **Appendix H.** It also includes some copies of actual plotted records of trenches and pipelines.



SURVEYING VESSEL

FIGUIRE 6.2: A SKETCH OF A SURVEYING VESSEL COORDINATING ITS POSITION WITH A SHORE CONTROL STATION AND PLOTTING A PROFILE OF THE BREAKWATER

ACOUSTIC PROBLEMS

Several factors limit the application and accuracy of these instruments(Caldwell, 1950):

1. The speed of the survey craft must be slow enough to avoid entraining air under the vessel. Air bubbles are sound blocks.

2. The angular size of the sound cone could vary with the radius of the transmitter plate and the frequency of the sound signal.

3. The strength of the echo necessary to enable the amplifier-recorder to detect the echo and give a usable record depends on the surface of the seafloor. If it is clean rock or sand bottom the echo is well defined, but if it is unconsolidated mud bottom it will return a faint echo.

4. The absorption of sound energy by the water is limited to the depth to which the sounder can operate. This particular limitation is of little concern, because breakwaters have been constructed in water depth not exceeding 50m.

ADVANTAGES OF SOUNDERS

Generally, the supersonic sounders are preferred because of (Ingham,1975):

1. The high directivity from a small source.
2. Their relative freedom from the influence of random engine noises and similar disturbances.
3. It is light and often portable.

LASER DEPTH SOUNDING SYSTEM

The Missile Systems Divisions for the U.S. Naval Oceanographic Office has developed a pulsed laser airborne depth sounding system (PLADS). This system is based on the use of the high power output of frequency-doubled, Q-spoiled Yttrium Aluminum Garnet (YAG) lasers for penetration of the ocean. (Rattman and Smith, 1971)

Similar to PLADS, the latest equipment developed in Australia (Clegg and Penny,1978) consists of a pulse laser transmitter with an optical receiver to measure the difference in arrival time of pulses reflected from the water surface and the seabed. In the vicinity of Adelaide, Australia, reflections from the seabed were obtained regularly in depths of 30m and in places down to 40m. At present this technique is restricted to profiling in the vertical plane just like an echosounder.

The next step in airborne laser-beam technology is to design a system where by the beam is scanned across the track of the aircraft to produce a swath survey similar to the side scan sonar systems described earlier (Milne,1980).

Barring the expense of the operation, this method could be used to provide the profile of the breakwater, but a major disadvantage is that its depth is limited to the clarity of the water.

At the present time the Missile Systems Division is working on a better version of the system that will work more accurately and enabling a substantial increase in speed.

Until the laser depth sounding system is affordable this method is not recommended for the use of profiling breakwater core construction.

CONCLUSION

A breakwater is a complete system. It is built of several components but it functions as one unit. Failure to recognize this very important concept in the past has led to severe damage and failures.

The abundance of rock and familiarity with quarrying methods to produce large quantities at an economical cost have led to the adoption of rubble mound breakwaters for most of the recent construction, to the extent where the main requirement in any decision-making regarding the design or construction of a breakwater has usually been to obtain the lowest costs possible, often without adequate consideration of the many other factors involved. Especially for deep water breakwaters, a more thorough evaluation is indicated.

One of the principal components of a breakwater is the core, and this therefore has been the subject of this study. The functions of the core have been defined as:

1. Furnishing the mass.
2. Providing stability.
3. Providing support for the rip-rap and armor units.
4. Reducing wave energy permeability.
5. Acting to transfer the wave forces and gravity loads to the foundation.

In designing rubble mound breakwaters, attention must be given to the practicable means and capabilities for accurate construction and control. Before starting construction the contractor should collaborate with the designer to determine if any changes can be made to provide a better structure and to facilitate a more efficient method of construction.

Site investigation is an important factor in building a successful structure. Reinforcing a weak or unstable soil by one of the methods described in Chapter III will provide a stronger foundation to the entire structure. The use of a mattress before placing the core material will act as a filter between the foundation and core rock. This will minimize the migration of small particles from the foundation up into the core and will also prevent the settlement of the bigger rock into a sandy seafloor. Migration of sand, scour, settlement or liquifaction can cause instability to the whole structure which may lead to failure. Data from bathymetric surveys, soil corings, and subbottom profiles should be studied to provide an accurate assesment of bottom conditions. The engineering properties of the seabed, such as soil shear strength, density, and natural water content, may be accurately determined from soil samples.

Specifications and tests regarding the quality of the core rock material should be specific, and should address character of fragments, their gradation, durability and abrasion resistance.

The construction of a breakwater requires a tremendous amount of rubble, (up to as much as ten million cubic meters in major breakwaters). The selection of a quarry for the exploitation of such a volume of suitable rock is an important engineering gealogical task. Petrographic evaluation can be a valuable tool in recognizing destabilizing qualities of rock. The shear test may be more valuable than the compression test, since rocks in breakwater are more likely to fail in shear than in compression.

Construction procedures and equipment should be carefully selected. Designer and contractor should be aware of the weather and sea states that will pertain during construction, so that suitable and appropriate methods and equipment will be selected.

The use of floating craft in constructing breakwaters has the advantages of flexibility but its use may be interrupted by storm or heavy wave action.

Dumping from barges may present a problem in that if material is allowed to fall freely through water, segregation tends to occur. A wider spread of the rock dumped will also occur if there is fast current or heavy wave action.

Placement in large masses, that is dumping all at once, tends to reduce segregation, but produces a wider pattern and flatter slopes than dumping the rock slowly.

The core should be properly protected as soon as possible after placement by the placement of class B rock so as to prevent excessive leaching of fines and sloughing of the slopes.

Survey and control are very important operations during and after construction. Maximum use should be made of modern electronic survey techniques.

Horizontal position control can be obtained by a wide variety of means: ranges, sextant, lasers, or electronic positioning.

Seabed and breakwater bathymetry and breakwater profiles may be drawn using underwater profiling techniques, and using multi-channel and multi-angle acoustic sensor techniques. Above water profiles may be developed by aerial photogrammetry.

The design should recognize the need for reasonable and practicable tolerances, and the sequence of construction activities involving foundation preparation, core rock placement, filter layers and armoring.

THE RECOMMENDATIONS ARE SUMMARIZED AS FOLLOWS:

1. In designing a breakwater attention should be given to the practicability for attaining accurate construction and control, taking into consideration the environment and the proposed methods of construction.

2. The seabed should be investigated thoroughly. Where indicated, compaction and consolidation methods should be utilized to insure a proper and firm bed underneath the breakwater.

3. Specifications and tests of core material should be more accurate and more specific. Physical and chemical properties as well as gradation requirements should be specified so as to insure a durable core which will properly fulfill its functions.

4. Construction procedures and equipment should be very carefully chosen with awareness of the weather and sea states which wil occur during construction, in order to ensure that proper methods and equipment are adopted.

5. Special attention should be paid to methods of survey and control, making maximum use of

modern electronic, acoustical, and photogrammetric techniques to ensure accurate construction.

6. There is a need for co-ordination of research not only in the laboratory but also in the field. Most laboratories are heavily committed to testing of designs for specific proposed future projects, and insufficient research appears to have been directed to studying the conditions and performance of already constructed and existing breakwaters.

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BROCHURE

"ZINKEN.......OF VERDRINKEN!
aannemers combinatie zinkwerken n.v. ACZ

BROCHURE

"ELIMINATE WATER-CAUSED SOIL EROSION"
SPECIFY: POLY-FILTER X, FILTER X, POLY-FILTER

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APPENDIX A

APPENDIX A

FILTER MAT ASSEMBLY PLANT

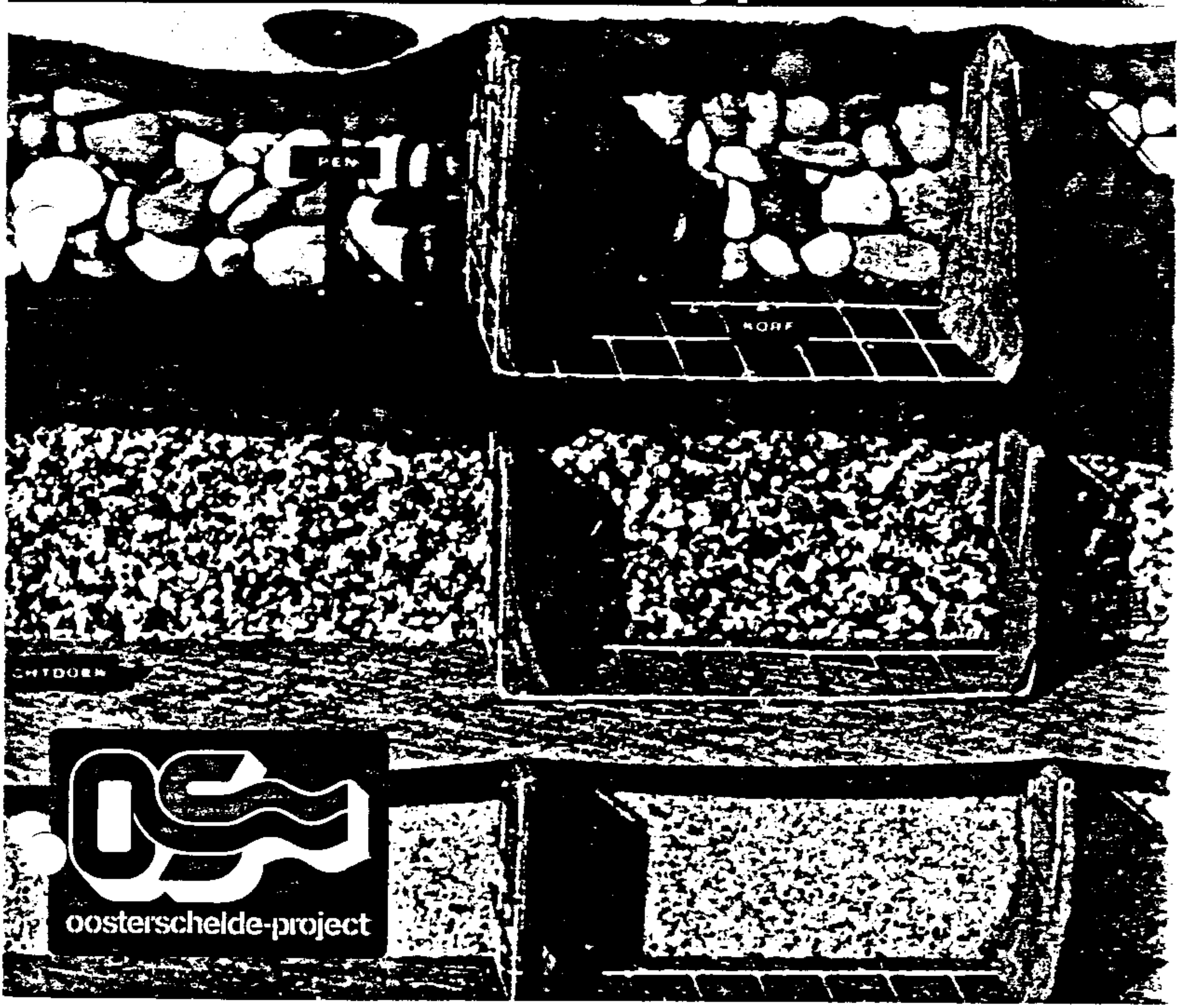
Appendix A is a brochure presenting the Filter Mat Assembly Plant (prefabricated foundation bed) used as the foundation for the storm-surge barrier which is part of The Delta Project across the Eastern Scheldt estuary in Holland.This brochure has been prepared by the design team Dosbouw v.o.f. and Tebodin B.V.

This filter mat could be used for the preparation of the foundation of breakwater whenever it is appropriate.

oosterschelde storm surge barrier

dosbouw

filter mat assembly plant



oosterschelde-project

Filter Mat Assembly Plant
(prefabricated foundation bed)

Principal:

Ministry of Transport and Public Works, Delta Works Department (Rijkswaterstaat)

Main contractor:

Dosbouw v.o.f.
Dosbouw Construction Consortium consists of:

Ballast-Nedam Group N.V.
Royal Boskalis Westminster B.V.
Baggermaatschappij Breejenbout B.V.
Hollandsche Beton Groep N.V.
Van Oord-Utrecht N.V.-A.C.Z. B.V.
Royal Volker Stevin N.V.

Design team:

DOSBOUW v.o.f.
TEBODIN B.V.

Introduction

As part of the Delta Project a storm-surge barrier is being built across the Eastern Scheldt estuary in the remaining channels Hammen, Schaar van Roggeplaat and Roompot. The barrier consists of 66 prefabricated piers and 63 sliding steel gates which can be lowered between the piers in case of emergency. The piers will be installed on the sea bed without the need of pile foundations. However, in order to ensure the stability of the sandy soil beneath the piers a filter construction will be placed on the sea bed.
This construction, consisting of prefabricated mats, will form the foundation bed directly supporting the piers. These so-called filter mats are being assembled near the Roompot construction dock on the Geul dam section in a specially designed plant which is scheduled to supply the required number of mats within a period of 1½ years.

Foundation bed

The 66 piers of the Eastern Scheldt storm-surge barrier are being built in construction docks. Subsequently, they will be transported tot their final position and installed on the sea bed by the lifting vessel **Ostrea** (fig. 1). Considerable attention has been paid to the foundation bed on which the piers will be placed. In the estuary channels **extensive soil improvements** have been carried out. In some places existing layers of sand with poor bearing capacity have been removed by dredging and replaced by better quality sand. The subsoil including the clean sand newly brought in is compacted by the specially designed barge **Mytilus** (fig. 2) to reduce porosity.
After these soil improvements the prefabricated filter mats will be laid.

Function of filter mats

The mats consist of a filter construction of three graded layers. They are impermeable to the sand of the sea bed and form a carefully composed transition to the layers of ever coarse stone which will be placed around the base of the piers.
They hold the underlying sand of the sea bed in position despite the pressures to which the barrier may be exposed, and at the same time provide sufficient drainage for the water. This latter function is of prime importance, thus ensuring that the upper layers of sand are not washed away should the barrier be exposed to severe current action or heavy waves.
The mats, approximately 42 m wide and 200 m long, weigh about 5.500 tons each.
In order to prevent the mats from being damaged before and during the installation of the piers, a second smaller mat, 31 m x 60 m (see fig. 3), is used immediately below the base of the pier.
In view of the very high standard of requirements both regarding quality and exact position of the filter construction, it was decided, after various ways of production had been considered, to prefabricate the mats and to transport them to their location. The same procedure, used before when the 'block-mats' were so successfully laid, will be followed.
The filter mats are assembled on shore and then wound on to a gigantic cylinder floating in front of the plant.

Laying filter mats on the Eastern Scheldt sea bed

The filter mats are placed on the sea bed in water, having a depth of maximum 35 m, by the specially built rig **Cardium** (fig. 4). This vessel is also equipped with a dust pan suction nozzle which will dredge and level off the Eastern Scheldt sea bed to the

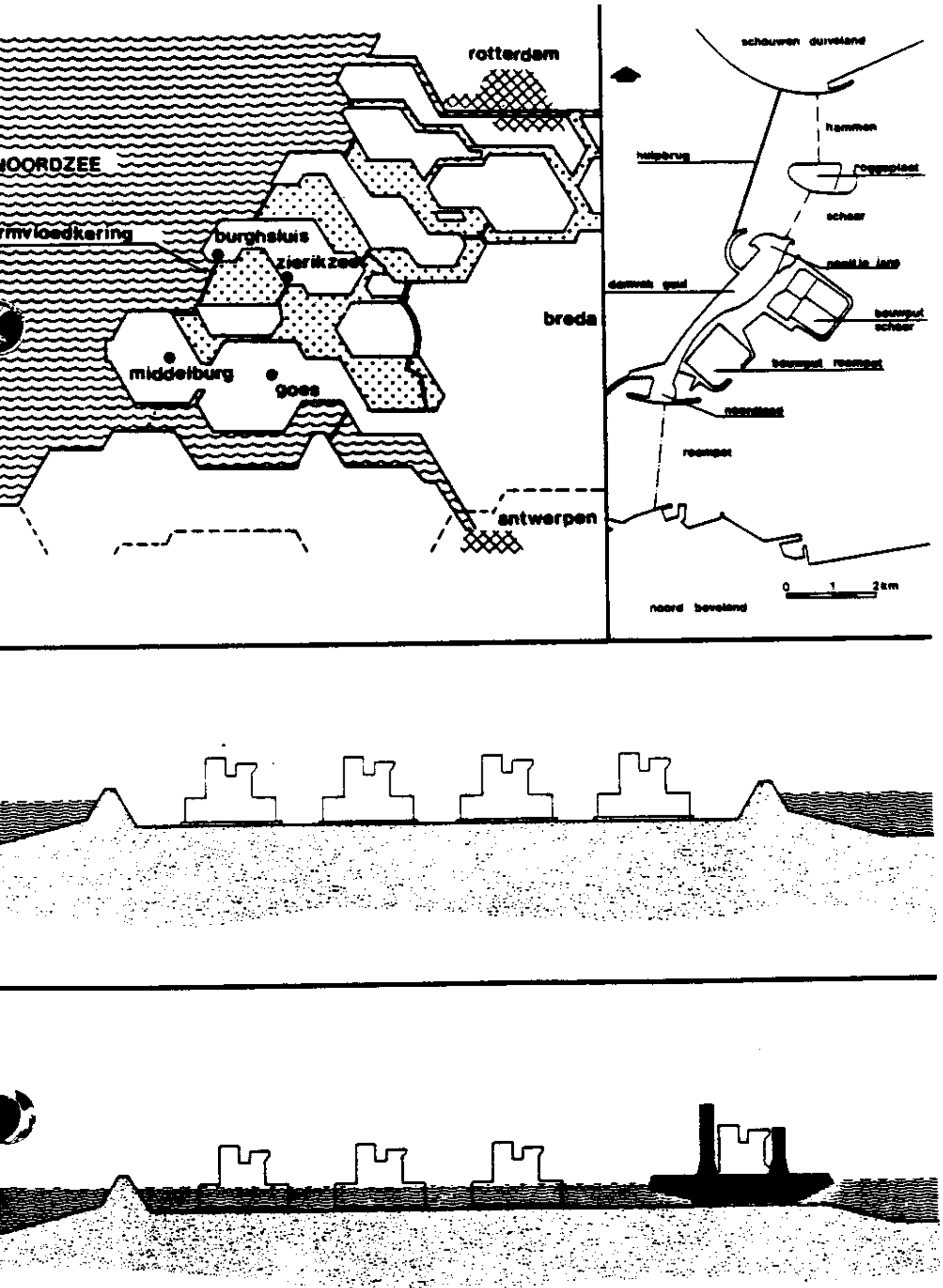


fig. 1

correct depth (see fig. 4).
The filter mats having been laid will then be compacted by a beam suspended from the **Cardium.**

Composition of filter mats

Lower mat:

Length	appr. 200	m
Width	appr. 42	m
Thickness	appr. 0.36	m
	(after compaction 0.32 m)	

The mat is composed of three layers of material, namely:

Sand	(0.3 - 2 mm)	layer thickness 110 mm
Gravel-sand	(2 - 8 mm)	layer thickness 110 mm
Gravel	(8 - 40 mm)	layer thickness 140 mm

The respective layers of filter material are separated from each other by synthetic fabric (interlays) through which water can permeate.
The underside of the mat consists of a so-called supporting fabric of synthetic material, reinforced lengthways with steel wires.
In order to achieve the required measurement nine widths of supporting fabric are stitched together.
The top of the filter mat consists also of synthetic fibre. Across the mat, within each layer of filter material, partitions are created by inserting steel wire baskets in order to prevent the filter material form shifting during winding and unwinding operations.
The top edges of these U-shaped baskets are turned inwards 180° to enclose a membrane (see photograph of scale model, fig. 5).
To consolidate the layers together into a compact mat steel pins, having a diameter of 6 mm, are vertically injected through the mat and secured on both sides (see photograph of scale model, fig. 5.). Approximately eight pins per square metre are thus injected.

272

Upper mat:

Length	appr. 60	m
Width	appr. 31	m
Thickness	appr. 0.36	m

The upper mat is assembled in exactly the same way as the lower mat except that it is composed of three layers of gravel (8-40 mm).

General

- To enclose the filter layers at both ends of the mat wooden end units are fitted to which the various fabrics are stapled. These wooden units are of the same thickness as the layers of filter material, and are joined to each other with nails and bolts.
- To facilitate the winding and unwinding of the mats steel 'handles', so-called holding beams, are to be fitted at both ends of the mats. Of these two beams the tail beam, which guides the mat on to the cylinder but which is the last to be placed on the sea bed, has to cope with a much greater load than the head beam.
  To connect the tail beam to the mat wedges of synthetic resin are moulded on to the wires in the supporting fabric.
  These wedges are fed into clamping devices on the tail beam. At the other end of the mat the supporting fabric is turned back around a steel reel which is then inserted into a clamp on the head beam, called a wrapping clamp.

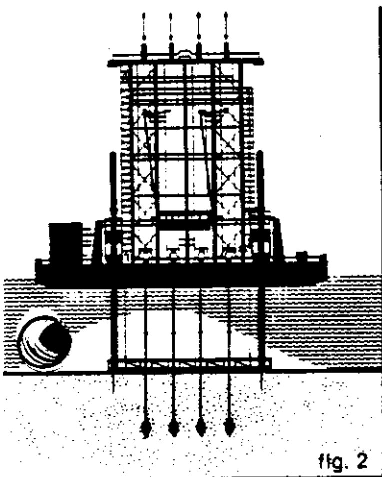


fig. 2

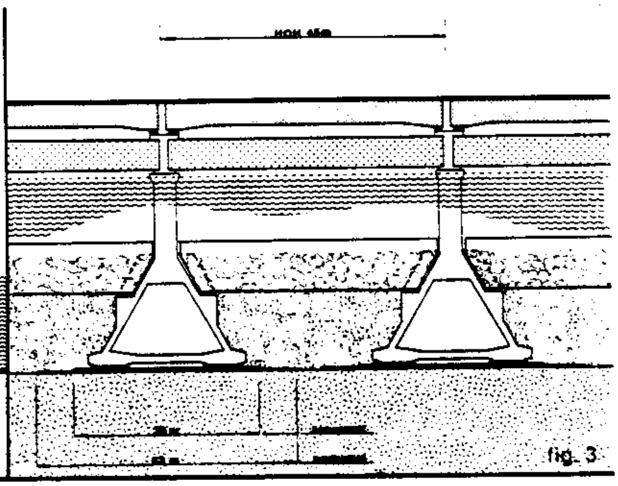


fig. 3

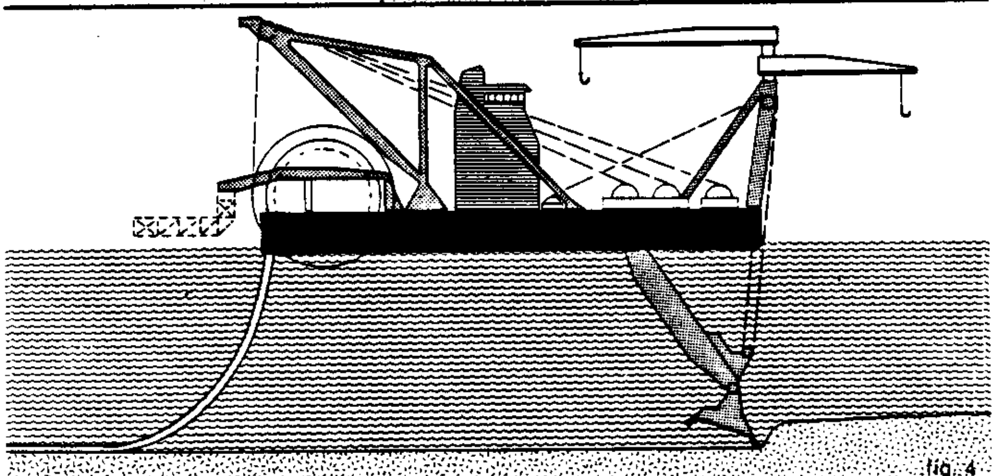


fig. 4

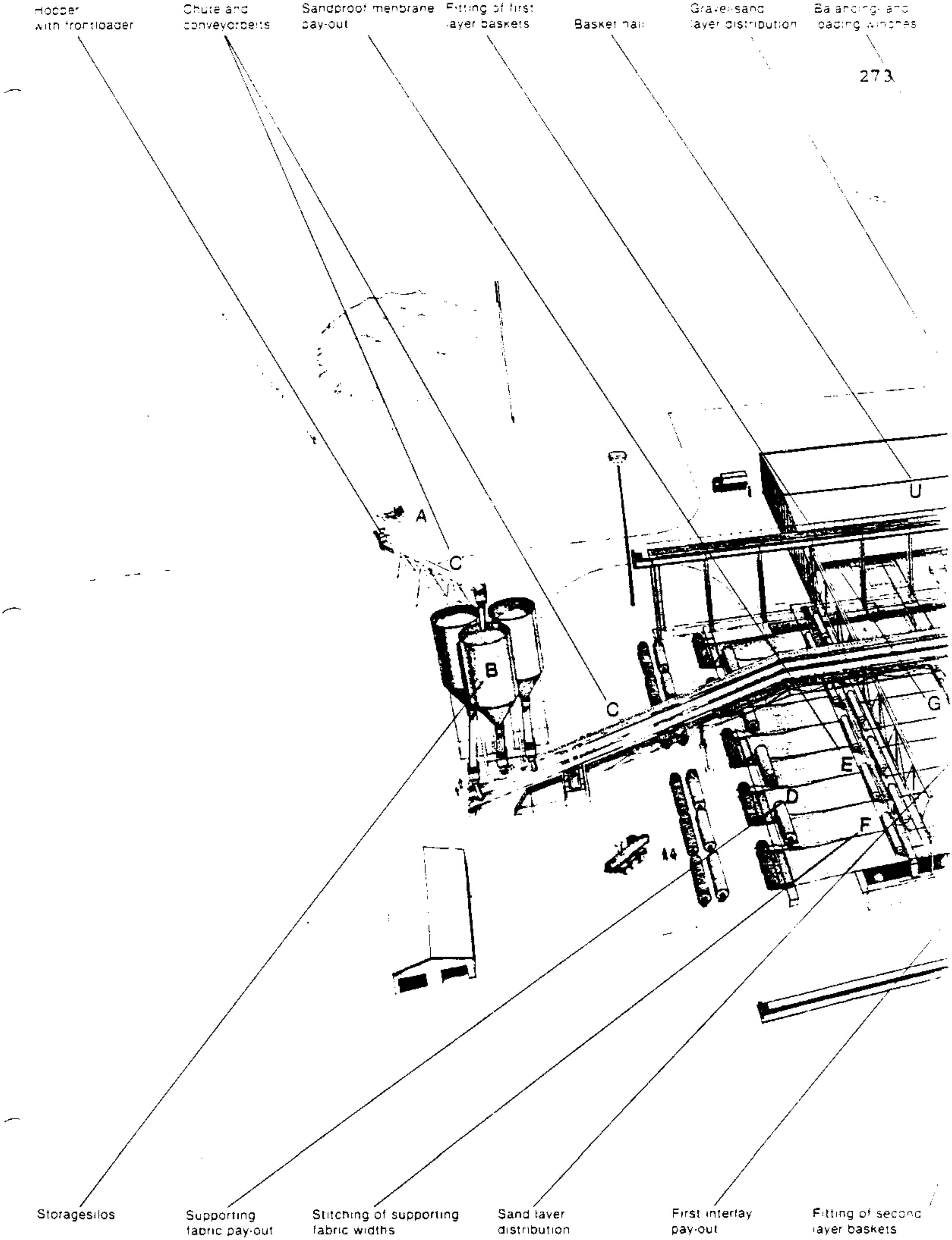
Assembling process

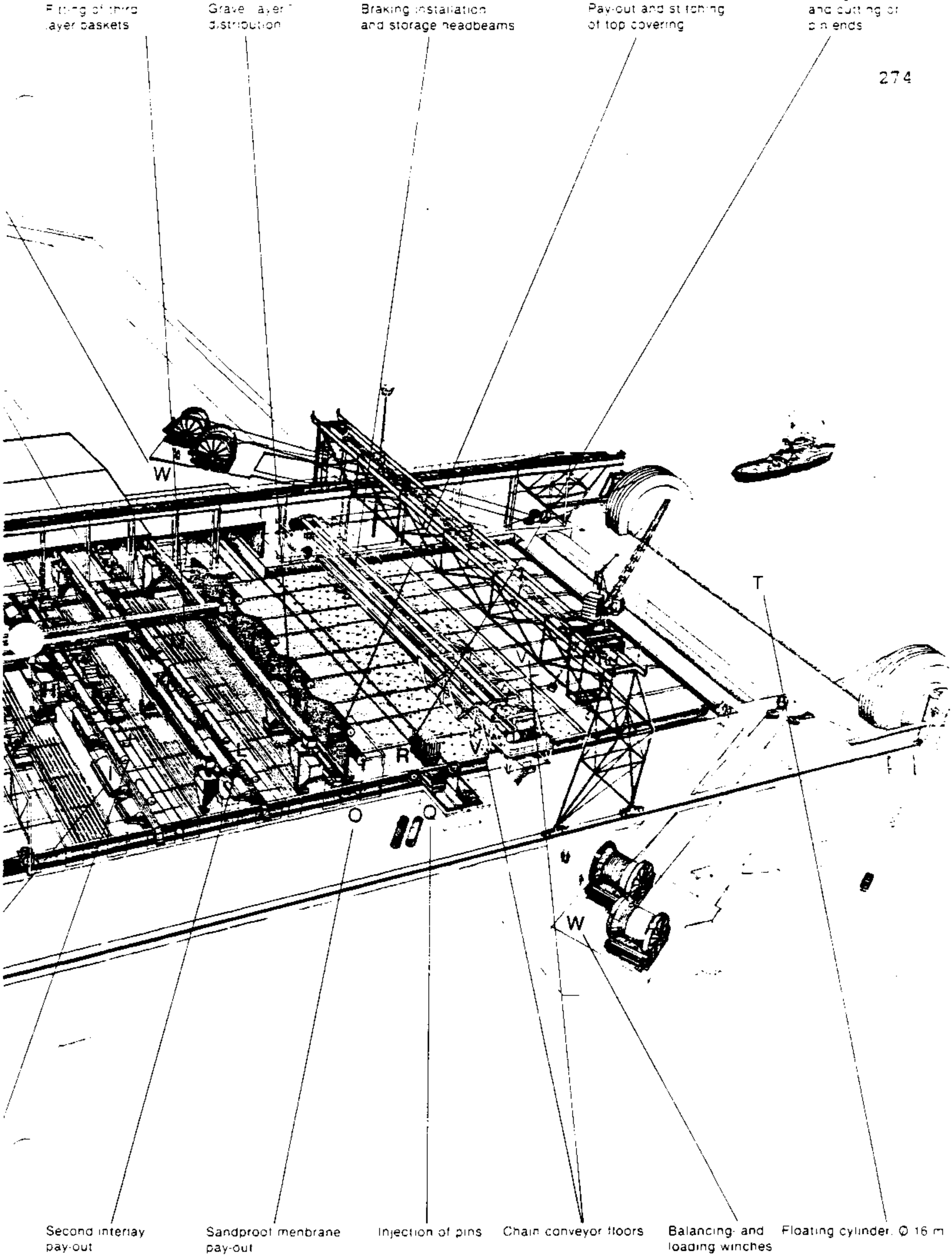
General

The assembly plant consists mainly of:
- production hall
- loading quay
- auxiliary installations

The production of the mat assembly plant is independent of the mat laying process in the Eastern Scheldt. **two** transport cylinders being used for this purpose.
The production capacity, however, is determined by the required progress at the construction site of the barrier.
This means, in fact, that on average one filter mat must be completed every 36 hours.
During the assembling process the mat is moved underneath stationary machinery. For this purpose the floors of both assembly plant and loading quay consist of nine synchronised chain conveyors driven by electric motors.
The assembling process takes place at ground level and has been phased in such a way that a two metre strip of mat can be assembled every ten minutes. The whole mat is then moved

fig. 5


Hopper
with frontloader
Chute and
conveyorbelts
Sandproof membrane
pay-out
Fitting of first
layer baskets
Basket hall
Gravel-sand
layer distribution
Balancing- and
loading winches
A
B
C
C
D
E
F
G
U
Storagesilos
Supporting
fabric pay-out
Stitching of supporting
fabric widths
Sand layer
distribution
First interlay
pay-out
Fitting of second
layer baskets


layer baskets
distribution
Braking installation
and storage headbeams
Pay-out and stitching
of top covering
and cutting of
pin ends
W
T
R
V
O
Q
W
Second interlay
pay-out
Sandproof membrane
pay-out
Injection of pins
Chain conveyor floors
Balancing- and
loading winches
Floating cylinder. Ø 16 m

forward a further two metres at the same height towards the loading quay. The conveyors inside the production hall can operate independently of those on the loading quay, so that the assembling process can be disconnected from the operations taking place on the loading quay.
The operating machinery of the plant and loading quay floors is housed in a large concrete basement which also contains some machinery for parts of the assembling process.
The time required to assemble a mat of 200 m is 20 hours, the time to handle the same mat on the quay is about 7½ hours.
During the assembling process the mat moves through the following stages:

Preparatory work · Storage and feeding in of supporting fabric

In order to connect the tail beam to the beginning of the mat wedges of synthetic resin must be fitted on to the steel wires in the supporting fabric, a connecting device similar to that used with steel cable sockets. This operation is carried out in the so-called wedging shed which is situated to the west of the assembly plant. From here the rolls of supporting fabric are then taken to the west side entrance of the plant by side loader and mounted to be paid out (D).

Stitching of the supporting fabric (F)

Nine widths of supporting fabric, each about five metres broad, are stitched together to an overall width of 42 m by eight sewing machines housed in the basement.

Positioning of sandproof membranes, interlays and top covering

With the exception of the supporting fabric all other reels of material arrive at the plant on the south side and are fitted into their pay-out position by a mobile crane mounted overhead, called the 'serving trolley'.

Fitting of sandproof membrane and interlays (I, L and O)

The nine widths of these fabrics, overlapping each other, are fitted over the sand or gravel layers. Both ends of the fabrics are stapled on to the wooden end units.
The sandproof membrane is used only in the lower mat, namely on top of the supporting fabric, at the sides and at both ends of the mat where sand is used. In the upper mat a cementproof membrane, called the upper mat membrane, is inserted.

Fitting of wooden end units

Both ends of the mat consist of wooden end units the first of which is fitted on to the supporting fabric in the wedging shed.
The wooden unit at the other end of the mat is bolted to the supporting fabric.

All interlays and the top covering are fastened with staples. The wooden end units are nailed to each other. Both staples and nails are driven in pneumatically.
The wooden end units are transported by crane and monorail system from the arrival area at the south side of the plant to the designated assembly point.

Manufacturing and fitting of baskets (U and G)

To avoid bulky storage within the plant the baskets are manufactured in the basket hall, situated north of the mat assembly plant, at the same time as the filter mats. The basket hall houses three machines which mould the steel netting and cut it to size. Both top edges of the U-shaped baskets are turned back 180° to enclose a membrane. After having been placed in position, ready to be fed in, the baskets are transported to the assembly plant by four basket transporters. These transporters are equipped with adjustable clamps which place the baskets in the correct position on the mat. Three such installations operate parallel to each other.

Depot for storage of sand, gravel-sand and gravel (A)

These materials are brought to the site by lorry. The storage capacity of the depot has been calculated ont the basis of the requirements for a seven to eight weeks' production.

Distribution system for sand, gravel-sand and gravel (B, C, H, K and N)

The materials are transferred to a hopper by a front loader and transported to the appropriate storage silos via a movable chute and a conveyor belt. Three independently operating conveyor belt systems feed the materials into three moving, distributing hoppers and four stationary hoppers which deal in particular whith the sides of the lower mats. Each time they have distributed the material on a two metre strip of mat, the moving, distributing hoppers are re-filled, which takes only two minutes, whilst the mat moves on. The stationary hoppers are re-filled only every four hours, a process which has to be completed in approximately eight minutes.
The adjustable vibrating chutes below the hoppers ensure an even layer of filter material, unaffected by the inevitable sagging of the supporting construction and chain conveyors.

Finishing the sides of the mats

The side ends of the various fabrics which, during the assembling process, have been turned up vertically are stitched together with hand sewing machines before the pins are injected through the mat.

Stitching of the top covering (P)

The widths of the top covering are stitched together while they are unrolled vertically, and are then transferred to a horizontal position. Both ends are stapled on to the wooden end units.

Injection of pins (Q)

The installation consists of an upper and an under carriage. The under carriage, 12 m long, three metres wide and 2.55 m high, running on rails in the basement, hydraulically injects 75 pins in every 5 m × 2 m area of the filter mat. A detection and positioning system ensures that the injected pins do not hit the steel wires in the supporting fabric.
The synchronised upper carriage, nine metres long, 2.50 m wide and three metres high, running on a steel construction above the mat, meets the 75 pins, fixes bowl-shaped snap-locks on to them and cuts off the sharp ends.
The whole installation has been designed and built as a result of trials and prototypes.
It is possible to inject, secure and cut 800 pins during the ten minute period when the mat is stationary.
The whole installation must, undoubtedly, qualify as a very complicated piece of equipment.

Loading quay

Head and tail beam system

As soon as the beginning of the filter mat approaches the fixed quay floor at the end of the second chain conveyor, it is connected to the tail beam by means of the fitted wedges. The tail beam itself is coupled to the filter mat cylinder (diameter 16 m) by steel cables (T).

Braking system (S)

In the final stages of the mat being wound on to the cylinder there remains only a small part of the mat on the quay. To prevent this end of the mat from suddenly sliding off the quay, as well as to control the whole winding process, a braking system has been devised. This system consists essentially of a heavy steel bridge construction, weighing 130 tons and having a free span of 48 metres, on which two heavy winches have been positioned, each capable of delivering a braking force of 150 tons. The winches are also used to pull the holding beams on to the quay (pulling force: 125 tons per winch).

Balancing and loading winches (W)

In order to keep the cylinder in balance during the winding of the mat two balancing winches have been installed. The winding operation itself is carried out by two loading winches.
The winding operation proceeds

intermittently as each two metre length of mat, which is finished and fed through every ten minutes, takes only one minute to be wound on to the cylinder. When the mat is being wound on to the cylinder, the load on each of the two winch cables will progressively increase to a maximum of 225 tons, relative to balance, mat radius, mat weight and water level.
The force in the two balancing cables, providing the pre-stressing in the cable system, is kept as much as possible to a constant 30 tons per cable.

Portal crane

The whole of plant and quay area is bridged by a portal crane, designed for use during the building as well as the running of the plant.
During the running of the plant the crane, which has a lifting capacity of 120 tons, is used mainly for operations on the quay, such as the handling of the head beam, weighing approximately 110 tons, and the tail beam, having a weight of approximately 85 tons.
The portal crane is equipped also with a separate moving platform which is used when the cables are fixed to the cylinder. The crane has a span of 60 metres an is fitted with both a fixed and a moving tackle, each of 60 tons. The moving tackle is equipped with a turning jib having a hoisting capacity of about four tons at a radius of 16 metres.

Auxiliary installations

Electro-technical installation

Two transformers, each of 1,000 kVA, supply the plant with power. They feed a low voltage main distributing installation, housed under the central control room and electrically linked to a separate emergency power supply for the deep well system and the lighting. In case of a failure of both 1,000 kVA transformers the power supply is switched to a 630 kVA transformer installed elsewhere. For reasons of centralisation, as well as to facilitate localising a fault, all distributors for the package units have been installed in the same area. All cables have been mounted in cable channels.
The covered assembly plant and the basket hall are lit by fluorescent strip lighting with an illuminance of 150 lux. On the quay, next to where the cylinders are moored, standard lamps, 25 m high, have been erected.
An intercom system has been installed between the central console and the various package unit operators, so that mutual communication is possible.
The winch operator (quay supervisor) has the same facilities at his disposal as the central operator.

Control of electro-technical installation

In view of the special safety and control regulations, the whole installation has been designed on the principle that it should be possible to operate each part locally, both manually and automatically.
The various package units transmit progress, fault and release signals to a centrally installed control and display console where the whole process can be followed and from where, after the package units have signalled release, the mat can then be moved forward. In some cases control of the electro-technical installation is carried out through the Programmable Logic Control (P.L.C.).
After the mat has been moved on the central operator signals the local operators that work can be resumed. When the mat has been attached to the cylinder, the central operator hands over control of both moving and winding the mat to the quay supervisor who is in a better position to handle this combined operation. However, the central operator still continues to supervise the whole procedure.

Compressed air installation

The compressed air installation consists of two compressors, each with a capacity of 5 m³/min. Compressed air is required for some of the operations, for the nailing and stapling machines, for tools and cleaning purposes.

Water supply

To cope with the great variation in water usage a storage tank of 40 m³ capacity has been installed which is fed by a water mains, 6" in diameter, running along the main road.
The water is required for both sanitary and cleaning purposes.

Civil engineering and constructional works/steel constructions

- **Sheet piling construction of quay**

  To build the quay an anchored sheet piling construction was used consisting of profiled sheet piling driven into the ground to a depth of 19 to 23 metres below Amsterdam Datum Point (NAP).

- **Foundation filter mat assembly plant**

  After investigations of the soil by the Delft Soil Mechanics Laboratory, it was decided to construct the foundations of the filter mat assembly plant directly on the sandy subsoil without using pile driving.

The level of the site is 4 m above Amsterdam Datum Point (NAP).
Under the greater part of the plant area a basement, housing the operating machinery for the chain conveyor floors, was to be built at 1.50 m above Amsterdam Datum Point. Consequently, a level of one metre above Amsterdam Datum Point was chosen for the foundations of the basement walls which had to bear a relatively heavy load. The maximum ground pressure allowed was 1½ kg/cm². To avoid complications from ground water pressure, since the ground water level may rise higher than one metre above Amsterdam Datum Point, a drainage system was installed.
The water is carried away by a deep well installation which must be capable of constantly coping with all overpressure over a 1½ to 2 year period during which the plant will be in operation.
Quantities of approximately 6,250 m³ of concrete and approximately 600 tons of reinforcement steel were used.

- **Steel constructions**

  An open lattice construction was chosen with lattice girders running across the plant.
  The greater part of the installation is fixed, or moves, at approximately the same level as the underside of the lattice girders.
  Initially, the specifications stipulated that only vital parts of the plant should be protected from the weather by coverings. Later, however, it was decided to roof the whole of the whole of the assembly plant.
  The basket hall, situated to the north of the plant, was built as a standard hall, complete with front and side walls.
  The total weight of the steel used for buildings and supporting constructions is approximately 1,200 tons, exclusive of the chain conveyor floors which weigh about another 1,000 tons.

Investments

Investments for the plant, complete with all installations, amount to approximately £19 million about half of which was needed for installations.

Main contractors and suppliers

Design	Tabodin, The Hague Dosbouw, Burghsluis
Civil engineering	Dosbouw, Burghsluis
Chain conveyors	Koch Transporttechnik Wadgassen/Saar
Steelconstructions/ platforms	Demy, Geleen
Portal crane	Hollandsche Beton Maatschappij Afd. Materieel, Amsterdam
Electrical measuring/ regulation installations	Croon, Rotterdam
Distributing installation for sand, gravel-sand, gravel	Technische Industrie TBO, Ede
Pin injecting, securing/ cutting installation	Aarding, Nunspeet, in co-operation with Tekon, Haaksbergen
Basket construction	Bekaert, Zwevegem
Basket transport installation	Mason, Son
Balancing/loading winches	V. d. Graaf, Zwijndrecht
Fabric feeding-in system	Hollestelle, Goes
Roller conveyors	Muider, Boskoop
Winch bridge	Demy, Geleen
Holding winches	Demy/KEMACH, Lemmer
Make sewing machines supplied by Union Special	Van Vliet, Goes
Plumbing	Technisch Bureau Verburg-Holland, Colijnsplaat
Compressed air installation	Hollestelle, Goes
Steel wires	Staalkabel, Amsterdam
Basket hall	Remco, Valkenswaard
Sheaves	Kreber, Vlaardingen
Cabins	Metamo, Barneveld

Materials for the assembly of the filter mats are supplied by:

Supporting fabric	Robusta, Genemuiden
Wire netting for baskets	Bekaert, Zwevegem
Interlays	Woudenberg, Dieren Robusta, Genemuiden
Top covering	Nicolon, Enschede
Wire lock resin	Staalkabel, Amsterdam
Snap-locks	Bakker, Hengelo
Pins	Sico, The Hague

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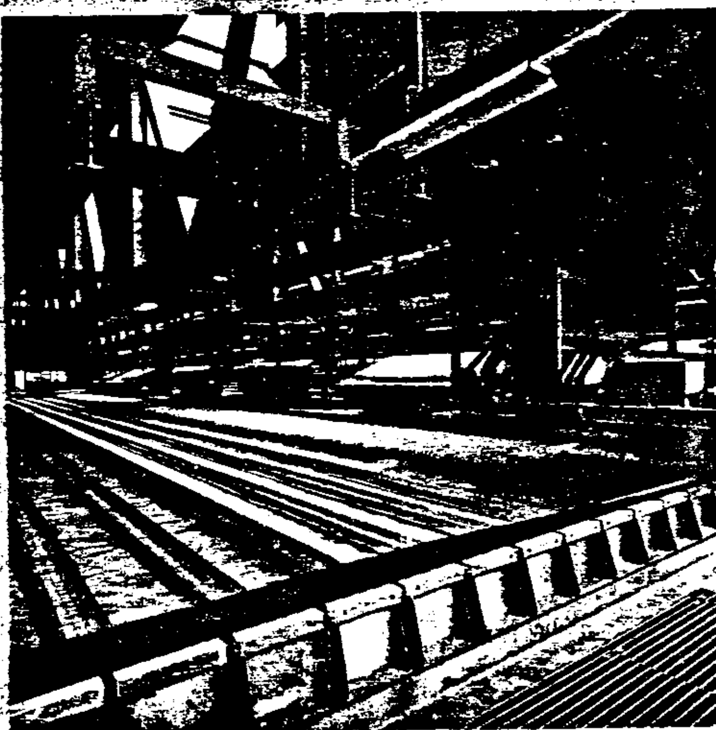
Many industries were involved from the engineering stage onwards, so that specific knowledge and expertise could be called upon at the earliest possible moment.
It will be clear from the above check list that the filter mat assembly plant for which many parts had to be specially developed is virtually an exclusive product of Dutch industry.



COLOPHON:

Design: Studio Koos van Leeuwen, Bleiswijk
Artist impression: Unicon, Den Haag
Fotograph: Jack van Bodegom, Serooskerke
Print: B.V. Drukkerij v/h Lakeman & Ochtman, Zierikzee

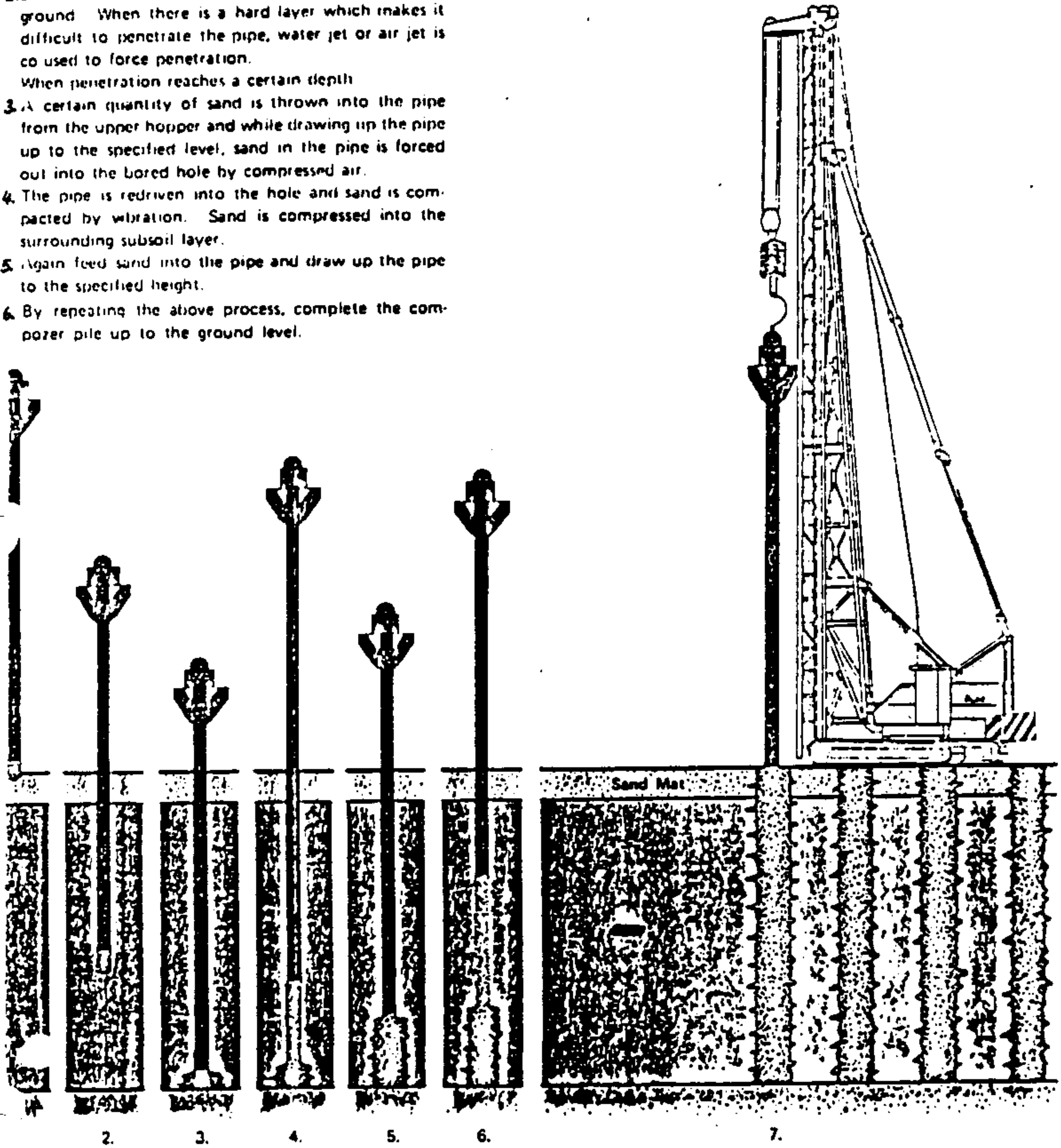
APPENDIX B

APPENDIX B

Appendix B represents a schematic method for the stabilization of soil: the Compozer method. It enlists the different steps executed to provide a more stable foundation by vibrating a pipe placed at the site where sand is injected in it. This sand is then compacted by vibration.

Order of Compozer Execution Steps

1. Install the pipe at the specified site on the ground.
2. Excite the vibrator to penetrate the pipe into the ground. When there is a hard layer which makes it difficult to penetrate the pipe, water jet or air jet is co used to force penetration.
When penetration reaches a certain depth
3. A certain quantity of sand is thrown into the pipe from the upper hopper and while drawing up the pipe up to the specified level, sand in the pipe is forced out into the bored hole by compressed air.
4. The pipe is redriven into the hole and sand is compacted by vibration. Sand is compressed into the surrounding subsoil layer.
5. Again feed sand into the pipe and draw up the pipe to the specified height.
6. By repeating the above process, complete the compozer pile up to the ground level.



APPENDIX C

APPENDIX C

CONTRACT SPECIFICATIONS

Appendix C consists of typical contract specifications for the construction of breakwaters.

The first part of this Appendix (pp.303 to pp.324) is an extract from the specifications for the proposed Atlantic Generating Offshore Nuclear Power Station, as prepared by Frederic R. Harris for the Public Service Electric and Gas Co. of New Jersey.

The second part represents a draft of specifications prepared by the U.S. Corps of Engineers for the rehabilitation of Half Moon Bay breakwater (pp.325 to pp.333).

3.15 CONSTRUCTION REQUIREMENTS

3.15.1 BREAKWATER

3.15.1.1 Breakwater Stone and Fill Materials

The types and sizes of stone and fill material will be as follows:

Type	Size and Weight	Description
I	#200 Sieve to 3#	Caisson Sand Fill
II	#10 Sieve to 6"	Filter Layer
III	3" to 6"	Leveling Course
IV	20# to 400#	Core Stone
IX	6^T to 10^T	Armor Stone
X	10^T to 15^T	Armor Stone
XI	600# to 1600#	Stone
XII	1.5^T to 4^T	Armor Stone

Type I material will consist of particles of sand or gravel with no more than 10 percent fines passing the number 200 sieve and no size 1 larger than 3 inches. The material shall consist of natural inert materials that are clean, hard and durable.

Type II fill material will consist of particles or fragments of granular aggregates.

Type III, IV and XI stone will consist of quarryblasted ledge rock that is sufficiently durable. The rock will be free from open fissures, planes of weakness, all foreign material and other undesirable qualities which might contribute to crumbling or breaking during handling, placing or weathering.

Type IV core stone will be rough and angular in shape with the least principal dimension not less than one-fourth (1/4) the greatest dimension. Type IV core stone will be tested to determine its shear strength. The testing will consist of a series of consolidated drained triaxial tests. The 12-inch diameter triaxial cell of Geotesting, Inc. of San Rafael, California is currently being used to test various stone for acceptability in the breakwater. Type XI material will consist of uniformly graded rock with not less than 50 percent by weight, in pieces weighing the average weight (1100 lbs.) of that type stone.

Types IX, X and XII Armor Stone will consist of select quarry rock. The Armor Stone will be free from open fissures, planes of weakness, all foreign material and other undesirable qualities which might, contribute to crumbling or breaking during handling, placing or weathering. No less than 50 percent by weight will be in pieces weighing more than the average weight of that type. Armor Stone will be rough and angular in shape with the least principal dimension not less than one-third (1/3) the greatest dimension.

Extensive investigations indicate that there are a number of sources 1
of materials suitable for breakwater stone and fill, and that such sources are worked by experienced owners, each capable of meeting the supply requirements of the present project. The specific type(s) of natural material to be used, and the actual supplier(s) of this material, have not been identified at the present time. It is expected that this information will be furnished in September 1974.
Consequenly the present response is outlined in a generic manner. 9

A qualitative evaluation of rock durability relates to the geological origin and history of the rock; and is based on, in the first instance, a petrographic examination of the mineral and crystallographic makeup of the rock. Larger scale macro studies of hand specimens and field observations of large fragments are made to determine weak seams, such as poorly healed joints or fractures and incipient cracks. All of the above factors influence the potential durability of the rock. A further criterion is the construction history of the rock under consideration in any marine structure.

All of the quantitative ASTM tests listed below, relate to a greater or lesser degree, to durability:

a. The Los Angeles Abrasion Test ASTM C-535. (The ability of rock fragments to remain intact when subjected to the extensive abrasion and impact which occurs when these fragments are rotated with iron balls in a drum, reflects the toughness and durability of the rock.)

b. The Soundness Tests by Magnesium and Sodium Sulphate - ASTM C-88. (These tests determine the resistance of rock to disintegration by saturated solutions of magnesium and sodium sulphate. The tests are designed to simulate weathering action and to supplement information on the service records of the material when exposed to actual weathering conditions.)

c. Test for Absorption and Bulk Specific Gravity - ASTM C-97. 1
(This test provides an indication of the rocks porosity and is a measure of resistance of stone to freezing and thawing. The specific gravity provides a measure of a fragment susceptibility to movement and indirectly relates to mechanical abrasion in the breakwater.)

d. Test for Compression Strength - ASTM C-170. (This test provides an index of the general toughness of the rock element and so also is an index of durability.)

The following discussion presents in a logical manner the requirements of suitable rock and identifies sources of rock available for use in the Atlantic Generating Station breakwater.

Possible sources of caisson fill are the loose sea floor sands in the general vicinity of the plant site. These sands are chemically and mechanically stable. However, the most probable source of fill would be from presently operating sand pits in the New Jersey Coastal Plain.

The types of rock most suitable for constructing a protective breakwater for the Atlantic Generating Station are: (a) those which have a source favorably located for economically producing fragments of rubble of the required sizes and (b) those types of rock sufficiently tough to withstand both the short-term processes associated with quarrying, transportation and placing and the longer term processes associated with the life cycle of the breakwater.

Geologically, the most suitable materials are derived from massive,
fine-grained igneous rock such as diabase or basalt; the massive
metamorphic rocks such as greenstone and quartzite and the massive
sedimentary rocks such as hard sandstone and limestone. Coarser-
grained igneous and metamorphic rocks such as granite and grantic
gneiss are also suitable. Lithologic suitability applies especially
to the larger size ranges of rubble, since for the most fine-grained 1
igneous and sedimentary rocks, their inherently closer joint spacing
or bedding tends to make the production of stone above 8 or 10 tons
impractical.

Highly foliated rocks such as schist, micaceous gneisses and some amphibolites, as well as thinly bedded, sheared, and excessively jointed rocks are not suitable, since these are likely to shatter on blasting, producing a large number of flat fragments and a high percentage of fines.

The importance of the characteristics of natural discontinuities (i.e. jointing, bedding, etc.) in the rock mass cannot be overstressed, since these control not only the ability of the rock to break into angular roughly equidimensional fragments showing suitable surface roughness; but they control also the distribution of fragment sizes (gradation) in the production blast. Such properties, if favorable, contribute significantly to a relatively dense rockfill with a relatively high angle of internal friction.

Suitable breakwater stone is further geologically characterized by being resistant to physical and chemical weathering. Rock with a minimum of void spaces is usually least prone to frost action. Chemical weathering involves rock decay accompanied by marked changes in chemical and mineralogical composition. Over a geological time span, the complex silicates that make up the bulk of igneous and metamorphic rocks commonly change into hydrous silicates, hydrous oxides, and carbonates; some materials, such as carbonates, are lost by solution. With the exception of carbonates, the susceptibility of a rock to chemical weathering is related to the temperature at which the rock formed. Higher temperature rocks (igneous and metamorphis) are more susceptible than lower temperature rocks 1
(sedimentary). Except under extreme climatic condition (polar or tropical), weathering is a very long-term process and the possibility of weathering influencing any sound rock placed in the breakwater during its 40 year life cycle is extremely remote.

Favorable geological rock types relate to materials with favorable engineering index properties. In other words, properties such as mass per unit volume, compressive strength and durability in terms of mechanical and chemical stability, are a function of the mineral composition, texture, and macro and micro structure of the rock element. Favorable properties are generally associated with such features as fine grain size, a lack of micro-fracture or well-developed cleavage, hard stable minerals, isotropic mineral arrangement, and good crystal interlock. Conversely, less well suited rocks are associated with coarser grain size, the presence of micro-fractures, unstable soft minerals, an abundance of minerals with well-developed cleavage, a well-defined anisotropic mineral arrangement, a clastic texture, and other deleterious properties. It may be emphasized, however, that the petrographic features described above should be used to provide guidelines in selecting suitable rock and not to establish rigid diagnostic criteria for needlessly condemning potential useful sources.

Many rocks having one or more of the features sited as being characteristic of a poorer material have been satisfactorily used in marine environments. In this case, a study of the service records of the rock in existing marind environments could be conducted to establish a realistic evaluation of that particular rock.

Although the exact source of breakwater stone has not yet been determined, the stone selected will be one of the following types: L) diabase-b asalt, 2) New England granite or, 3) granitic gneiss. The susceptibility of a rock type such as diabase, granite or granitic gneiss to weathering or breakdown in a salt water environment would be almost nonexistent. Most chemical weathering phenomena occur in an acid environment (pH 7.0) under more favorable oxidation conditions and where precipitation containing dissolved CO_2 acts as the solvent. In the ocean, however, the pH is always in the neighborhood of 8.0 as the combination of several solutes (most notably the CO_2-H_2-CO_3-HCO_3-CO_3 system) serve as effective buffers. This inhibits the rapid influctuation of pH and has probably caused equivalent conditions to prevail in the oceans for a long time in the geologic past.

While the dissolution of minerals within diabase, granite or granitic gneiss in a salt water environment is feasible thermodynamically, it is kinetically improbable. The concentrations of anions and cations already present in sea water (and common to those in the subject crystalline rocks) can approach saturation at certain temperatures and pressures. Such a condition would make it highly unlikely that solution of any of these minerals would take place. This is evidenced, in part, by several petrographic analyses of rock specimens from already existing breakwaters. Clinton Point Dolomite from the Jones Beach Breakwater (N.Y.) and granite from the Hyannis Port Breakwater (Mass.) (granite from Stonington, Maine) were analyzed in thin section and showed no mineral alteration from up to 40 years exposure to a salt water environment.

On a relative scale, Reiche (Reference 3.15.1-4) proposed a weathering potential index, WPI, for rocks. Those rocks with a high stability (resistance to weathering attack) have a low index . Conversely, those rocks and minerals with a low stability (less resistance to weathering attack) have a high index. Shown below is a tabulation of the WPI for several common minerals and rock types:

Material	WPI Average
Quartz	1
Granite	7
Granitic Gneiss	10
Basalt (diabase)	20
Olivine	54

The interpretation of such an index would show that a basalt or diabase would weather three times more rapidly than a granite under the same weathering conditions. It does not indicate, however, the length of time involved in the chemical breakdown of minerals that make up these crystalline rocks. Studies have shown that most saprolites (weathered rock) were formed during the Tertiary period, up to 10 million years ago (Reference 3.15.1-5).

The properties of the rock element may be evaluated in quantitative terms in accordance with standarized laboratory procedures. Specific gravity is an important index property in this respect since the specific gravity of a rock is directly related to its resistance to movement. An illustrative example is discussed by Treasher (Reference 3.15.1-1) who compares two similar sized stones, one having 10% greater density in air than the other. The heavier stone, when immersed in water has 50% more resistance against movement by wave action than the lighter stone. A specific gravity of greater than 2.6 is desirable.

The specification requirements and test procedures for the various types of breakwater materials to be used at the Atlantic Generating Station are given in Table 3.15-1.

Design suitability of rock is evaluated in terms of the efficiency with which individual rock fragments recombine and act as a single rockfill mass. The critical design parameter in this respect is the angle of internal friction which defines the strength of the rockfill structure. The highest angle of internal friction is naturally desirable. Determination of this parameter should be consistent with the confining stresses and density anticipated for the core material of the breakwater.

A broad sweep of the geological brush indicates that the northeastern United States are adequately endowed with sources of highly suitable basalt, diabase, gneiss, dolomitic limestone and granite on or within easy reach of water transportation. Such sources are worked by several well recognized and experienced owners, in whom employ competent staffs and would be capable of expanding existing production to meet the schedule necessary for the construction of a two unit breakwater. Although existing suppliers are well-qualified and capable of providing the necessary rock, the development of a new quarry in the northeastern states is still considered a viable alternative at this time.

There are 135 principal crushed stone quarries in the Northeastern
United States (Reference 3.15.1-2) producing 100,000,000 + tons/year. 1
These quarries are all in rock types described previously as being well-suited for breakwater stone or listed in Corps of Engineers records as being potential sources of aggregate. Several of these quarries have produced large broken rubble in the past and could produce such rock in the future. Also many of these quarries are on or have good access to water and/or rail transportation. Thus the Atlantic Generating Station site has access to a sufficient number of alternative sources of suitable rock within economical haulage distances to make the concept of a rubble mound breakwater highly feasible.

Table 3.15-2 provides additional data on operations considered to be most suitable for supplying breakwater materials. The quarries listed were selected from the principal crushed-stone producers in the region as being best potentially able to provide the required amounts of stone for the least cost and with the least environmental impact (Reference 3.15.1-3).

For location in the breakwater cross-section of stone types, sizes, and weights listed in Sec. 3.15.1.1 and in Table 3.15-1, see Fig. 3.12.5-3.

Based on model tests at the Waterways Experiment Station (WES) in Vicksburg, Mississippi, recommendations were made for a typical main breakwater cross-section, a typical cross-section at the roundheads and a typical cross-section of the closure caisson showing caisson fill material. These recommended sections are shown in Figs. 3.14.2-2, 3.14.2-4, and 3.14.2-5, respectively.

The recommended cross-section for the main breakwater shown in Fig. 3.14.2-2 is based on a prototype which compares to the actual design cross-section as follows: 9

Prototype	Actual
37 ton dolos	40 ton dolos
7 ton rock	6^T to 10^T
736# rock	600# to 1600#
37#	20# to 400#
5 ton rock	6^T to 10^T
1 3/4 ton rock	1.5^T to 4^T

The recommended cross-section for the roundheads shown in Fig. 3.14.2-4 is based on a prototype which compares to the actual cross-section as follows:

Prototype	Actual
62 ton dolos	62 ton dolos
12 ton rock	10^T to 16^T
1500# rock	60# to 1600#
20 to 400# rock core	20# to 400#

As seen from the above comparisons, the recommended breakwater cross-section is exactly the same as the design cross-section for the roundheads and is almost the same for the main breakwater cross-section. In the actual main breakwater design cross-section, the dolos size has been increased from 37^T in the prototype to 40^T and slightly heavier rock (6^T to 10^T as opposed to 5^T in the prototype) is used as a primary layer topping the caissons. In both of the above cases, the changes are on the conservative side and provide for a more stable cross-section. 9

The design cross-section of the material in the closure breakwater caissons were changed to provide 2 additional layers of stone (Core Material - 20# to 400# range and filter stone, #6 sieve to 6") under the 1.5^T to 4^T armor stone and the design cross-section of the material in the main breakwater caissons was changed to provide one additional filter layer (#6 sieve to 6") under the 600# to 1600# stone. In both cases, additional screening layers of stone were placed over the caisson sand to prevent the possibility of sand liquifying and creating a quick condition whereby the heavier stone would sink into the underlying sand.

3.15.1.2 Construction Methods For Breakwater Stone and Fill Materials

The seabed beneath the caissons will be prepared by placing Type IV material to a rough grade one to three (3) feet below the elevation of the bottom surface of the caissons. Concrete grade (leveling) sills or screed frames will then be placed to the elevation of the bottom surface of the caissons in such an arrangement as to establish a finished grade line for the base of the caissons. Type III material will then be placed on the Type IV material and screeded to final elevation.

Type IV rock will be placed underwater by dumping from scows until the rock is so high that the scows could not float. After that stage has been reached, it will be placed by clam shell buckets. Material will be deposited in such a manner as to insure a dense core. There is no assurance that material will naturally come to rest on exact slopes of 2 horizontal to 1 vertical. Consequently, most of the slopes will need to be dressed to the required configuration by the use of clam shell buckets. 9

Type XI material will be deposited by dropping or dumping. The outer limits of the material will be covered with the largest sizes of each type and shall be rough, to achieve keying between layers.

Types IX, X and XII armor stone will be individually placed so that adjoining pieces will be firmly bedded on the stones below and be in juxtaposition with each other, to create interlocking of adjoining pieces where possible. Materials will be placed at the lowest elevations first, working up toward the higher elevations and shall be laid in horizontal layers. Materials will be placed as closely together as practicable with vertical joints staggered wherever possible.

The tentative stages of breakwater construction are as follows:

(a) Stage I - Dredge seabed.

Place core material and leveling course to elevation of underside of main breakwater caissons and closure breakwater caissons.

Place main breakwater ciassons (Type I) and caissons Type IV and V of closure breakwater.

Place core material, filter layer, armor stone and dolosse to elevation -19 feet.

(b) Stage II - Place core material, filter stone, armor stone and dolosse to elevation +37 feet for main breakwater rubble mound.

Place temporary sheeting for circulating water discharge sections and basin. Excavate and place Type III leveling course material.

Place circulating water discharge basin and backfill.

Place mooring caissons, Types VI and VII. 1

(c) Stage III - Dredge, place core material, leveling course and caissons Type II and III.

Place armor stone and dolosse to elevation +66 feet (final elevation) for main breakwater rubble mound.

Breakwater construction is tentatively staged in the above order for the purpose of insuring stability during construction and the corresponding increase in strength of the underlying clay. The placing of mooring caissons and circulating water discharge basins has been included under breakwater construction stages because of the proximity of those structures to the breakwater itself.

Filling operations under Stage I involve seabed preparation which includes placing the leveling course Type III material. Each caisson will be placed as soon as the seabed is preapred for their placement.

Immediately after sinking, caissons will be filled with caisson sand fill, Type I material. A minimum time before stone filling is allowed will be established in order to permit consolidation of sand fill. An instrumentation program will be established to monitor the soil properties during construction.

3.15.2 CONCRETE DOLOSSE

Concrete for dolosse will be proportioned, mixed and placed in accordance with the requirements of Section 3.15.4. Concrete for dolos units will have a compressive strength at 28 days of not less than 5,000 psi.

Forms for the casting of dolos units are to be made of steel. Each dolos unit will be poured in one continuous operation. There will be no cold joints and retempering of concrete will not be permitted.

Except for lifting loops for removal of dolosse from their forms, 1
there will be no steel or any other material embedded in the dolos armor units. No reinforcing steel is required for the dolosse.

Dolosse will be stored after removal from the forms until the time they have achieved at least the compressive strength specified for 28 days and have been approved as to quality and acceptability up to that point in their history. Thereafter they will be moved to the breakwater for final placing.

Dolosse will be placed on the prepared stone slopes, beginning at the toe of the slopes and working upward. Dolosse will be individually positioned.

The density of the dolosse units on the surface of the breakwater slopes will be maintained by placing the correct number of units per 1,000 square feet of surface area, in double layer pattern. The required number of units per 1,000 square feet will be as follows: 27 for 11 ton units; 11 for 40 ton units; and 8 for 62 ton units. In general, 55 percent of the dolosse units will be placed in the first layer.

Equipment utilized for placing dolosse on the breakwater surfaces will permit the dolosse to swivel to any random position so there will be no systematic alignments of the dolosse units in place in any area except as otherwise stated herein.

The first dolosse rank at the toe of the slope will be placed with the outer flukes standing vertically and their inner flukes lying horizontally. The remaining units will be random placed, not packed in a pattern. This specific placing will be maintained completely around the toe of the breakwater, around the roundheads of the breakwater and along the inside of the breakwater on the top of the caissons so as to completely surround both the main and closure 1
breakwaters.

Concrete for dolosse manufacture will weigh a minimum of 150 pounds per cubic foot.

3.15.3 CONCRETE CAISSONS

Concrete caissons shall be of the following types:

(a) Type I - Stationary Caisson in Main Breakwater (Bottom elevation - 34 ft.)

(b) Type II - Stationary Discharge Caisson in Closure Breakwater (Bottom Elevation - 58 ft.)

(c) Type III - Removable Caisson in Closure Breakwater (Bottom Elevation - 38 ft.)

d) Type IV - Stationary Caisson (for Power Cables) in Closure Breakwater (Bottom Elevation - 58 ft.)

(e) Type V - Stationary Caisson in Closure Breakwater (Bottom Elevation - 38 ft.)

(f) Types VI and VII - Stationary Caissons for the Mooring System (Bottom Elevation - 44 ft.)

All materials required for the fabrication of ciassons will be in accordance with the requirements of Section 3.15.4 and as specified herein. Caisson concrete will be air-entrained and have a minimum compressive strength of 4,000 psi at 28 days. Reinforcing steel will conform to the requirements of ASTM A-615, Grade 60. Welded steel wire fabric will conform to the requirements of ASTM A-185. Steel sheet piling to form the closure between breakwater caissons will conform to the requirements of ASTM A-328.

A certified model test laboratory will be engaged to conduct towing 1
tests for concrete caissons. The objective of the testing will be to obtain the following information:

(a) The maximum sea state for safe towing of the caisson in the open sea.
(b) The maximum moments and forces induced into the caisson during towing.
(c) The number and size of the tugs required to safely tow the caisson through the maximum sea state conditions.
(d) The optimum ballasted conditions for the caisson during various sea states.

Templates will be used between the main breakwater caissons to gauge the dimensions and aid alignment in placement and shall have absorption characteristics sufficient to counter the lateral forces of impact anticipated during placement. Templates will be of such dimensions as to protect the full width of the caissons.

The caissons for the main breakwater will be placed first so as to form a protective harbor as quickly as possible.

The two removable caissons in the closure breakwater on the Unit No. 2 side of the closure breakwater will be in place prior to the arrival of Unit No. 1 at the site. The two removable caissons on the Unit No. 1 side will be installed after Unit No. 1 is in place.

3.15.4 CONCRETE WORK

The cement to be used will be a Type I or Type II Portland Cement. It will meet the minimum requirements established in the American Society of Testing Material Specification No. C-150.

Both large and small aggregates will consist of natural inert
materials that are clean, hard and durable, free from organic matter 1
and uncoated with clay, dirt or any deleterious materials. They will meet the requirements of the Standard Specifications for Concrete Aggregates of the American Society for Testing Materials C-33 except as noted.

The coarse aggregate will be either natural occurring gravels of which a minimum of 40% is crushed or be 100% quarry crushed stone.

The following criteria will apply for all fresh water and/or ice that may be required:

(a) It will not contain more than 100 ppm each of chlorides, sulfides, and nitrates.
(b) Its turbidity will not exceed 2,000 ppm.
(c) It will be potable.
(d) Contain no deleterious substances.

Admixtures being considered for use in concrete include an air-entraining agent and a water reduction and/or retardation admixture. If used, the admixtures shall conform to ASTM Specifications No. C-260 and C-494 respectively.

All reinforcing steel will conform to "Standard Specification for Deformed Billet-Steel Bars for Concrete Reinforcement", ASTM A-615, for Grade 40 and 60 steel.

A liquid membrane curing compound, if used, shall conform to and be certified to meet ASTM C-309.

3.15.5 MOORING SYSTEM

Mooring System concrete will have a minimum compressive strength of 5,000 psi at 28 days and conform to the requirements of Section 3.15.4. I

Reinforcement will conform to the requirements of Section 3.15.4.

Sand fill material will be Type I material as described in Section 3.15.1.1.

Mooring system steel and rubber bumpers shall be as described in Section 3.13.

Mooring caissons will be constructed in accordance with the applicable provisions of Section 3.15.3.

Mooring caissons shall have a maximum draft during tow of thirty-two (32) feet. Therefore in order to meet the maximum draft requirement it is anticipated that portions of the mooring ciassons may be cast at the site. Appropriate procedures will be established for site concrete casting.

All pontoons or other auxiliary buoyancy tanks used in the towing of the caissons shall be designed for the stresses imposed during tow and placement of the caissons. After positioning the caissons will be seated on the bottom by flooding the interior compartments as soon as possible.

A survey to locate accurately the position of the caissons will be made after the caissons have been set. When the survey has been made, the buffer assembly will be positioned in the caissons to meet the following criteria:

(a) The center lines of the shaft in the caisson will be set to an elevation of plus or minus one inch of the design location.
(b) The horizontal strut pins on the caisson side will be positioned so that the axis of all four pins are in a straight line and parallel to each other. 1
(c) The projection of the axis of the horizontal pins of the second caisson of the mooring system will be in the same plane as the horizontal pins of the first caisson and will intersect the line projection of the four horizontal pins of the first caisson at 90 degrees.

When the mooring caisson has been set and surveyed, the panel assembly shall be installed. It is intended that these units be set in the dry with the use of temporary bulkheads. The steel panel with the two shafts and assembly retaining the strut vertical pin shall be set in position. The shafts will be set into the slots provided in the concrete walls so that they are centered in the holes of the walls after the placement of concrete. The split bearing rings at the shaft ends and the concrete wall liners at these locations will then be installed. The shaft collar rings in either side of the steel grillage are then fixed.

Alignment of the horizontal pins will be checked to determine that they are all in a straight line. Adjustment may be made by moving the panels within their respective walls laterally.

After the above alignment, measurements between the concrete walls and the face of the bearing panel will be made. The steel sheets which form the slide housing for the rubber bumper units will be accurately positioned to allow insertion of the rubber bumpers, so that the bumpers are force fit against the panels. When the steel sheets are positioned, the space between the walls and the plates will be grouted.

The steel stops will then be positioned accurately and the space between the back plate and the concrete walls grouted. 1

The rubber bumpers will be inserted into the slide housing and force fit.

After positioning, the shafts and buffer assmely in accordance with the foregoing, the "U" shaped slots in the concrete walls surrounding the buffer assembly will be cast in the dry.

A temporary mooring system will be designed to hold the plants in position after they are placed in the basin and until the permanent mooring system installation is complete.

The bracket attachments to the Plant Hull will be installed on the Hull prior to its arrival at the site.

LIST OF REFERENCES

3.15.1-1 Treasher, Ray, C., 1964, Geologic Investigations for Sources of Large Rubble in Trask, P.D. and Kiersch, G.A., editors Engineering Geology Case Histories: Vo. 4, pp 273-286.

3.15.1-2 U.S. Brueau of Mines, 1972, Minerals, Yearbook. Vol. II, 1
Area Reports: Domestic, 1970; U.S. Dept. of Interior, 807 p.

3.15.1-3 Dames & Moore, 1973, Preliminary Report, Rock Borrow Investigation - Atlantic Generating Station.

3.15.1-4 Reiche, P., 1950; "A Survey of Weathering Processes
and Products"; New Mexico Univ. Publ. in Geology, 9
No. 3, 95 pp.

3.15.1-5 Carroll, D; 1970; "Rock Weathering"; Monographs in Geoscience, Plenum Press, N.Y., N.Y. 200p.

STANDARD FORM 23
JANUARY 1961 EDITION
GENERAL SERVICES ADMINISTRATION
FED. PROC. REG. (41 CFR) 1-16.401
SFD Rev. 3 ... 1972

CONSTRUCTION CONTRACT

(See instructions on reverse)

CONTRACT NO.: DACW07-7[illegible]-C-0022

DATE OF CONTRACT: 1978 March 01

NAME AND ADDRESS OF CONTRACTOR

R & D WATSON, INC.
Box 7010
Carmel, CA 93921

CHECK APPROPRIATE BOX

- [ ] Individual
- [ ] Partnership
- [ ] Joint Venture
- [X] Corporation, incorporated in the State of California

DEPARTMENT OR AGENCY

Department of the Army, U.S. Army Engineer District, San Francisco, Corps of Engineers, 211 Main Street, San Francisco, CA 94105

CONTRACT FOR (Work to be performed)

HALF MOON BAY BREAKWATER REPAIR,

PLACE

SAN MATEO COUNTY, CALIFORNIA

CONTRACT PRICE (Express in words and figures)

ONE MILLION TWO HUNDRED SEVENTY FIVE THOUSAND AND NO/100TH DOLLARS ($1,275,000.00), as itemized in Unit Price Schedule, Supplementary page 2a hereof

ADMINISTRATIVE DATA (Optional)

PAYMENT TO BE MADE BY: DISBURSING OFFICER
U.S. ARMY ENGINEER DISTRICT, SAN FRANCISCO
CORPS OF ENGINEERS
211 MAIN STREET
SAN FRANCISCO, CA 94105

The United States of America (hereinafter called the Government), represented by the Contracting Officer executing this contract, and the individual, partnership, joint venture, or corporation named above (hereinafter called the Contractor), mutually agree to perform this contract in strict accordance with the General Provisions (Standard Form 23-A), ~~[illegible]~~, and the following designated specifications, schedules, drawings, and conditions: Half Moon Bay Harbor, Breakwater Repair, San Mateo County, CA., dated 1978 January 16 and Amendment No. 0001 thereto, dated 1978 Feb 0[illegible], Serial DACW07-78-B-0003.

ACCOUNTING AND APPROPRIATION DATA: 96X3123 O & M General, Civil, CofE 004203 (CA1052600A[illegible]000CCC)

AUTHORITY: River and Harbor Act of 30 June 19[illegible], House Document No. [illegible], 80th Congress, Second Session.

WORK SHALL BE STARTED: Within [illegible] (3) calendar days after date of receipt of Notice to Proceed

WORK SHALL BE COMPLETED: In accordance with Paragraph SP-1 of the Contract Special Provisions

DIVISION 2 - SITE WORK

SECTION 2A

STONEWORK

1. WORK INCLUDED.

The work covered by this section consists of furnishing all plant, labor, materials, equipment, supplies and incidentals and performing all operations required to repair the west and east breakwaters as shown on the drawings.

2. APPLICABLE PUBLICATION.

The following American Society for Testing and Materials (ASTM) Specification of the issue listed below, but referred to thereafter by basic designation only, forms a part of this specification to the extent indicated by references thereto.

American Society for Testing and Materials (ASTM) Standards.

C127-73	Specific Gravity and Absorption of Coarse Aggregate

3. MOBILIZATION AND DEMOBILIZATION.

Mobilization shall consist of all work required in preparing the Contractor's plant and equipment for shipment; moving such plant, equipment, supplies and incidentals onto the jobsite and preparation for construction operations. The Contractor's plant and equipment proposed for use in the work shall be of sufficient size, capacity and efficiency to meet the job requirements and will be subject to approval by the Contracting Officer's Representative. Demobilization shall consist of all work required to remove the Contractor's plant, equipment, unused supplies and incidentals from the jobsite at the completion of the contract work, including cleaning up.

4. MATERIALS.

4.1 Quality of Stone. All stone shall be sound, durable, hard, free from laminations or cleavages, and of such character that it will not disintegrate from the action of air, sea water, or the conditions to be met in handling and placing. All stone shall be angular quarried material and stone shall have the greatest dimension not greater than 3 times the least dimension. Quarry operations shall include selective quarrying, screening or grizzlying, handling, and loading to produce rock conforming to gradation requirements. The Contractor will not be granted any extension of time or extra compensation due to any delay caused by sampling or testing of material under the requirements of these specification .

4.2 Source of Materials. Stone materials may be obtained from sources listed in subparagraph "Sources of Stone Materials", under paragraph "QUALITY CONTROL", or from any source proposed by the Contractor and approved by the Contracting Officer in accordance with subparagraph "Materials", under paragraph "QUALITY CONTROL". The Contractor shall make all arrangements, pay all royalties and secure all permits necessary for furnishing, transporting and placing stone from any source.

4.3 Bedding. Bedding material shall be crushed rock, reasonably well graded. Individual pieces shall be angular in shape and shall be free from vegetable matter and fines in excess of 5 percent.

4.4 Quality. Suitable tests and service records will be used to determine the acceptability of the stone materials. In the event suitable test reports and a service record, that are satisfactory to the Contracting Officer, are not available, as in the case of newly operated sources, the material shall be subjected to such tests as are necessary to determine its acceptability for use in the work. Tests to which the materials may be subjected include petrographic analysis, specific gravity, abrasion, absorption, wetting and drying, freezing and thawing, and such other tests as may be considered necessary to demonstrate to the satisfaction of the Contracting Officer that the materials are acceptable for use in the work. All tests will be made by or under the supervision of the Government and at its expense.

4.5 Stone Classes and Weights. The minimum, average and maximum stone weights for each class of stone shall be as listed below. The average weight of the total of the individual pieces of stone for each class shall not be less than the listed average weight.

Stone Type (Class)	Minimum Weight	Average Weight	Maximum Weight
A-1	6.0 Ton	8.0 Ton	10.0 Ton
A-2	9.0 Ton	13.0 Ton	17.0 Ton
A-3	12.0 Ton	16.0 Ton	20.0 Ton
B	2.0 Ton	4. 0 Ton	6.0 Ton
C	200.0 Lbs.	600.0 Lbs.	1000.0 lbs.
Bedding Material	No.4 Sieve	1 Inch	6 Inches

4.6 Stone Density. All stone shall have a density of not less than 150 pounds per cubic foot. Stone average weights shown hereinabove are based on stone density of 165 pounds per cubic foot. Stone densities greater or lesser than 165 pounds per cubic foot may result in new stone weights as determined by the Contracting Officer. The stone density shall be based on the saturated surface dry specific gravity of the stone determined in accordance with ASTM C127.

5. PLACEMENT.

5.1 Bedding material shall be placed on the existing surface within the limits and to the thickness shown on the drawings. The placement shall be such as to obtain a graded mass of uniform thickness after placement. The thickness of the layer may have a tolerance of plus 2 inches or minus zero.

5.2 Stone.

5.2.1 General. The final limits of stone, in place, shall be to the lines and grades indicated on the drawings, with reasonable variation. No stone shall be placed or moved after original placement unless a representative of the Contracting Officer is present.

The contractor shall exercise extreme care during placement operations so as to avoid disturbance of "well nested" existing or newly placed stones.

5.2.2 Method of Placement. The stones shall be carefully placed, keyed and set by placing or relocating with a crane, derrick or similar approved equipment. In general, the longitudinal axis of each outer stone shall be normal to the axis of the breakwater and shall slope downward toward the center of the breakwater.

5.3 Rejected Stone. Rejected stone is defined as stone not suitable as to quality or size as specified herein. Any rejected stone will not be paid for and shall be promptly removed from the jobsite at no expense to the Government. Any rejected stone placed in the permanent work shall be removed by the Contractor at his expense and such stone shall be replaced with stone as specified.

6. TRUCK DELIVERY.

All stone delivered by truck shall be weighed and the scale tickets certified by authorized weighers provided by the Contractor. All trucks used for delivering stone shall be plainly numbered.

7. SCALES.

Scales, including truck scales, track scales and other scales used for the work, shall be of the standard beam type. The scales shall be of sufficient capacities to satisfy all weighing requirements. Scales shall be tested, approved and sealed by an inspector of the State Inspection Bureau or applicable Government agency charged with scales inspection within the State of California. Scales shall be calibrated and resealed as often as necessary to insure continuous accuracy. The necessary number of standard weights for testing the scales shall be on hand at all times and, if an official inspection bureau of the State or other applicable agency is not available, the scales will be tested by the Contracting Officer. Where scales are found to be defective or their accuracy becomes questionable, stone deliveries shall immediately cease and shall not resume until approved scales are available. Approved scales of a type which can be attached to the crane lifting line may be used for weighing individual stones.

8. SCALE TICKETS AND RECORDS.

Copies of scale tickets or records of weights including displacement weight data, shall be submitted to the Contracting Officer during the progress of the work. The Contracting Officer will determine from the displacement weight data, the weight of stone shipped by barge and will certify displacement weight records. Each scale ticket or record shall include the gross, tare, dunnage and net weight of stone. The weight of dunnage for each load will be determined, recorded and certified by the Contracting Officer. Deliveries of barge stone transhipped after weighing shall be accompanied by certified and numbered scale tickets or records using an approved system to maintain delivery control. Copies of scale tickets and/or receipts shall accompany each load of stone for all methods of transportation and a copy shall be delivered to the Contracting Officer on delivery of the stone. Before the final payment is made, the Contractor shall file with the Contracting Officer certified scale tickets or certified records for all stone used in the construction covered by the contract.

9. WEIGHING STONE BY DISPLACEMENT.

Stone delivered on barges may have its weight calculated by the displacement method. In such calculation one cubic foot of displacement will be defined as 64 pounds of weight. All facilities for verifying displacement and for determining the relation between draft tonnage, between depth of water in the hold and bilges, and net stone tonnage shall be placed on barges by and at the expense of the Contractor. All barges shall be loaded to approximate level displacement. All barges shall be plainly numbered. Where the bottom of load compartments is below the draft of the loaded barge, stone in water will be defined as occupying [illegible] percent of any water in the compartment with a buoyancy of 64 pounds per cubic foot. Displacement weight for each barge shall be obtained prior to loading, subsequent to loading and upon arrival at the construction area.

10. CALIBRATION OF BARGES.

10.1 Barges are designated as including deck, hopper, bottomdump, and tilt barges, scows and other marine equipment used for transporting stone or for measuring stone quantities by displacement method. The Contractor shall furnish plans of each barge as constructed, including all equipment and modifications, or barges shall be drydocked for inspection as directed by the Contracting Officer. No barge shall be put in use conveying rock to the breakwaters without prior approval of the Contracting Officer.

10.2 Prior to use on the work, the Contractor shall provide all facilities and equipment for the gaging of or calibration of each barge by the Contracting Officer. The bilges and hold (as applicable) of each barge shall be filled progressively with sea water as directed to provide tables of displacement versus water levels in the bilges and hold. Before calibration of a barge, each bilge shall be fitted with gage wells as may be required and all bilges shall be satisfactorily limbered.

10.3 A barge shall be recalibrated when, in the opinion of the Contracting Officer, changes to the barge and/or equipment void the previous calibration.

11. MAINTENANCE OF MARINE EQUIPMENT.

Barges shall be maintained in good operating condition including soundness of hull and operation of mechanical equipment. Barges shall be maintained, loaded and operated to preclude loss of transported materials. Placement barges and placing equipment shall have suitable conveniences for safe and adequate inspection of the work.

12. QUALITY CONTROL.

12.1 Sources of Stone Materials. The Contractor shall designate in writing within 5 days after award of the contract the sources or sources from which he will furnish the stone to be incorporated into the work. Listing of sources in "Sources of Material" shall not constitute representation by the Government that the source or sources will produce the quantity or sizes required.

12.2 Sources of Material. Stone meeting the quality requirements specified has been previously obtained from the sources listed below.

12.2.1 Davenport Quarry (Lone Star Industries). Located about 12 miles north of Santa Cruz. Formerly source of limestone for cement plant. Rock is hard, gray sandstone which is stockpiled at the quarry.

12.2.2 Basalt Rock Company, McNear's Quarry. Located on San Francisco Bay at San Rafael. Rock is fresh, blue-gray sandstone.

12.2.3 Fisher Quarry (Umpqua River Navigation Company) Camas, Washington. The Fisher Quarry is located on the Columbia River. Rock is a fresh, light gray, dense, columnar basalt.

12.2.4. Quarry Products, Inc. Located at Pacifica, San Mateo County, California. Suitable for bedding material and quarry run surfacing material.

12.3 Materials. Material may be obtained from the sources indicated above. If the Contractor proposes to furnish material from sources not listed and test reports or service records covering the materials from such sources that are satisfactory to the Contracting Officer are not available, the material will be tested by the Government for quality to determine its acceptability for use in the work. When the Contractor desires to use materials from a source not listed, or if the Government elects to retest a source that is listed, suitable samples for quality evaluation shall be taken by the Contractor under the supervision of the Contracting Officer. Samples shall be delivered by the Contractor to the Corps of Engineers, South Pacific Division Laboratory, Sausalito, California. Sampling and shipping of samples shall be at the Contractor's expense. Listing of a source or sources shall not be construed as approval of all materials obtained from that source or sources. The right is reserved to reject material produced from localized areas, zones, or strata when such materials are unsuitable for intended use.

12.4 Samples and Testing. Samples of material from one unlisted source, proposed by the Contractor, will be taken and tested by and at the expense of the Government. Samples of materials from additional sources, proposed by the Contractor, will be taken and tested by the Government and the costs of such testing will be deducted from amounts due or to become due the Contractor. All work required to produce samples of material, representative of the proposed sources, shall be done by and at the expense of the Contractor, and the material, ready for sampling by the Government, shall be made available at the proposed quarry site, at least 60 days in advance of the time when the placing of stone is expected to begin.

12.5 Inspection. The Contractor shall establish and maintain quality control for all quarrying, loading and placing operations to assure compliance with contract requirements and maintain records of his quality control for all operations, including, but not limited to the following:

Quarrying stone
Quality of furnished stone
Placement methods
Size and weight of stone, in place

The records of such quality control and any corrective action taken to maintain contract compliance will be noted in the Contractor's Quality Control Report. None of the above requirements shall be construed as abrogating the rights of the Government to inspect the work and to direct changes when required to conform to the drawings and specifications.

13. MEASUREMENT AND PAYMENT.

13.1 Measurement. The quantity of stone to be paid for by weight as indicated in the bidding schedule shall be determined either by scale weights or weights calculated by the displacement method. The unit of weight measurement shall be the ton equaling 2,000 pounds. Measurement will be to the nearest one tenth (1/10) of a ton of total quantity of material acceptably placed in the completed work. Detached stone not placed as a part of the specified work will not be included in the measured quantities for payment. Payment having once been made, quantity measurements for such payments will not be reopened except on evidence of collusion, fraud or obvious error.

13.2 Density of Stone. The estimated tonnage of stone under the pay item for stone is based on stone with a specific gravity of 2.65. Stone from any source where the specific gravity varies more than 5% from that used in estimating the tonnage will result in a new estimated tonnage being derived, based on a specific gravity of the approved stone to be used. General Provision "VARIATIONS IN ESTIMATED QUANTITIES" will then be applicable to the tonnage so derived.

13.3 Mobilization and Demobilization. Payment for mobilization and demobilization will be made at the contract lump sum price for "Mobilization and Demobilization" in accordance with Special Provision "MOBILIZATION AND DEMOBILIZATION", which price and payment shall constitute full compensation for moving all labor, plant, materials and equipment necessary to perform the stonework onto the jobsite, ready for work and removing same from the jobsite upon completion of the contract work.

13.[illegible] Stonework. Payment for furnishing and placing new stone will be made at the applicable contract unit price for "A-1 Stone (8 Ton Avg.), In Place", "A-2 Stone (13 Ton Avg.), In Place", A-3 Stone (16 Ton Avg.), In Place," "B Stone (4 Ton Avg.), In Place", "C Stone (600 Lb. Avg.), In Place" and "Bedding Material (No. [illegible] Sieve to 6" Max.), In Place", which price and payment shall constitute full compensation for furnishing all plant, labor, materials and equipment and for performing all operations necessary to complete the stonework in accordance with the drawings and specifications.

* * *

SAFETY IS A TEAM EFFORT

APPENDIX D

APPENDIX D

BERM SELECTION IN MINING OPERATIONS

This part presents a copy of the considerations used in the design of quarry operations. It is copied from the Operating Handbook Of Surface mining.

Select berm width to contain local failures

Dennis C. Martin and **Douglas R. Piteau**, D. R. Piteau Associates Ltd.

STABILITY ANALYSES for open pit slope design must consider the possibility of the failure of individual benches as well as the failure of the overall slope. In many cases, the probability of overall slope failure along major faults or weak zones may prove to be small, while the design of individual benches against excessive failure may be the controlling factor for design of the overall slope. Small failures can cause major disruptions to pit operations and can limit accessibility. A graphical method for design of individual benches to control small failures is described here.

Considerations for basic slope design

In rock slopes, instability occurs as a result of failure along structural discontinuities, such as bedding planes, joints, geological contacts, and faults. Instability seldom occurs in homogeneous material unless the rock is weathered or soft.

The most important single factor in stability analyses and design of rock slopes is the determination and evaluation of the orientation, geometry, and spatial distribution of discontinuities in the slopes. This process should be followed by evaluation of possible alternative angles of the proposed pit slopes relative to the orientation of the discontinuities.

Slope control may be accomplished by designing the slope so that no failure can occur or by excavating the pit under controlled conditions, with the slope designed for adequate access, while minor failures are caught on berms and removed as needed. The first solution is usually too conservative to be economical. The second solution requires a thorough consideration of slope geometry.

Parameters that govern the geometry of a slope are bench height H, berm width l, and bench face angle β. (See Fig. 1.) Normally, these parameters are determined by the strength and nature of the material of the slope, the size and type of equipment to be used, and mining regulations. Bench height should provide a safe working slope. For a given slope, higher benches permit wider berms.

In general, berms should be designed wide enough to entrap falling debris and provide access for cleanup.

Inclined bench faces reduce the likelihood of high stresses near bench crests and minimize tension cracks and overhangs. Problems of rockfall are thus reduced, and the safety of the slope is increased.

Slope and failure geometry are related

The volume of material in a bench failure is inversely proportional to either the dip of a plane failure or the plunge of a wedge failure. The volume varies with bench height. The shallower the dip or plunge of a failure, the greater the volume of material involved. Calculation of

Benches at this mine in British Columbia were designed to contain the wedge failures predicted by stability analysis.

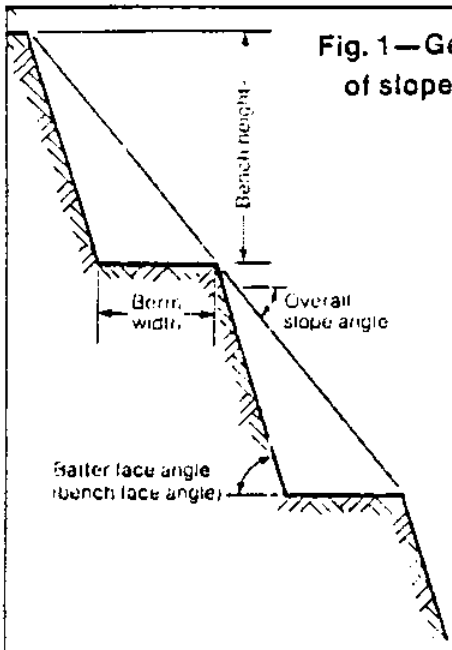


Fig. 1—Geometry of slope design

the cross-sectional area of a potential bench failure provides an estimate of the volume of failed material per unit width of berm.

For plane failures, the slumped material on the berm is assumed to form a uniform debris slope with a triangular cross section. Slumped material from a wedge failure is assumed to form an approximately conical debris slope. This slope extends outward across the berm along the projection of the line of intersection. It is assumed that the maximum extent of slumped material from a wedge failure will occur along the projection of the line of intersection and will have a triangular cross section — similar to that of a plane failure — of unit width along the line of intersection. Therefore, calculation of the cross-sectional area of a wedge failure along the line of intersection provides an estimate of the maximum volume of material, per unit width, that could slump onto the berm.

The vertical cross-sectional area A of a failure taken parallel to the direction of dip of the failure plane — or parallel to the line of intersection of a wedge failure — is calculated in Fig. 2 using the formula:

$$A = \frac{H}{2}\left(\frac{1}{\tan \beta_w} - \frac{1}{\tan \beta}\right) \qquad (1)$$

Fig. 2—Elements of bench design

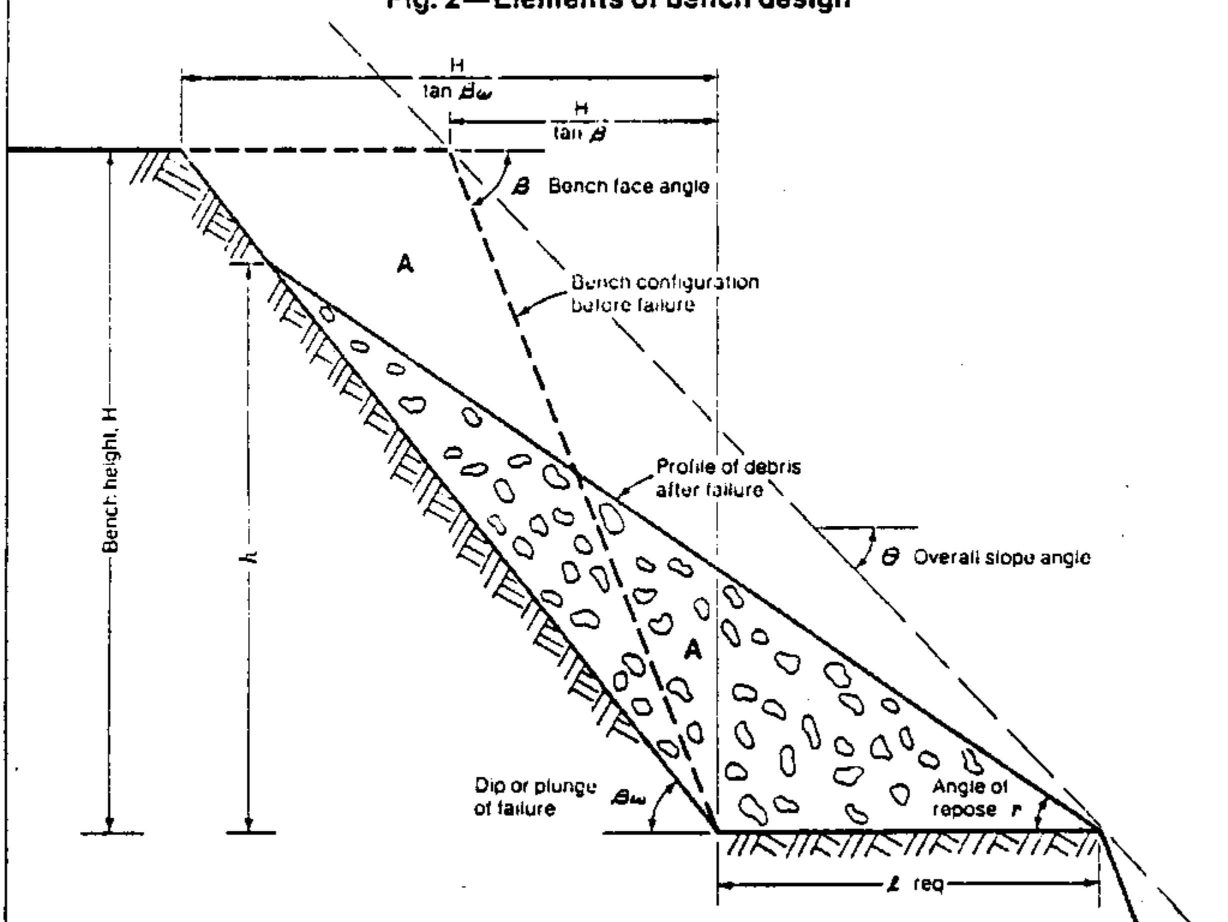


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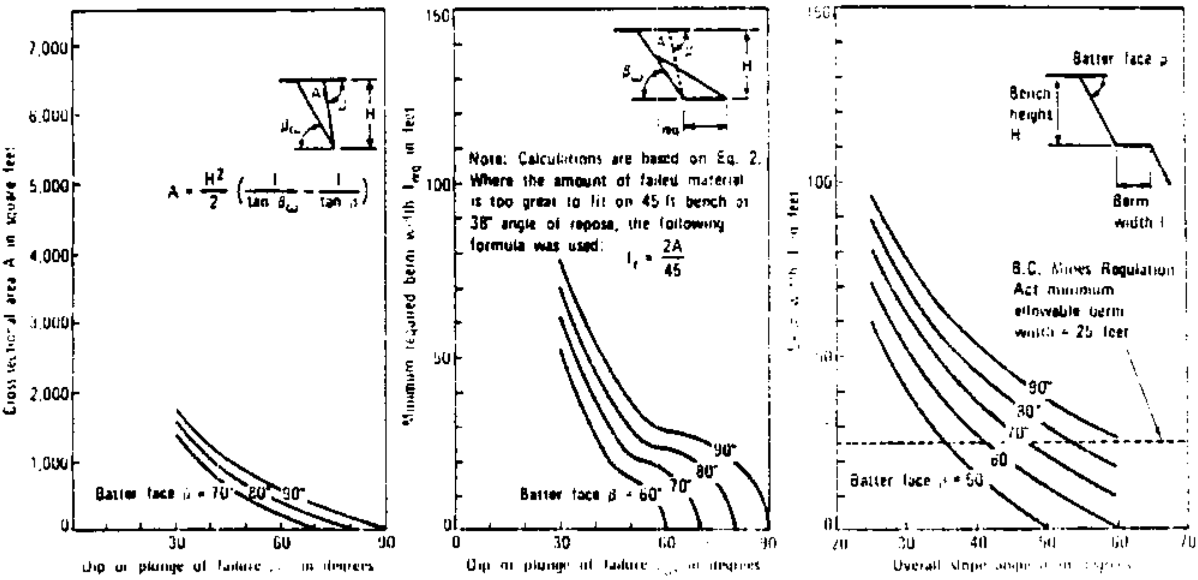
Fig. 3—Interrelationship of slope geometry and failure geometry, 45-ft benches

(38° natural angle of repose of debris)



(A) Variation of cross-sectional area with dip or plunge of a failure involving a single 45-foot-high bench

(B) Variation of minimum berm width required to catch failed material with dip or plunge of the failure

(C) Geometric relationship of overall slope angle, berm width, and batter face angle

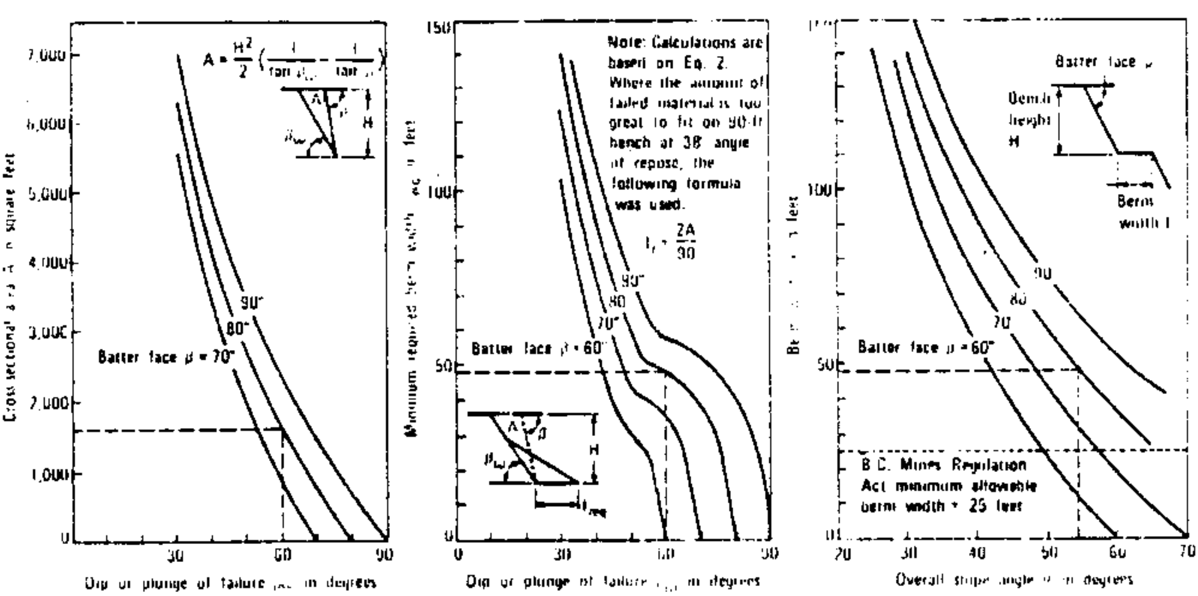
where H = bench height, β = bench face angle, and β_ω = the dip or plunge of the failure.

Cross-sectional area of a potential bench failure is used to calculate the minimum berm width l_{req} required to catch all failed material (Fig. 2). It is assumed that the failed material comes to rest at its natural angle of repose r, usually 35° to 40°. The formula used for calculation of the minimum berm width, from trigonometric relationships, is

$$l_{req} = \sqrt{\frac{2A}{\sin r \cos r + \frac{(\sin r)^2}{\tan(\beta - r)}}} \qquad (2)$$

Fig. 4—Interrelationship of slope geometry and failure geometry, 90-ft benches

(38° natural angle of repose of debris)

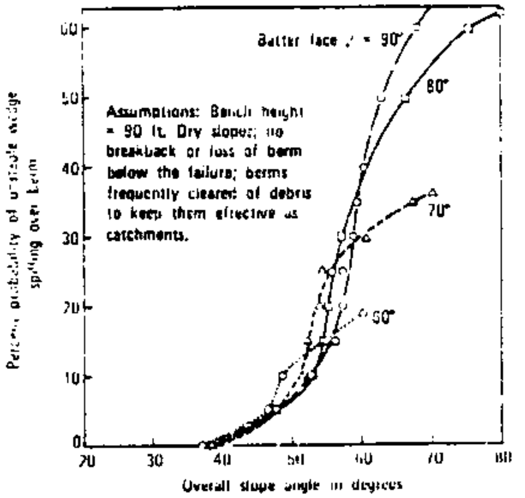


(A) Variation of cross-sectional area with dip or plunge of a failure involving a single 90-foot-high bench

(B) Variation of minimum berm width required to catch failed material with dip or plunge of the failure

(C) Geometric relationship of overall slope angle, berm width, and batter face angle

Fig. 5—Probability of wedge spillover for various slope angles



In cases where the amount of failed material is geometrically too large to be completely caught on the berm (for a particular bench height), it is assumed that the slump angle is less than r. The required berm width is then calculated on the assumption that a simple triangle of slumped material extends from the crest of the bench. In this case, the minimum berm width is calculated as:

$$l_{req} = \frac{2A}{H} \qquad (3)$$

The bulking (swell) factor—an increase in the volume of material during failure—is not considered. Bulking is a complex factor related to rock type, fracture intensity, and the mechanism of rock failure. Bulking can increase volume by as much as 50%. For conditions where Eq. 2 applies (e.g., steeply dipping failures), the minimum berm width is increased only by the square root of the bulking factor. On the other hand, where Eq. 3 applies (e.g., for shallow failures), the relationship is directly proportional, and the required minimum berm width can be 1.5 times the minimum berm width determined by that equation. Depending on the type and size of failure, bulking may be an important factor to be incorporated into the calculations. The factor can be determined experimentally by comparing unit weights of in-situ and of failed rock material.

Minimum berm width is used to calculate the overall slope angle for the particular bench height and bench face angle that are assumed in the analysis.

Acknowledgement: The technique described here was developed in conjunction with pit slope stability analyses and design work at the mines of Cyprus Anvil Mining Corp. and Cassiar Asbestos Corp. The authors wish to thank personnel of the two mines for their assistance.

Graphs of bench and failure geometry

For a particular bench height, the interrelationship of cross-sectional area A, minimum berm width l_{req}, and overall slope angle θ can be presented on three graphs. The graphs show the various bench face angles and dips, or plunges of the failure θ_w. The relationships for benches 45 ft high and 90 ft high are graphed in Figs. 3 and 4, respectively.

Design graphs for a specific bench height can be used to determine the size of a potential failure, including the berm width and overall slope angle necessary to contain the bench failure. Fig. 4 illustrates a typical example with 90-ft-high benches and 80° bench face angles. In this case, the geologic structure indicated that plane failures could develop, dipping at 60°.

Fig. 4 shows that the minimum berm width required to catch such failures is 48 ft, and the resulting overall allowable slope angle is about 54°. The amount of failure that would have to be cleaned up is approximately 1,600 cu ft per ft width of the failure (Fig. 4a). If the number and size of failures per unit length of bench can be estimated, the total amount of cleanup of failed material can be calculated.

The accompanying graphs assume that minimal backbreak of the bench crests will develop. Proper control of blasting and inclined bench faces will reduce backbreak. Where access is required along berms, berm width should be increased to permit equipment passage past failures. Where discontinuities are large and involve more than one bench, the overall slope angle may have to be flattened.

Graphical analysis applied to B.C. mine

The application of the analytical technique described here proved extremely useful at a mine in northern British Columbia. At this mine, analysis was made of the upper 700-ft-high section of an 1,100-ft-high hanging-wall slope of argillitic rock. Numerous potential wedge failures, formed by the combination of two joint sets, controlled slope geometry and the stability of the benches. (See photo.)

Graphical technique was used with probability theory to determine the optimum bench geometry required to contain a reasonable number of wedge failures upon the berms. The plunge of the wedges varied mathematically as a normal distribution about the mean plunge value. Probability techniques were applied to assess the likelihood of unstable wedges occurring whose plunge would be greater than that determined by probability criteria. The probability of an unstable wedge spilling over a berm, for a particular overall slope angle, was determined as shown in Fig. 5. Mine management was then in a position to evaluate alternative slope designs.

A more complete discussion of the technique described here is presented in "Slope Stability Analysis and Design Based on Probability Techniques at Cassiar Mine," a 1976 CIM Bulletin by D. R. Piteau and D. C. Martin. ☐

APPENDIX E

APPENDIX E

DIFFERENT PATTERNS OF BLASTING HOLES

This section presents a copy of different layouts for the drilled holes in the blasting operation. It is copied from the "Blasters' Handbook" (duPont,1977).

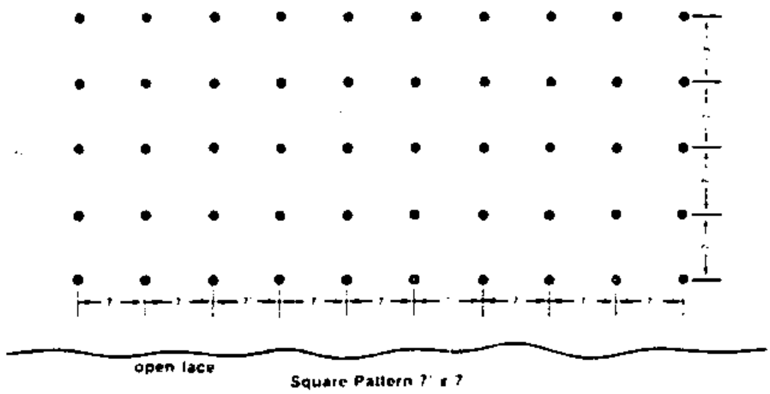


Figure 15-G. The square pattern features equal burden and spacing with the holes in each row aligned directly behind the corresponding holes in the pattern's front row.

SQUARE

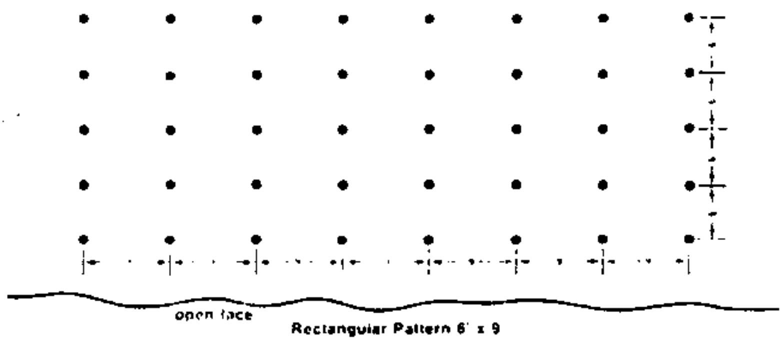


Figure 15-H. The rectangular pattern has the burden less than the spacing, and the holes in each row are again aligned directly behind corresponding holes in front row.

RECTANGULAR

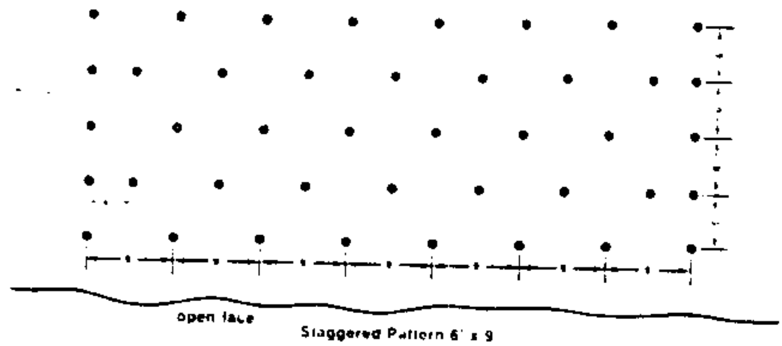


Figure 15-J. The staggered pattern may have the burden and spacing equal, but the holes in the alternate rows are in the middle of the spacings of the front row holes.

STAGGERED

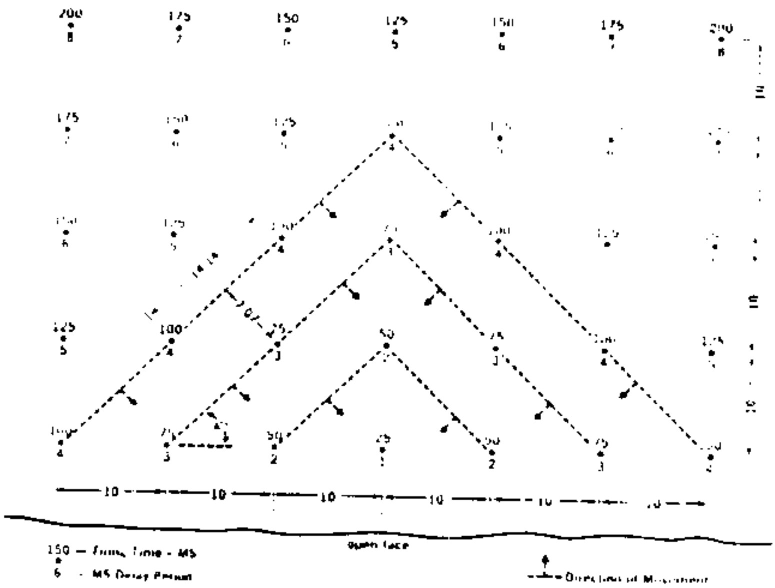


Figure 15-K. The V pattern is applicable to most types of formations. It is readily adaptable to square or rectangular patterns or can be used with the staggered pattern.

PATTERN

V - MS DELAY PATTERN

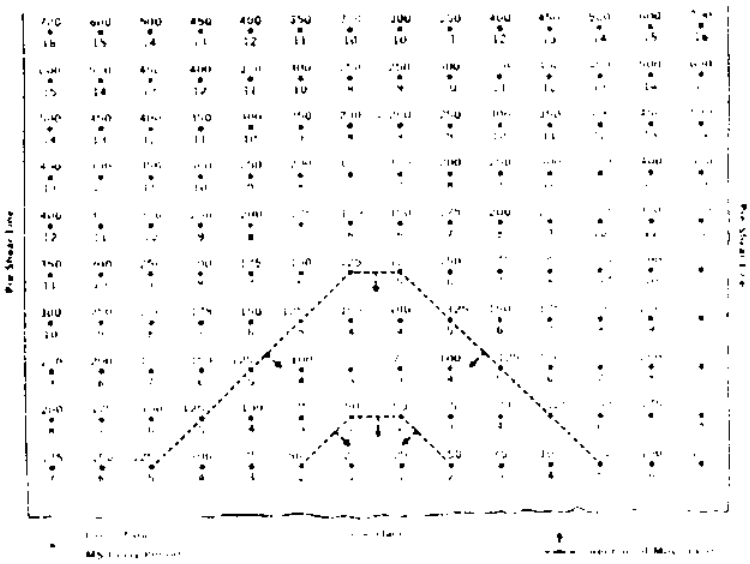


Figure 15-L (same as 19-I). Depending on formation and number of holes-per-row, it may be desirable to open blast with two holes on 25 MS delay (and move muck forward).

ECHELON MS DELAY PATTERN

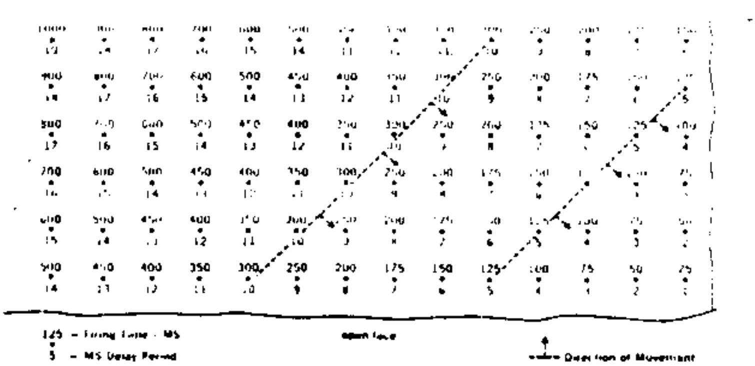


Figure 15-M. When the blast area is open on two adjacent sides in an external corner (lower right above), the blast should be designed to utilize this additional relief.

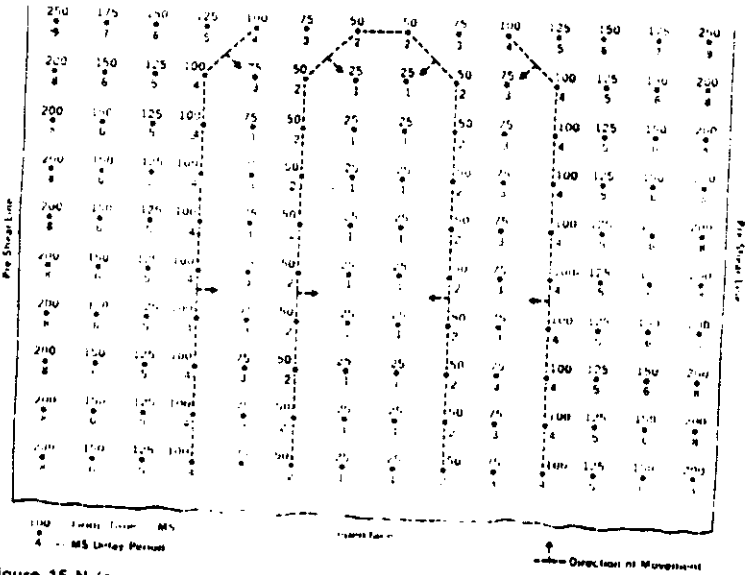


Figure 15-N (same as 19-H). Frequently quarries must be opened on side hill terrain where there is not enough open area in front of the blast for the forward movement associated with the V or eschelon MS delay patterns. In these cases a channel MS delay pattern like this can be used to confine movement of broken rock to blast area.

FLAT-FACE MS DELAY

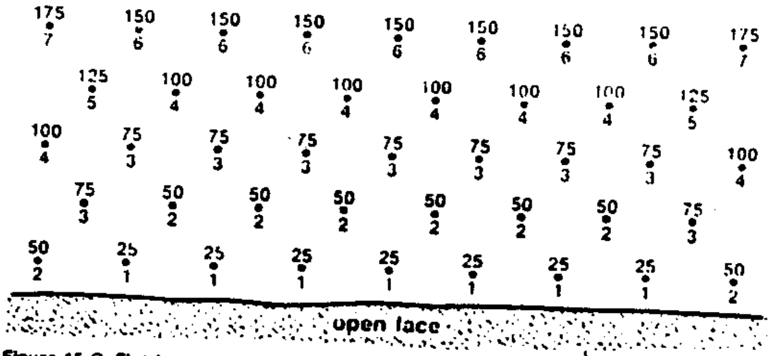


Figure 15-O. Flat-face pattern will move broken material farther from face and usually result in larger size rock. Flat-face is almost always used with the staggered pattern.

ALTERNATING HOLE MS DELAY

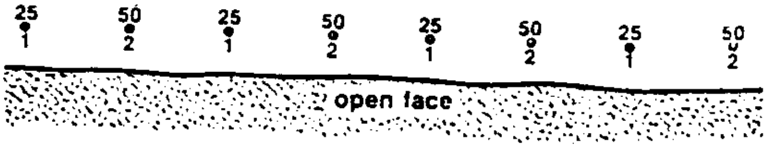


Figure 15-P. Alternating hole MS delay pattern has been used with limited success in thinly laminated formations on wide spacings. It s not recommended for most formations.

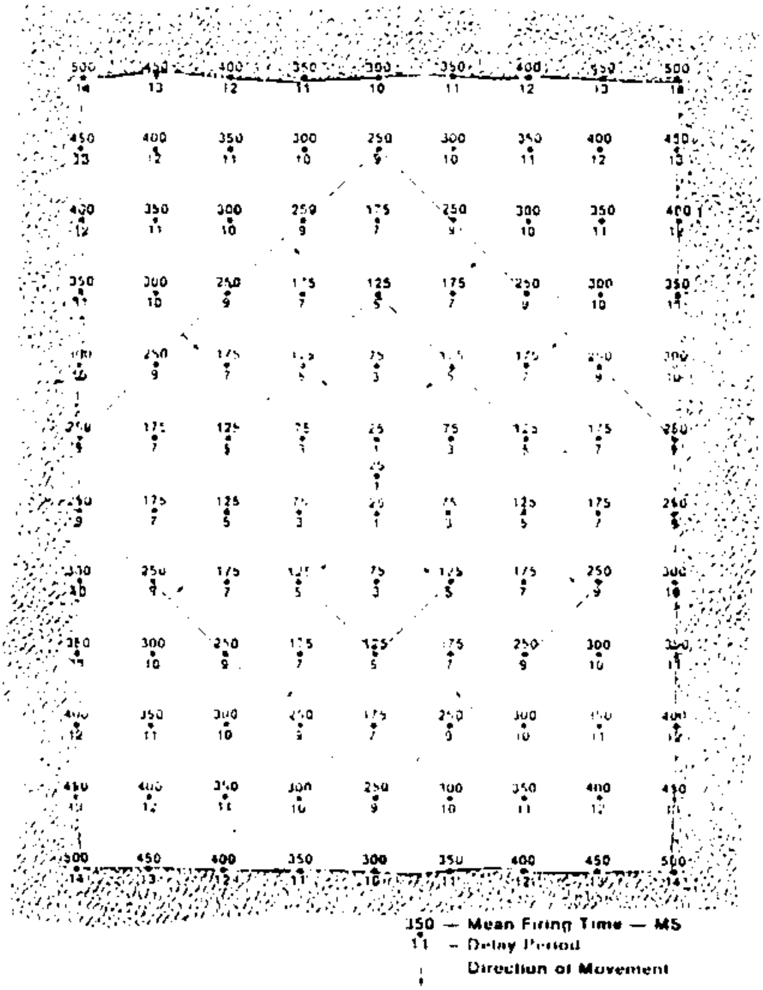


Figure 15-Q. Sinking blasts like this (and in Figure 15-R on next page) vary from most blasts because there is no open face for relief and movement must be vertical. Since the blast is "in the tight", the vibration and fly rock damage potential is high.

SINKING BLAST

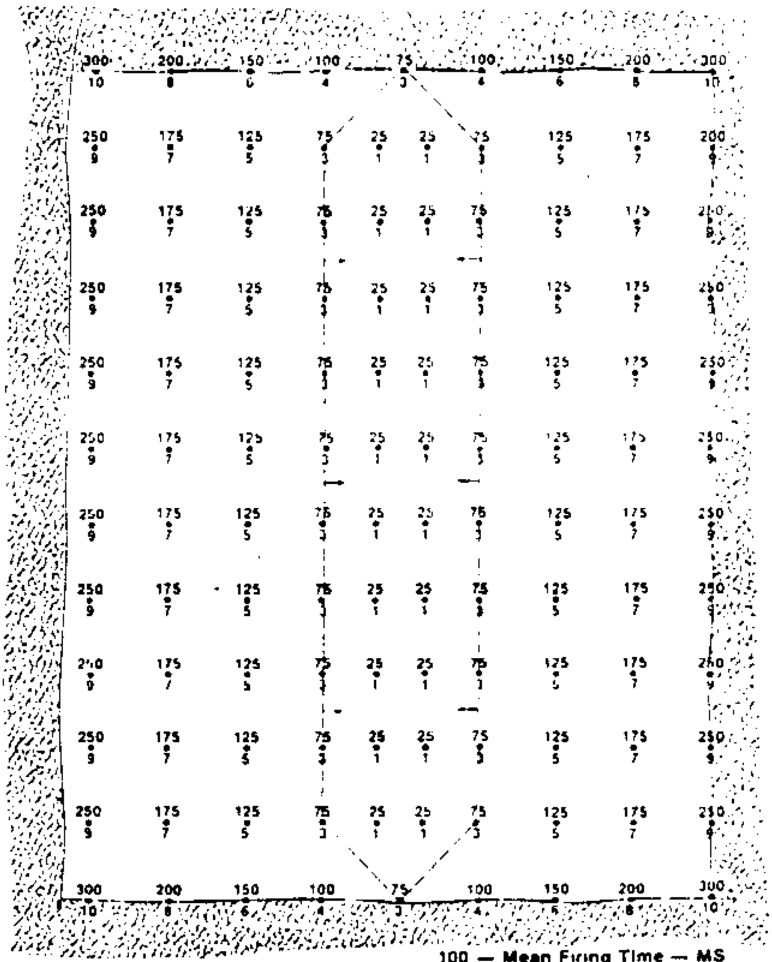


Figure 15-R In most formations where sinking blasts like this (and in Figure 15-Q on preceding page) are used, it is necessary to decrease the burden and spacings of initial holes in the delay pattern to open an area of relief for the remaining holes.

SINKING BLAST

DETONATING CORD PATTERN

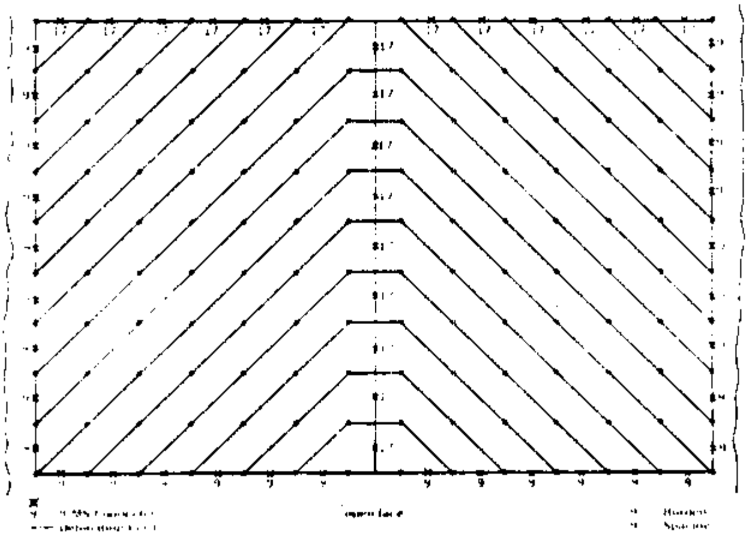


Figure 15-S (same as 19-L). With earth movement there is always a chance for cutting detonating cord. Systems for patterns using detonating cord (and MS delay connectors) should be designed like the one above to provide considerable redundance as safeguard.

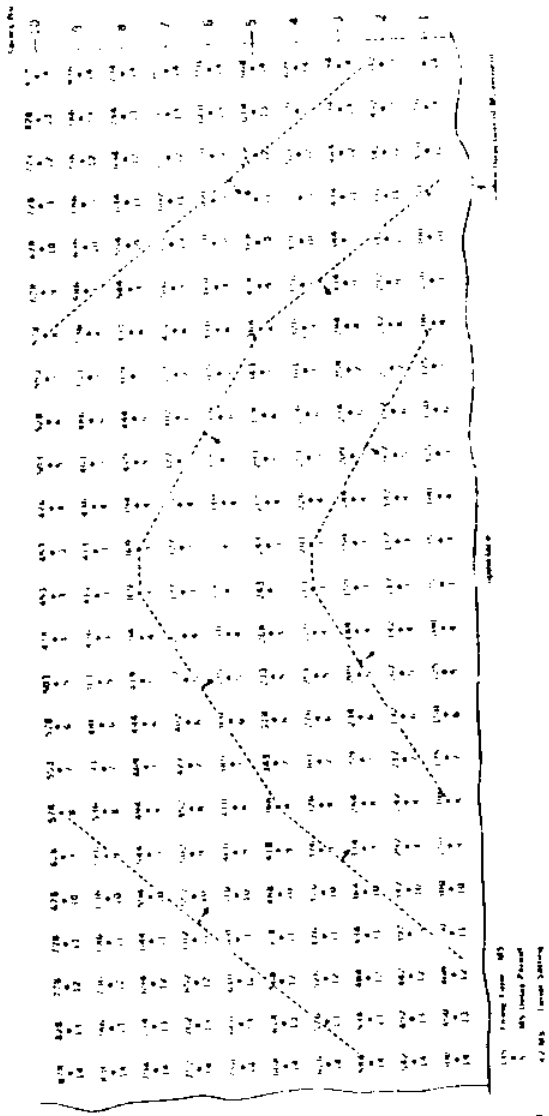
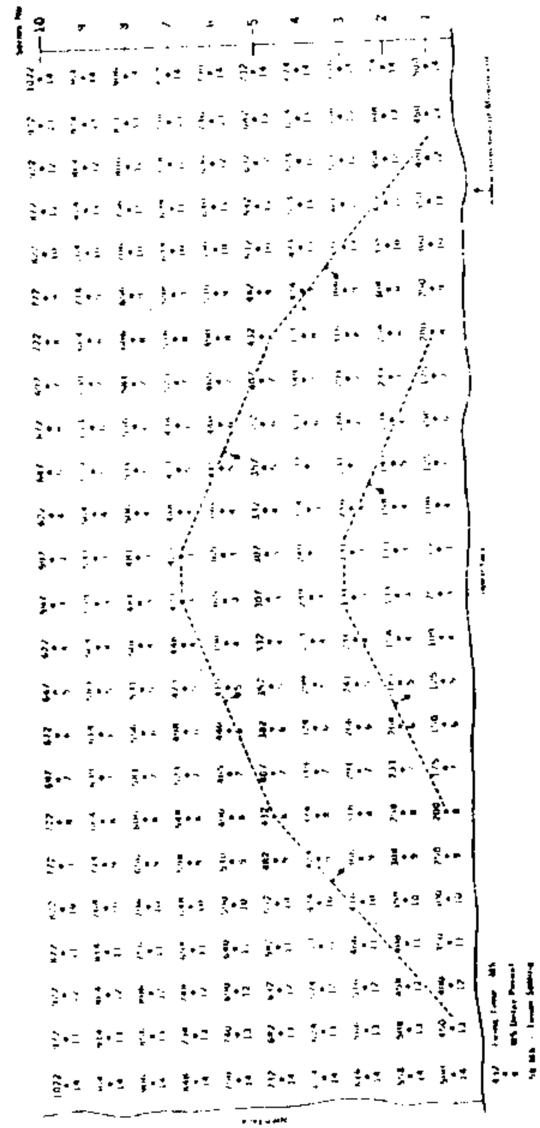


Figure 15-T. This V pattern has been used successfully on Sequential Timer settings of 33, 42 and 58 milliseconds on blasts up to 240 holes. The setting choice usually is determined by a combination of factors, formation, burden, spacing and hole depth.



F[igu]re 15-U. A change in the timer setting—be it 33, 42 or 58 milliseconds (see c[apt]ion for Figure 15-T opposite)—physically affects the angle of movement. It c[an] also materially affect the level of vibration that might result from the blast.

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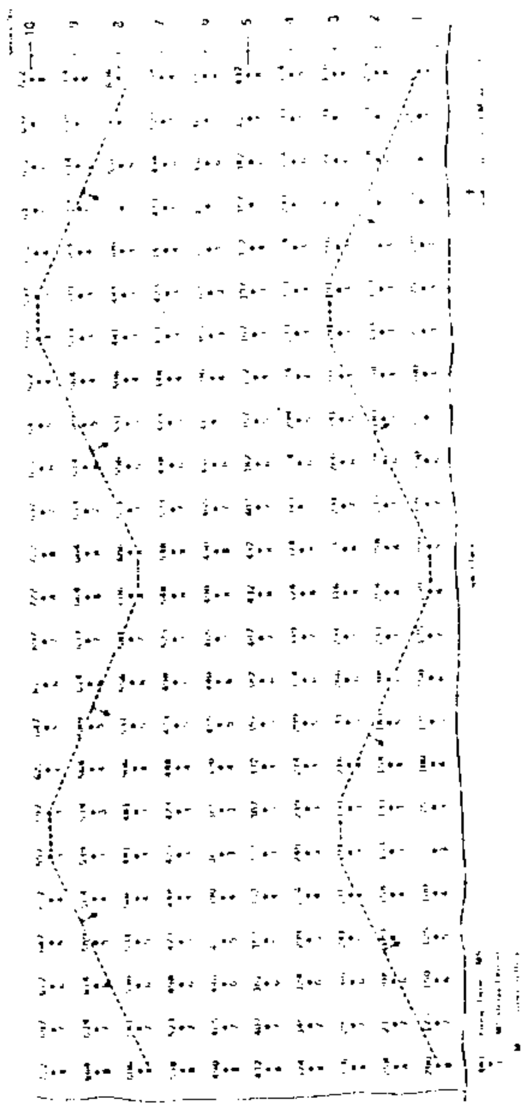

Figure 15 V. Where it is considered necessary to hold a constant angle of movement, a double V pattern like one above has proven very successful in a number of cases.

Where it is considered necessary to hold a constant angle of movement, a double V pattern, Figure 15-V, has been very successful.

The echelon pattern, Figure 15-W, has proven very successful in most formations and combined with the V-pattern can be used for the entire blasting operation in many quarries.

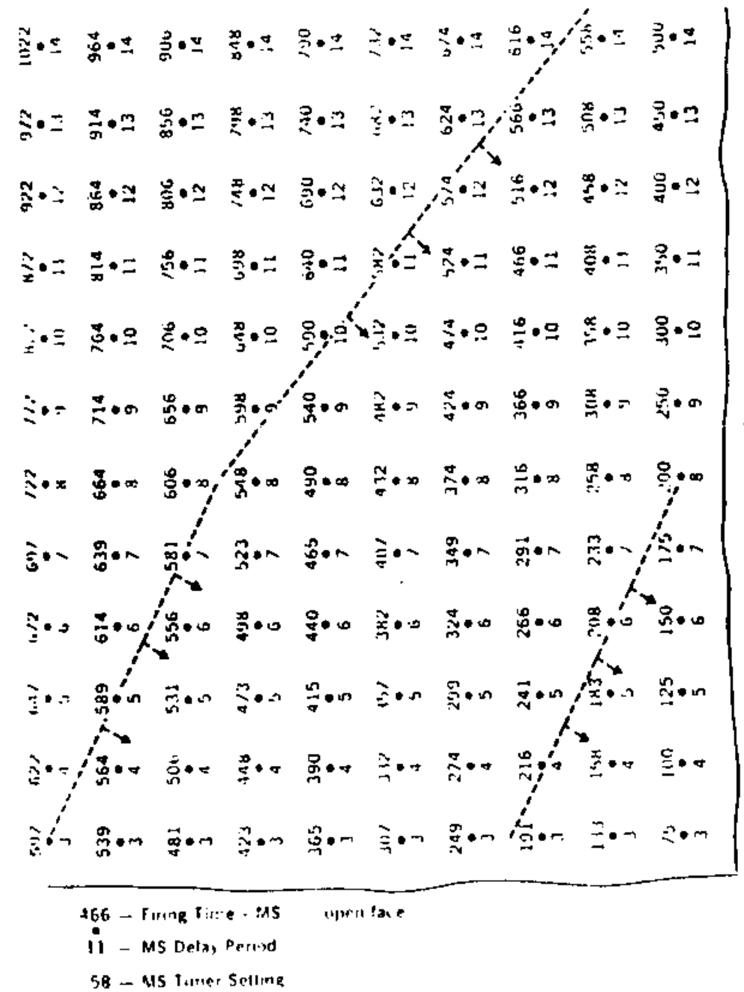

Figure 15-W. The echelon pattern has proven very successful in most formations. Combined with the V pattern it can be used for entire quarry blasting operation

The sinking blast pattern (Figure 15-X) has proven very successful in reducing vibrations to an acceptable level. In addition, it has produced outstanding results in pulling full depth and improving fragmentation with a minimum of fly rock.

Multideck loading in large-diameter holes (Figure 15-Y) has been very successful for quarries operating in densely populated areas.

Use of the sequential timer in conjunction with custom-designed blast patterns has been extremely successful in most quarry opera-

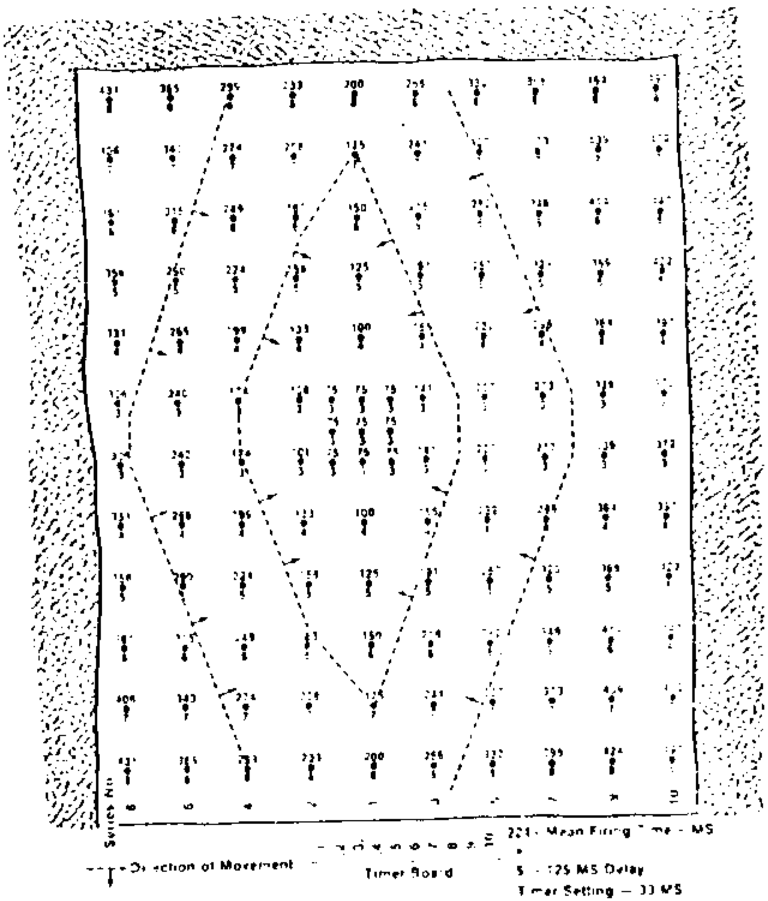

Figure 15-X. The sinking blast has reduced vibrations to an acceptable level and has produced outstanding results in pulling full depth and improving fragmentation.

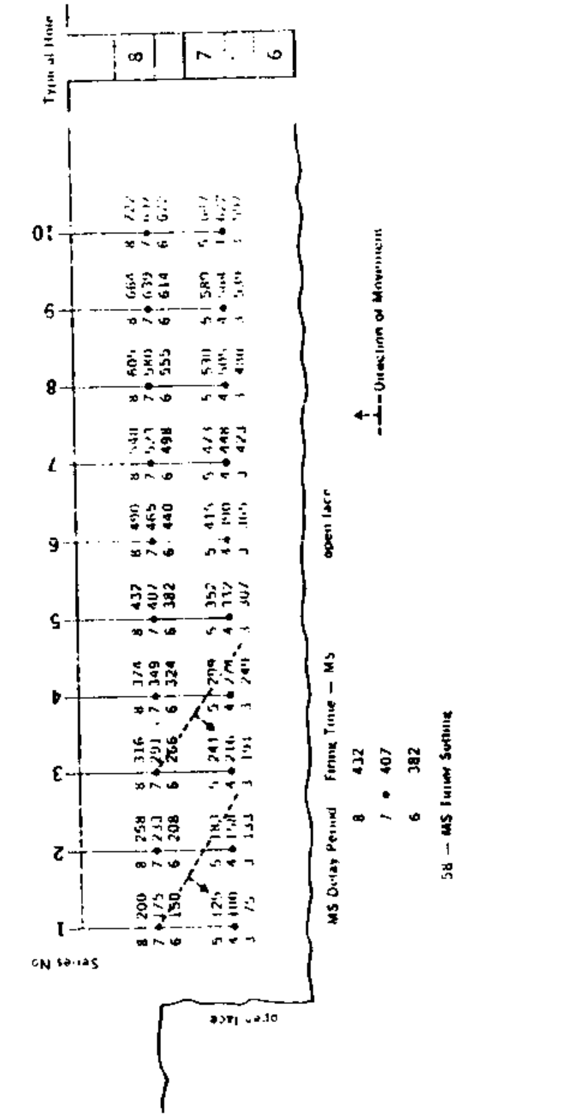

Figure 15-Y. Multideck loading in large diameter holes—detailed in the pattern above—has been very successful for quarries operating in densely populated areas.

TYPES OF HOLES

SNAKEHOLES

Snakeholes are horizontal holes drilled under a relatively low face to blast the rock.

SECONDARY BLASTING

Secondary blasting is an extremely expensive operation. Measures should be taken in the primary blast to hold the amount of secondary blasting to a minimum.

SPRUNG HOLES

For many years it was a common practice to drill small-diameter holes and spring or chamber the bottom of the hole to allow room for additional explosives as a bottom load. This practice is extremely hazardous and has been largely abandoned with the technical advances in drilling equipment.

COYOTE TUNNELS

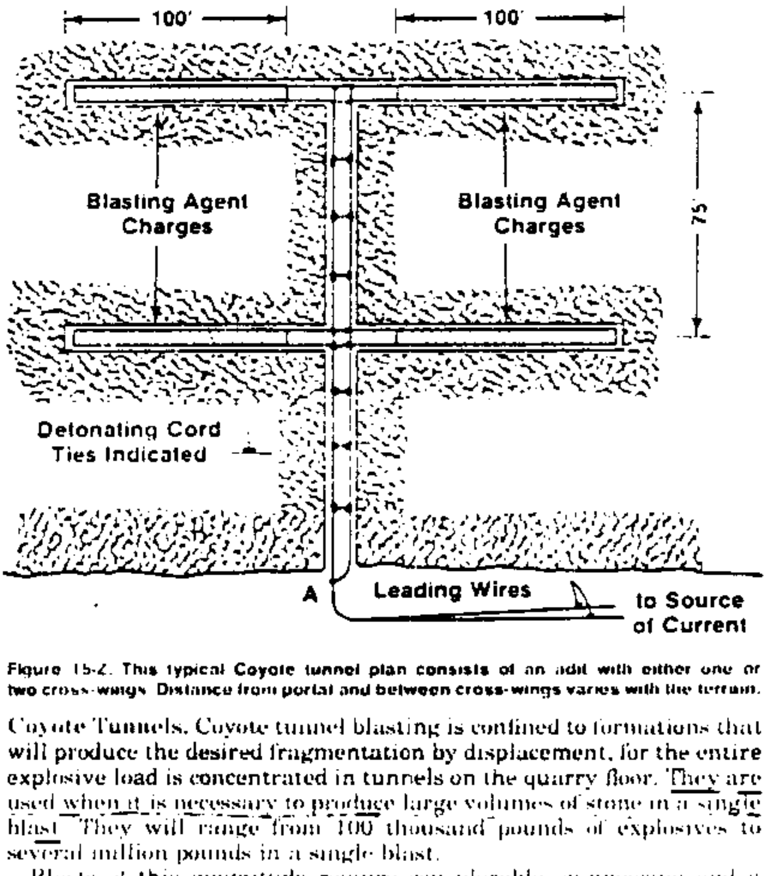

Figure 15-2. This typical Coyote tunnel plan consists of an adit with either one or two cross-wings. Distance from portal and between cross-wings varies with the terrain.

Coyote Tunnels. Coyote tunnel blasting is confined to formations that will produce the desired fragmentation by displacement, for the entire explosive load is concentrated in tunnels on the quarry floor. They are used when it is necessary to produce large volumes of stone in a single blast. They will range from 100 thousand pounds of explosives to several million pounds in a single blast.

Blasts of this magnitude require considerable engineering and a vast amount of experience. A typical coyote tunnel plan (Figure 15-2) consists of an adit with one or two cross-wings. The distance from the portal and the distance between wings will vary with the contour of the terrain. The height over the tunnels should be approximately 1½ times the distance between tunnels.

The volume of rock in front and over each tunnel must be accurately determined as should the area to be sheared in front of each tunnel. These two factors should be considered in relation to the type of formation in determining the amount of explosives to be loaded.

Your Du Pont representative should be consulted on any proposed coyote blast.

V-MS Delay Pattern. The V-pattern (Figure 15-8) is applicable to most types of formations. It can be readily adapted to the square or rectangular pattern. It may be used with a staggered pattern, but is not as practical for ease of loading under field conditions.

When a V-MS delay pattern is used in conjunction with a square drill pattern, the angle of movement is 45 degrees to the open face. Therefore, a 10 by 10-foot square pattern becomes a rectangular pattern with a 7.07-foot burden and a 14.14-foot spacing (or the burden is only one-half the spacing).

If a rectangular drill pattern is used, the angle of movement will vary in relation to the relative burden and spacing dimensions.

To determine the angle of movement in relation to the open face for a rectangular pattern where:

b = Burden

s = Spacing

A = Angle of movement

$$\tan A = \frac{b}{s}$$

To determine the effective burden and spacing in relation to the direction of movement where:

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ES = Effective Spacing
EB = Effective Burden

$$ES = \frac{b}{\sin A}$$

$$EB = s \times \sin A$$

Example:

A rectangular pattern with an eight-foot burden and a 13-foot spacing becomes:

$$\tan A = \frac{8}{13}$$

$$\tan A = 0.615$$

$$A = 31.6$$

$$ES = \frac{8}{\sin 31.6}$$

$$ES = \frac{8}{0.524}$$

$$ES = 15.27 \text{ ft.}$$

$$EB = 13 \times \sin 31.6$$

$$EB = 6.8 \text{ ft.}$$

Obviously, good judgment must be exercised when using a rectangular drill pattern in conjunction with a V-MS delay pattern to avoid getting the effective burden and spacing too far out of balance. In most formations the angle of movement should not be less than 15 degrees to the open face.

The V-MS delay pattern is the most frequently used pattern for 3½ to five-inch diameter holes of depths up to 60 feet. The forward movement, as shown in Figure 15-F, is controlled within reasonable limits, and the broken muck is deposited in a wind-row 90 degrees to the open face. Depending on the formation and the number of holes per row, it

Drilling snakeholes in a quarry — horizontal holes under a low face to blast rock.

APPENDIX F

APPENDIX F

MANITOWOC RINGER CRANE

This section presents a copy of the Manitowoc Crane copied from the company brochure. It includes the capacity and ranges of this equipment.

MANITOWOC ENGINEERING CO.

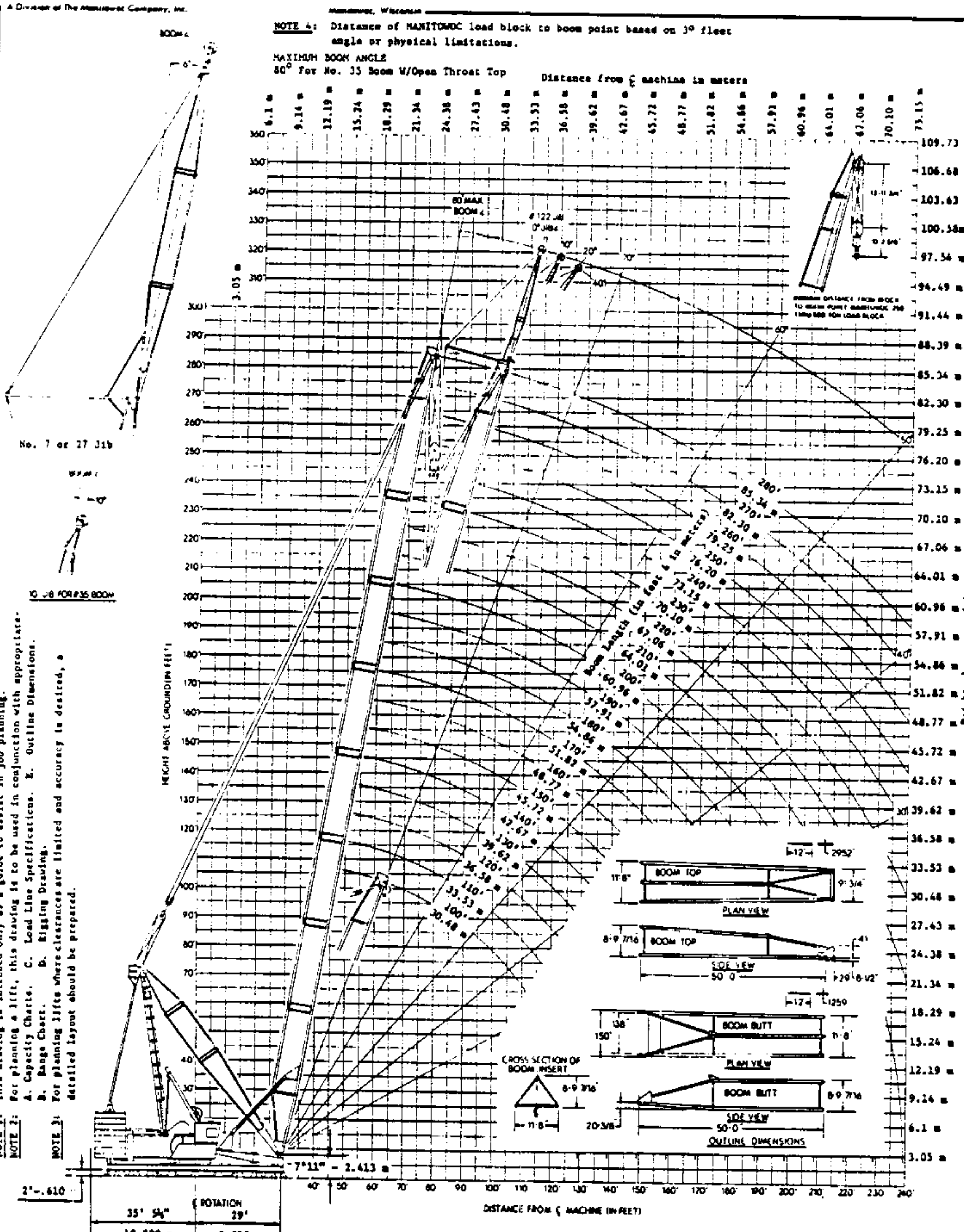
A Division of The Manitowoc Company, Inc. Manitowoc, Wisconsin

RANGE DIAGRAM-M4600-No.35 BOOM-60Ft. RINGER® SERIES 2

NOTE 4: Distance of MANITOWOC load block to boom point based on 3° fleet angle or physical limitations.

MAXIMUM BOOM ANGLE
80° For No. 35 Boom W/Open Throat Top

NOTE 1: This drawing is intended only as a guide to assist in job planning.
NOTE 2: For planning a lift, this drawing is to be used in conjunction with appropriate- A. Capacity Charts. C. Load Line Specifications. E. Outline Dimensions. B. Range Chart. D. Rigging Drawing.
NOTE 3: For planning lifts where clearances are limited and accuracy is desired, a detailed layout should be prepared.



DO NOT SCALE THIS DRAWING

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10-30-72

65125

LIFTCRANE CAPACITIES — MEETS ANSI B30.5 REQUIREMENTS — 4600 SERIES – 3 RINGER® SERIES – 2

BOOM NO. 37 — 60 FT. RINGER ATTACHMENT — ON BLOCKING 760,000 LB. AUXILIARY COUNTERWEIGHT

LIFTING CAPACITIES: Capacities for various boom lengths and operating radii may be based on per cent of tipping, strength of structural components, operating speeds and other factors.

Capacities are for freely suspended loads and do not exceed **75%** of static tipping loads. Capacities based on structural competence are shown by shaded areas.

Capacities are shown in pounds. Weight of jib, (see chart A), all load blocks, hooks, weight ball, slings, etc., is considered part of the main boom load. Boom is not to be lowered beyond radii where combined weights are greater than rated capacity. Where no capacity is shown, operation is not intended or approved.

OPERATING CONDITIONS: Machine to operate with roller path level within a tolerance of 1¼" in 60 ft. and properly supported and under conditions referred to in rigging drawing No. 65210 and load line specification chart.

Crane operator judgment must be used to allow for dynamic load effects of swinging, hoisting or lowering, as well as adverse operating conditions or physical machine depreciation.

OPERATING RADIUS: Operating radius is the horizontal distance from the axis of rotation to the center of vertical hoist line or load block with the load freely suspended.

Boom angle is the angle between horizontal and a line through the boom hinge that is parallel to the lower chords of inserts and boom top. Boom angle is an indication of operating radius, but in all cases, operating radius shall govern capacity.

BOOM POINT ELEVATION: Boom point elevation, in feet, is the vertical distance from ground level to centerline of boom point shaft.

MACHINE EQUIPMENT: Machine equipped with 26'1" crawlers, 60" treads, 30' heavy duty mast, 30 part independent boom hoist reeving, eight 1¾" galvanized boom support strand pendants, 80' fixed auxiliary mast, 112,000 lb. crane counterweight and 760,000 lb. auxiliary counterweight.

LOAD LINE SPECIFICATIONS: For machine with 30" width tandem drums and 43" width tandem drums, refer to Load Line Specification Chart No. 6429 or 6431.

MAXIMUM BOOM AND JIB LENGTHS LIFTED UNASSISTED		(A) DEDUCT FROM CAPACITIES WHEN JIB IS ATTACHED	
Boom Length	Jib No. 27	Jib Length	Jib No. 27
280'	—	80'	44,300 Lb.
260'	120'	100'	51,700 Lb.
		120'	57,300 Lb.
Load block, hook and weight ball on ground at start.			

For jib capacities, consult jib chart.

Boom Lgth.: Feet	Oper. Rad.: Feet	Boom Angle: Deg.	Boom Point: Elev.	Capacity:
100	48	79.9	106.1	1,200,000
	50	78.7	105.7	1,200,000
	52	77.5	105.2	1,200,000
	55	75.8	104.5	1,200,000
	57	74.6	103.9	1,200,000
	60	72.8	103.0	1,189,300
	65	69.6	101.2	1,158,700
	70	66.7	99.1	1,129,700
	75	63.5	96.7	1,055,900
	80	60.2	93.9	950,000
	85	56.6	90.8	862,900
	90	53.3	87.2	790,000
	95	49.6	83.1	712,400
	100	45.6	78.4	636,100
	105	41.4	72.9	565,600
	110	36.8	66.6	502,300
	115	31.6	59.0	444,000
	120	25.4	49.4	388,200
120	52	79.7	125.7	1,172,200
	55	78.2	125.1	1,156,700
	57	77.2	124.6	1,146,700
	60	75.7	123.9	1,132,100
	65	73.3	122.4	1,108,800
	70	70.7	120.7	1,086,300
	75	68.2	118.8	1,053,000
	80	65.6	116.6	947,100
	85	62.9	114.1	860,000
	90	60.2	111.3	787,100
	95	57.4	108.1	725,100
	100	54.4	104.7	671,900
	105	51.4	100.8	625,600
	110	48.3	96.5	585,000
	115	44.9	91.6	549,100
	120	41.4	86.2	503,700
	125	37.6	79.9	457,700
	130	33.4	72.7	411,300
	135	28.7	64.2	367,500
140	55	79.9	145.5	1,070,600
	57	79.1	145.1	1,062,900
	60	77.8	144.4	1,051,300
	65	75.7	143.2	1,029,000
	70	73.6	141.8	1,007,900
	75	71.4	140.2	987,800
	80	69.3	138.3	944,500
	85	67.0	136.2	857,400
	90	64.8	133.9	784,500
	95	62.5	131.4	722,500
	100	60.1	128.6	669,300
	105	57.[illegible]	125.5	623,000
	110	55.[illegible]	122.1	582,400
	115	52.[illegible]	118.4	546,500
	120	50.[illegible]	114.3	514,600
140	125	47.3	109.8	485,900
	130	44.4	104.9	460,100
	135	41.4	99.4	436,700
	140	38.2	93.2	409,500
	145	34.7	86.3	378,600
	150	30.8	78.4	344,500
	155	26.5	69.0	310,000
160	60	79.4	164.9	965,800
	65	77.5	163.8	949,900
	70	75.7	162.6	934,700
	75	73.8	161.2	920,100
	80	72.0	159.6	906,100
	85	70.1	157.8	854,300
	90	68.1	155.8	781,300
	95	66.2	153.7	719,400
	100	64.2	151.3	666,200
	105	62.2	148.7	619,900
	110	60.1	145.9	579,300
	115	58.0	142.8	543,400
	120	55.9	139.5	511,400
	125	53.7	135.9	482,800
	130	51.4	132.0	457,000
	135	49.0	127.8	433,600
	140	46.6	123.2	412,300
	145	44.1	118.1	392,800
	150	41.4	112.6	375,000
	155	38.6	106.5	358,500
	160	35.6	99.8	340,000
	165	32.3	92.2	316,300
	170	28.7	83.6	292,000
180	65	78.9	184.3	786,700
	70	77.3	183.2	772,100
	75	75.7	181.9	758,400
	80	74.0	180.5	745,500
	85	72.4	179.0	733,200
	90	70.7	177.3	721,600
	95	69.0	175.4	710,500
	100	67.2	173.3	664,500
	105	65.5	171.1	618,200
	110	63.7	168.7	577,600
	115	61.9	166.1	541,700
	120	60.1	163.2	509,800
	125	58.2	160.2	481,100
	130	56.3	156.9	455,300
	135	54.4	153.4	431,900
	140	52.4	149.6	410,600
	145	50.4	145.6	391,200
	150	48.2	[illegible]	373,300
	155	46.1	136.5	356,900
	160	43.8	131.4	341,700
180	165	41.4	125.8	327,600
	170	38.9	119.8	314,500
	175	36.3	113.2	302,300
	180	33.5	105.9	288,500
	185	30.4	97.7	269,600
	190	27.0	88.4	251,200
200	70	78.6	203.7	731,800
	75	77.1	202.6	719,000
	80	75.7	201.3	706,900
	85	74.2	199.9	695,500
	90	72.7	198.4	684,700
	95	71.2	196.7	674,500
	100	69.6	194.9	661,200
	105	68.1	192.9	615,000
	110	66.5	190.8	574,400
	115	65.0	188.5	538,500
	120	63.4	186.0	506,500
	125	61.7	183.4	477,900
	130	60.1	180.5	452,100
	135	58.4	177.5	428,700
	140	56.7	174.3	407,400
	145	55.0	170.8	387,900
	150	53.2	167.2	370,100
	155	51.4	163.2	353,600
	160	49.5	159.0	338,400
	165	47.6	154.6	324,300
	170	45.6	149.8	311,300
	175	43.5	144.6	299,100
	180	41.4	139.1	287,700
	185	39.2	133.1	277,000
	190	36.8	126.6	267,000
	195	34.3	119.5	257,600
	200	31.7	111.7	245,600
	205	28.8	102.9	230,200
	210	25.6	93.0	215,000
220	70	79.7	224.1	645,600
	75	78.3	223.1	636,300
	80	77.0	221.9	627,400
	85	75.6	220.7	619,000
	90	74.3	219.3	611,000
	95	72.9	217.8	603,400
	100	71.6	216.2	596,100
	105	70.2	214.4	589,100
	110	68.8	212.5	575,600
	115	67.4	210.4	539,700
	120	66.0	208.2	507,700
	125	64.5	205.9	479,100
	130	63.1	203.4	453,300
	135	61.6	200.7	429,900
	140	60.1	197.9	408,600
220	145	58.6	194.9	389,100
	150	57.0	191.7	371,300
	155	55.5	188.3	354,800
	160	53.8	184.7	339,700
	165	52.2	180.9	325,600
	170	50.5	176.8	312,500
	175	48.8	172.5	300,300
	180	47.0	167.9	288,900
	185	45.2	163.0	278,300
	190	43.4	157.9	268,300
	195	41.4	152.3	258,900
	200	39.4	146.3	250,000
	205	37.3	139.9	241,700
	210	35.0	133.0	233,800
	215	32.7	125.4	226,300
	220	30.1	117.1	216,300
	225	27.4	107.8	203,500

Capacities continued on reverse side.

Boom Lgth.: Fe[illegible]	Oper. Rad.: [illegible]	Boom Angle: Deg.	Boom Point: Elev.	Capacity:
240	.5	79.3	243.5	591,500
	30	78.1	242.4	584,000
	35	76.9	241.3	576,800
	90	75.6	240.0	570,000
	95	74.4	238 7	563,500
	100	73.2	237.2	557,300
	105	71.9	235.6	551,500
	110	70.6	233.8	545,900
	115	69.4	232.0	536,800
	120	68.1	230.0	504,900
	125	66.8	227.9	476,200
	130	65.5	225.6	450,400
	135	64.1	223.2	427,000
	140	62.8	220.7	405,700
	145	61.5	218.0	386,300
	150	60.1	215.2	368,400
	155	58.7	212.2	352,000
	160	57.3	209.0	336,800
	165	55.8	205.7	322,700
	170	54.4	202.1	309,600
	175	52.9	198.4	297,400
	180	51.4	194.5	286,000
	185	49.8	190.3	275,400
	190	48.2	185.9	265,400
	195	46.6	181.3	256,000
	200	44.9	176.3	247,100
	205	43.2	171.1	238,800
	210	41.4	165.5	230,900
	215	39.6	159.6	223,500
	220	37.6	153.2	216,400
	225	35.6	146.4	209,700
	230	33.5	139.1	203,300
	235	31.2	131.1	197,200
	240	28.8	122.3	187,100
	245	26.2	112.5	176,200
260	80	79.0	262.9	520,700
	85	77.9	261.8	515,400
	90	76.8	260.7	510,300
	95	75.6	259.4	505,400
	100	74.5	258.0	500,800
	105	73.3	256.6	496,400
	110	72.2	255.0	492,100
	115	71.0	253.3	488,100
	0	69.8	251.5	484,200
	5	68.7	249.5	474,100
	130	67.5	247.5	448,300
	135	66.3	245.3	424,900
	140	65.1	243.0	403,600
	145	63.8	240.6	384,100
	150	62.6	238.0	366,300
	155	61.3	235.4	349,800
	160	60.1	232.5	334,600
	165	58.8	229.5	320,600
	170	57.5	226.4	307,500
	175	56.2	223.1	295,300
	180	54.8	219.6	283,900
	185	53.5	215.9	273,300
	190	52.1	212.1	263,300
	195	50.7	208.0	253,900
	200	49.2	203.8	245,000
	205	47.7	199.3	236,700
	210	46.2	194.6	228,800
	215	44.7	189.6	221,300
	220	43.1	184.3	214,300
	225	41.4	178.8	207,500
	230	39.7	172.8	201,200
	235	37.9	166.6	195,100
	240	36.1	159.8	189,300
	245	34.2	152.6	174,000
	250	32.1	144.9	159,200
	255	30.0	136.5	145,000
	260	27.7	127.3	131,400

Boom Lgth.: Feet	Oper. Rad.: Feet	Boom Angle: Deg.	Boom Point: Elev.	Capacity:
280	80	79.8	283.2	478,600
	85	78.8	282.3	474,500
	90	77.7	281.2	470,500
	95	76.7	280.0	466,800
	100	75.6	278.8	463,300
	105	74.6	277.4	460,000
	110	73.5	275.9	456,800
	115	72.4	274.4	453,900
	120	71.3	272.7	451,200
	125	70.3	270.9	448,700
	130	69.2	269.1	445,300
	135	68.1	267.1	421,900
	140	67.0	265.0	400,600
	145	65.8	262.8	381,100
	150	64.7	260.4	363,300
	155	63.6	258.0	346,800
	160	62.4	255.4	331,600
	165	61.2	252.7	317,600
	170	60.1	249.8	304,500
	175	58.9	246.8	292,300
	180	57.7	243.7	280,900
	185	56.4	240.4	270,300
	190	55.2	237.0	260,300
	195	53.9	233.4	250,900
	200	52.7	229.6	242,000
	205	51.4	225.7	225,200
	210	50.0	221.6	208,200
	215	48.7	217.2	192,400
	220	47.3	212.7	177,500
	225	45.9	207.9	163,300
	230	44.4	202.9	150,000
	235	42.9	197.6	137,400
	240	41.4	192.0	125,400
	245	39.8	186.1	114,100
	250	38.2	179.9	103,200
	255	36.5	173.2	92,800
	260	34.7	166.2	82,800

Combined From Charts:

No. 6818-A	5-14-74
No. 6829-A	1-24-74
No. 6429	6-2-75
No. 6431	6-2-75

Form No. 6818-A, 6-2-75/L

SEE CONDITIONS ON REVERSE SIDE

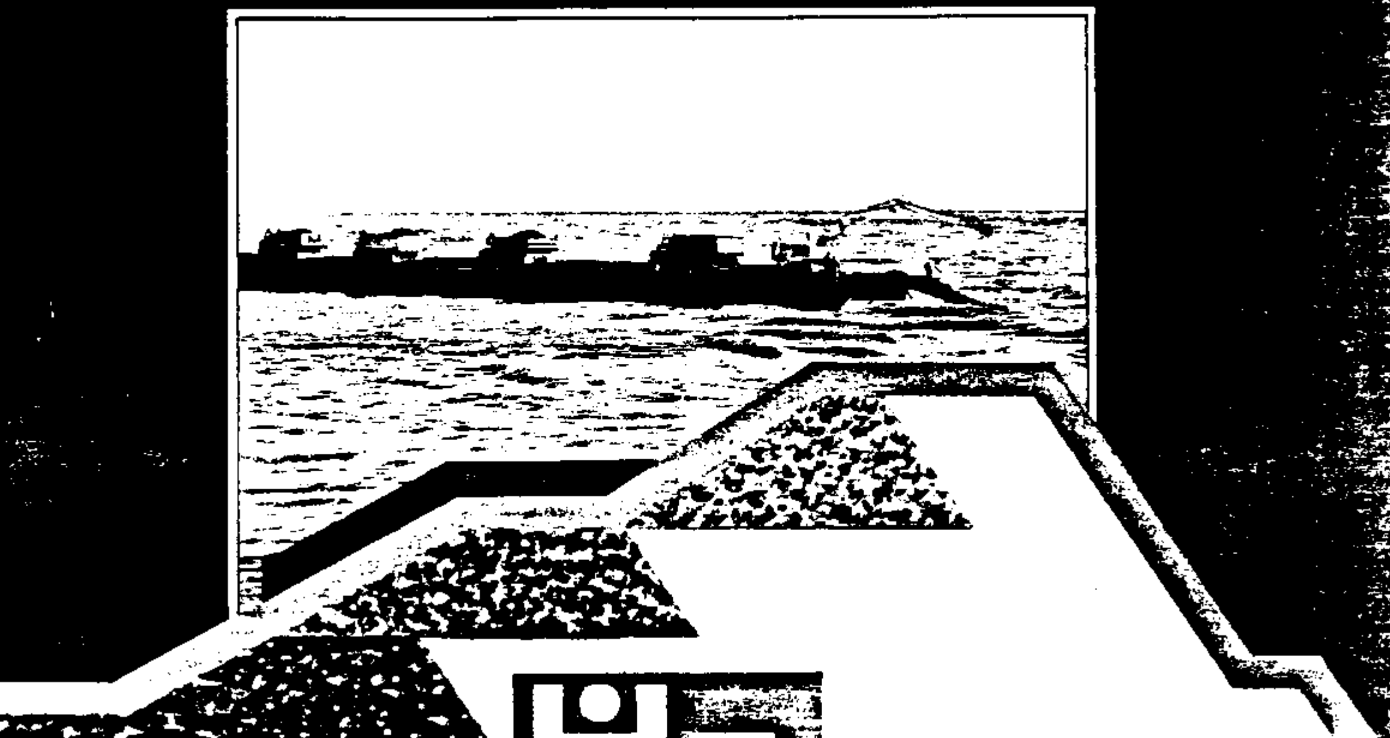
APPENDIX G

APPENDIX G

USE OF BITUMEN IN COSTAL STRUCTURES

Appendix G is a brochure supplied by Bitumarin, Company, demonstrating the several uses of Bitumen in hydraulic engineering projects. It gives details of the process as well as illustrating the different equipment used.

Bitumen in coastal hydraulic engineering



bitumarin

Introduction

Bituminous mixtures are being used with increasing frequency in hydraulic engineering. Originally these materials were used to replace clay and rock materials which are not always available everywhere in adequate quantities. Nowadays, however, bitumen is used in preference by virtue of its technological and economical advantages over traditional materials. It is flexible. Considering that in hydraulic engineering the sea bed is in constant motion, the binder should allow deformation easily. As binder, even in small amounts, it provides the locally available materials with extra possibilities.

Bitumarin: specialist in bitumen for hydraulic engineering.

It is not at all surprising that specifically the Dutch hydraulic engineering firm Bitumarin is the motive force behind the success of bituminous constructions in hydraulic engineering.

1. During the last decade gigantic hydraulic projects came into being in Holland, the challenge being the improvement of conventional constructions.
2. The activities of Bitumarin are concentrated fully on the development and realisation of bitumen techniques "in the wet": this being an unique specialisation, which, through constant research, has culminated in improved processes and by extrapolation of results gained by experience has reached high levels of perfection.
3. Development of modern equipment has opened the road to completely new methods.

The wide knowledge and broad experience of Bitumarin in bitumen techniques and in hydraulic engineering are at your disposal. Bitumarin likes to plan with you. A call at the right time can often lead to technically better alternatives which can save a lot of money.

Not only coastal engineering.

In coastal engineering Bitumarin is in its element, but also in other types of constructions, such as inland waterways and watertight constructions. Furthermore, Bitumarin can give you interesting solutions to offshore problems. Bitumarin has large and small equipment, matched to the size of the project: a harbour pier or a small ditch. For the smaller projects we will thankfully make use of standard equipment, but for the bigger ones large and sophisticated equipment is developed and constructed.

Bitumen techniques in coastal engineering.

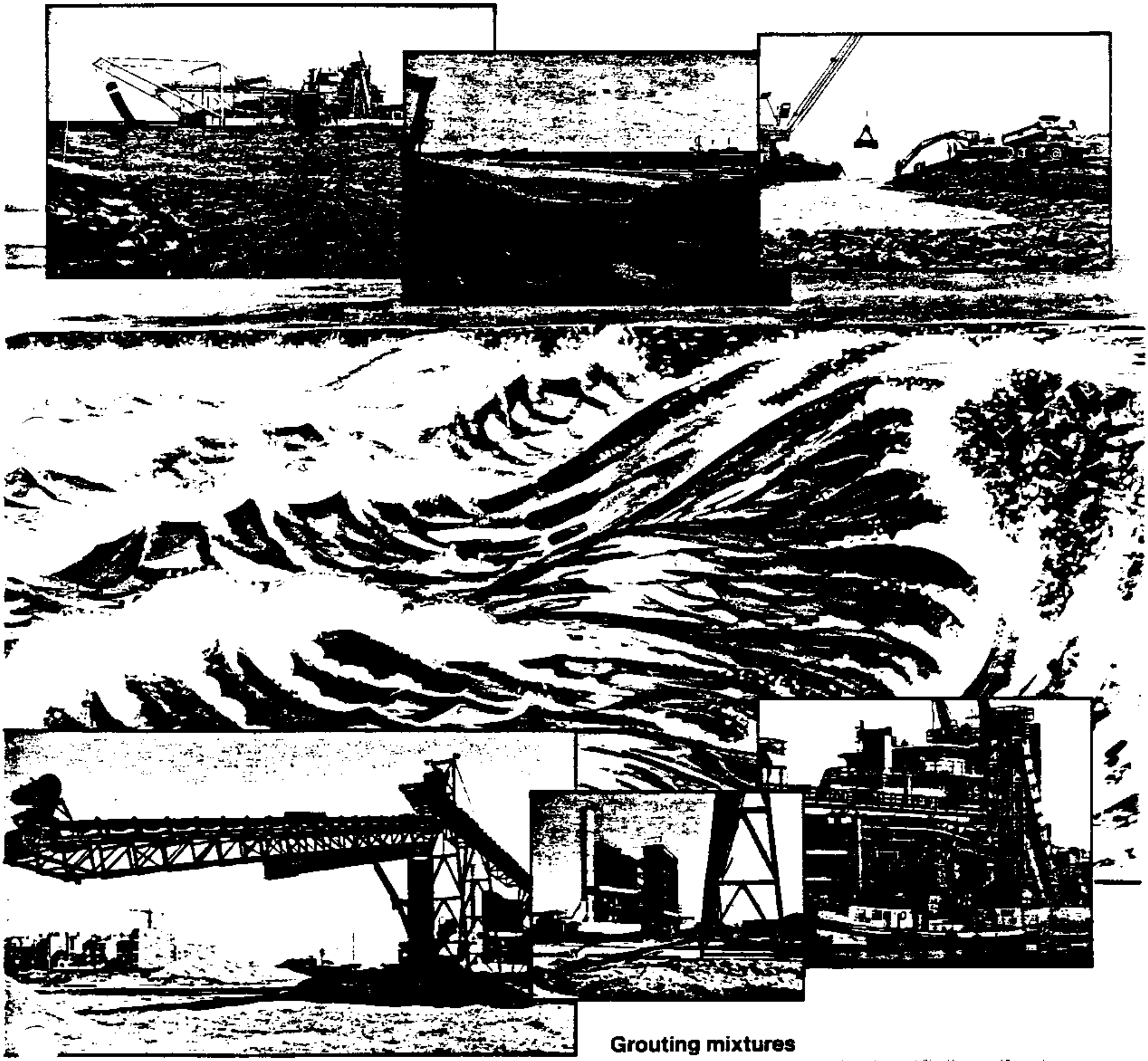
The following points speak for the application of bitumen in coastal engineering:

- greater flexibility of design
- relatively moderate quantities are needed when no loose materials (stones) are available
- less labour-intensive than the traditional construction methods
- high progress speed, this is especially important in case of tidal work
- also useful for temporary work, such as the construction of retaining dams.
- environmentally advantageous: in case of 'fixtone' and 'fixopen' vegetation possible, its colour becomes weathered, can be applied in different colours.

Bitumen is often used in coastal engineering. All the techniques applied by Bitumarin - and often developed by them - proved their succes under practical conditions.

Bed protection

Frequently the protection of the sea bed is not a goal on its own but a necessary provision to allow construction. Fixtone® mats, in situ poured sandmastic, weighted with rocks to resist the overpressure. This system was well proved in the Delta Works. Currents and eddies caused by construction obstacles usually make heavier constructions necessary. Sandmastic, stone-asphalt and Fixtone® may be considered.



Asphaltic concrete

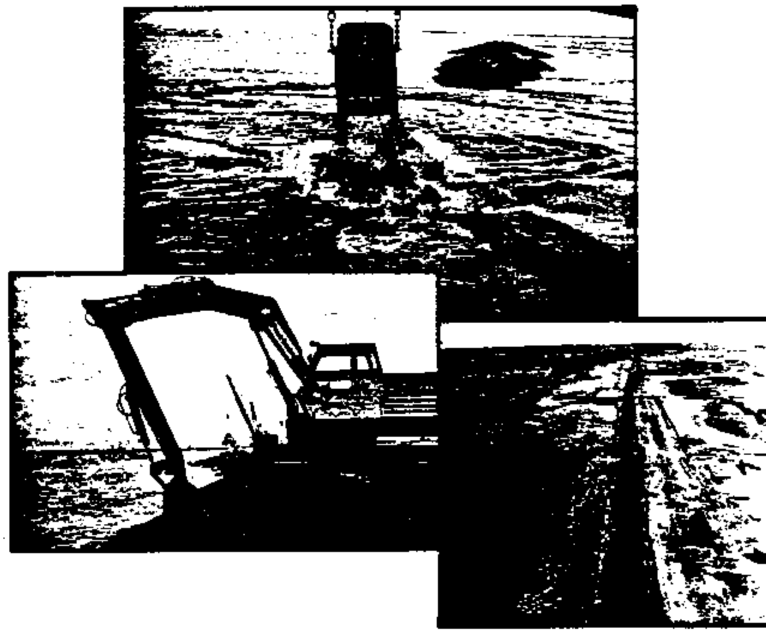
Asphaltic concrete slope protection represents the classical application of bitumen in hydraulic engineering. Applied "in the dry" for the watertight lining of steep slopes of dams and reservoirs.

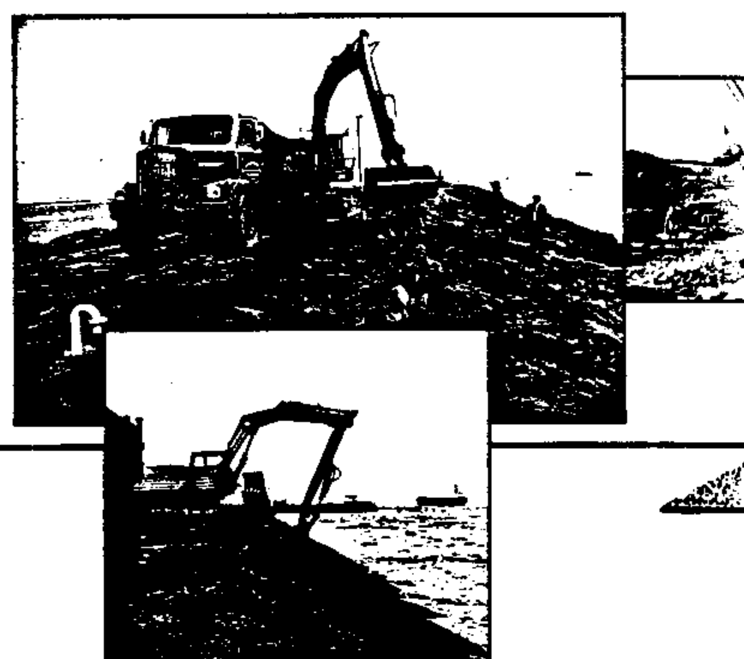
Grouting mixtures

Asphaltic grouting has been developed "in the wet" and "in the dry". It needs no compaction and its life-time is unlimited. For underwater applications Bitumarin uses special equipment, through which grout can be placed at any desired depth. Used as bed protection and for grouting discrete stones.

Breakwaters

Mostly in deep waters. Must resist the action of heavy waves. By preference their slopes are steep to save material, at the same time a gradual slope from the bottom to the top is desirable. So the compromise is a 1 : 2½ slope, with, for example, a limited height footing, such as a Fixtone®-mat. Under water the slope may be pattern grouted, but above water where the effect of the wave crests is felt, this pattern grouting may continue only if the waves are not too heavy (Hs = 4 m). With complete grouting even the heaviest seas can be resisted. Complete grouting is the logical continuation at the crown and on the inward slope, with the overtopping waves. In case of high breakwaters the obvious solution is a concrete crown. In this way a gradual change from horizontal to vertical can be achieved. For the core of the dam sand-bitumen is the obvious solution. The relative strength of this mixture makes it remarkably resistant against the action of the seas during the construction phase, much better than any material without a binder. Generally speaking, the application of bituminous mixtures, i.e., bonded materials, allows the use of simple designs with relatively thin protective layers and few or no intermediary layers. Also the weights to be handled during construction are relatively low. The use of asphalt is more competitive than it is often thought.





Stone-asphalt

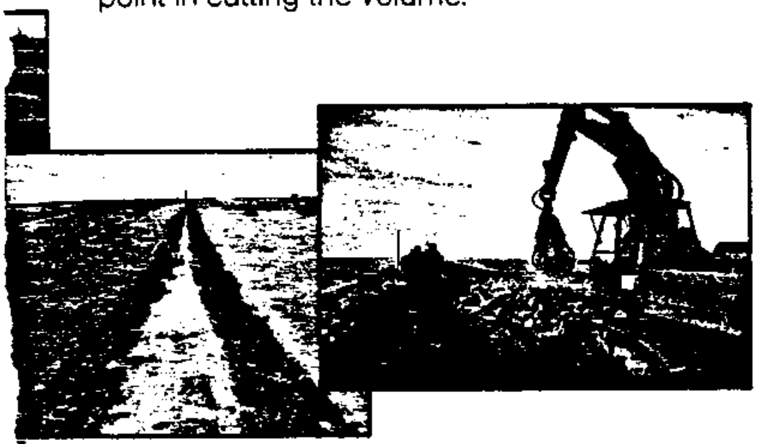
The excellent quality of grouted stone lead to the subsequent production of stone-asphalt, a mixture of asphalt, grout and stone up to 80 kg in weight. Placed under water it forms a monolith which is resistant against heavy seas and currents.

Pattern grouting

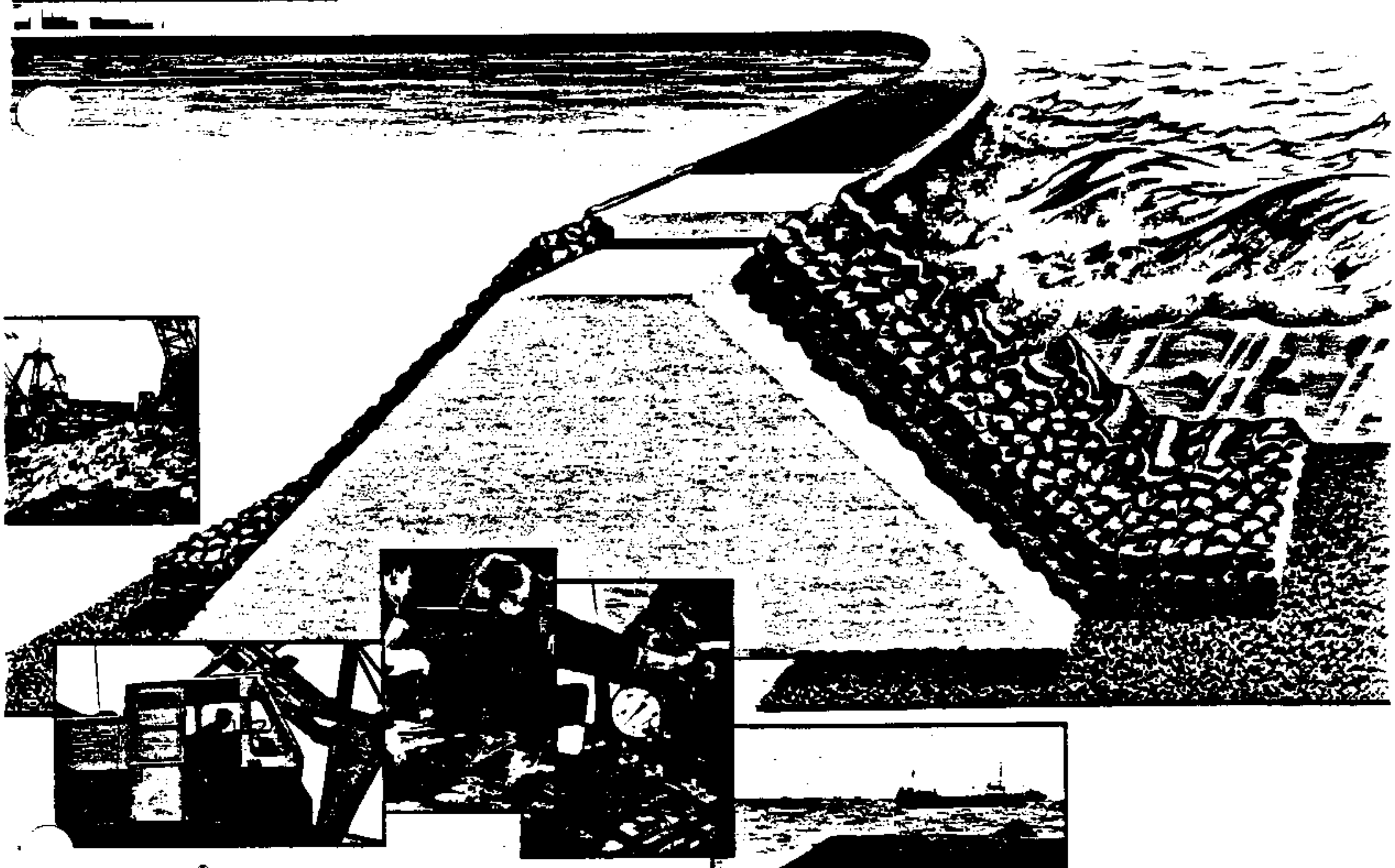
Bitumarin has developed a method of pattern grouting rock or stone in two ways. Lighter constructions are grouted with a low viscosity grout to stabilize the small stones c.q. coarse gravel. Heavy constructions are grouted with a high viscosity mix to a predetermined depth. This method of pattern grouting overcomes the disadvantages of impermeability. Despite the remaining voids the strength of the construction is comparable to fully grouted rock or stone.

Dams and dykes

During closing operations dams are usually constructed in artificially acquired shallow waters. Here the influence of the sea is only slight, both during and after the construction phase. As dams are usually associated with roads on top, customarily the slope is very moderate with slight wave run-up. Especially when the dam can be constructed with relatively inexpensive hydraulic fill, there is no point in cutting the volume.



Sand-bitumen retaining dams will provide adequate protection for the hydraulic fill, representing the ideal base for the final revetment. For the construction of the toe, which is so necessary in this case, in situ poured sandmastic may be used, and Fixtone®-mats are also useful. To protect the toe in the tidal zone the water-permeable Fixtone® is the ideal material. Grouted stone is also used at times, but in this case the developing water pressures may be so great that a much greater thickness(—weight) will be needed than that necessary for the strength.
Above high water level asphaltic concrete is preferable, dimensioned in accordance with the acting hydraulic pressure induced by pore-water within the body of the dam.

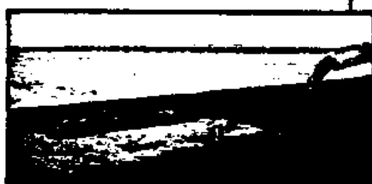


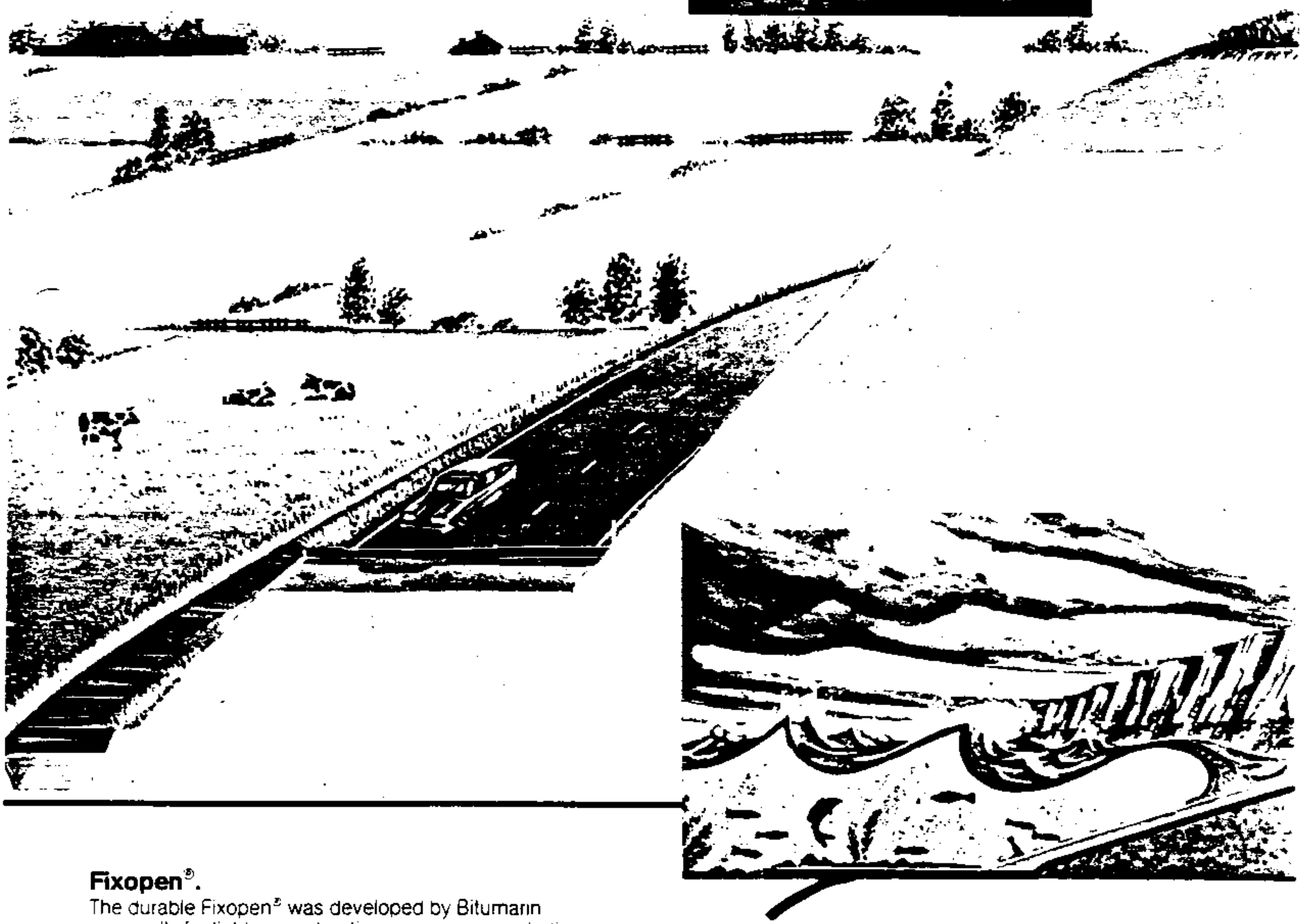
Fixtone®

A permeable and yet durable asphalt composition was a long awaited reality in hydraulic engineering, finally realised by Bitumarin with Fixtone®. A mixture of light stone and a small quantity of a grouting mixture, in which the pores remain open. On the deck of the purpose-built ship "Jan Heijmans" Fixtone® is produced in the form of mats, a flexible construction which may be placed to great depths. In thicker layers Fixtone is also used as protection for dykes and as protective layer on moderately exposed harbour structures.

Sea walls

Sea walls serve to protect low-lying land from the encroachment of the sea. On the seaward side such walls stand in shallow water. Since the horizontal width of construction must be restricted as far as possible, the slope is quite considerable. Also higher land may need protection against the sea. In Holland this usually concerns the protection of the bottom of dunes, while in other countries it may involve protection of cliffs threatened by erosion.





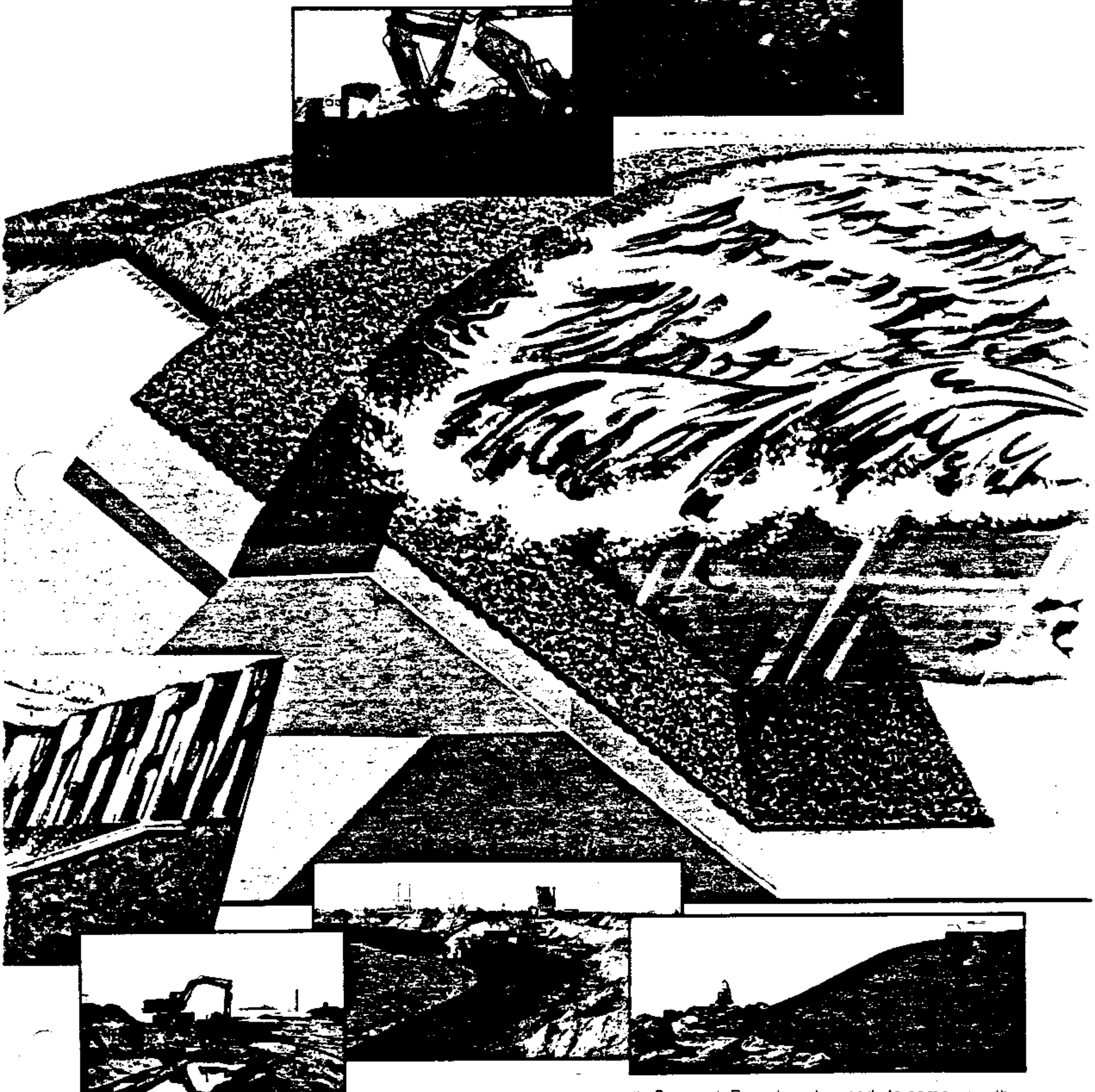
Fixopen®.
The durable Fixopen® was developed by Bitumarin especially for lighter constructions: an open penetration mortar. Sand-tight, but permeable. Finer than Fixtone®. Placed in a layer or as mattress it is intended for less heavily exposed applications.

Sand-bitumen
This is a mixture of sand and some (3%, 4%, sometimes more) bitumen. Its costs are relatively low. Very useful in massive applications where the moderate strength of this material is acceptable. As permeable as the sand it consists of: often a requirement in hydraulic engineering.

Bitumarin: construction firm for bituminous constructions in hydraulic engineering
Bitumarin is a specialised construction firm fully concentrating on bitumen techniques in hydraulic engineering. Established in 1960 as a subsidiary of then the leading companies in hydraulic bitumen technology Koninklijke Wegenbouw N.V., now the Roads and Asphalt Division of the Koninklijke Volker-Stevin concern, and the N.V. Verenigde Heijmans Bedrijven.

Headlands, groynes

For the protection of beaches short, low dams are constructed perpendicular to the shoreline. The limited construction height and the moderate slopes effectively suggest the application of bitumentechniques.



In 1971 the Hollandsche Beton Groep has joined these companies participating in Bitumarin. Bitumarin makes available to you all its specialised knowledge and experience. Its Research Department is ready to come up with the optimum solution for every specific problem. It acts as a neutral partner in discussions. Independent and competent.

APPENDIX H

APPENDIX H

SONAR PROFILERS

MesotecH Systems Ltd. supplied this section showing some of the leading equipment for underwater surveying. It describes a single axis, double axis and the super precision of the profiling sonars. It also includes some actual plotted records prepared for a trench survey, glory hole survey and pipe surveys.

MESOTECH

SONAR PROFILES
PRODUCED BY MODEL 952
PRECISION PROFILING SONAR

These are copies of actual plotted records, half the size of the originals, except #10 and #11.

#1, Page 2 is a series of profiles along a plowed trench.

#2, Page 3 is a single profile of a Glory Hole with Riser.

#3, Page 4 is the same Glory Hole as #2, to a larger scale.

#4, Page 5 is a series of 9 small plots on one page, with each profile on a different line of bearing.

#7, Page 6 shows two pipes in a trench, under ice.

#8, Page 7 shows 6 profiles of a pipe in a trench, under ice.

#9, Page 8 shows two scans of a Subsea Completion Manifold being pulled in to a Wellhead.

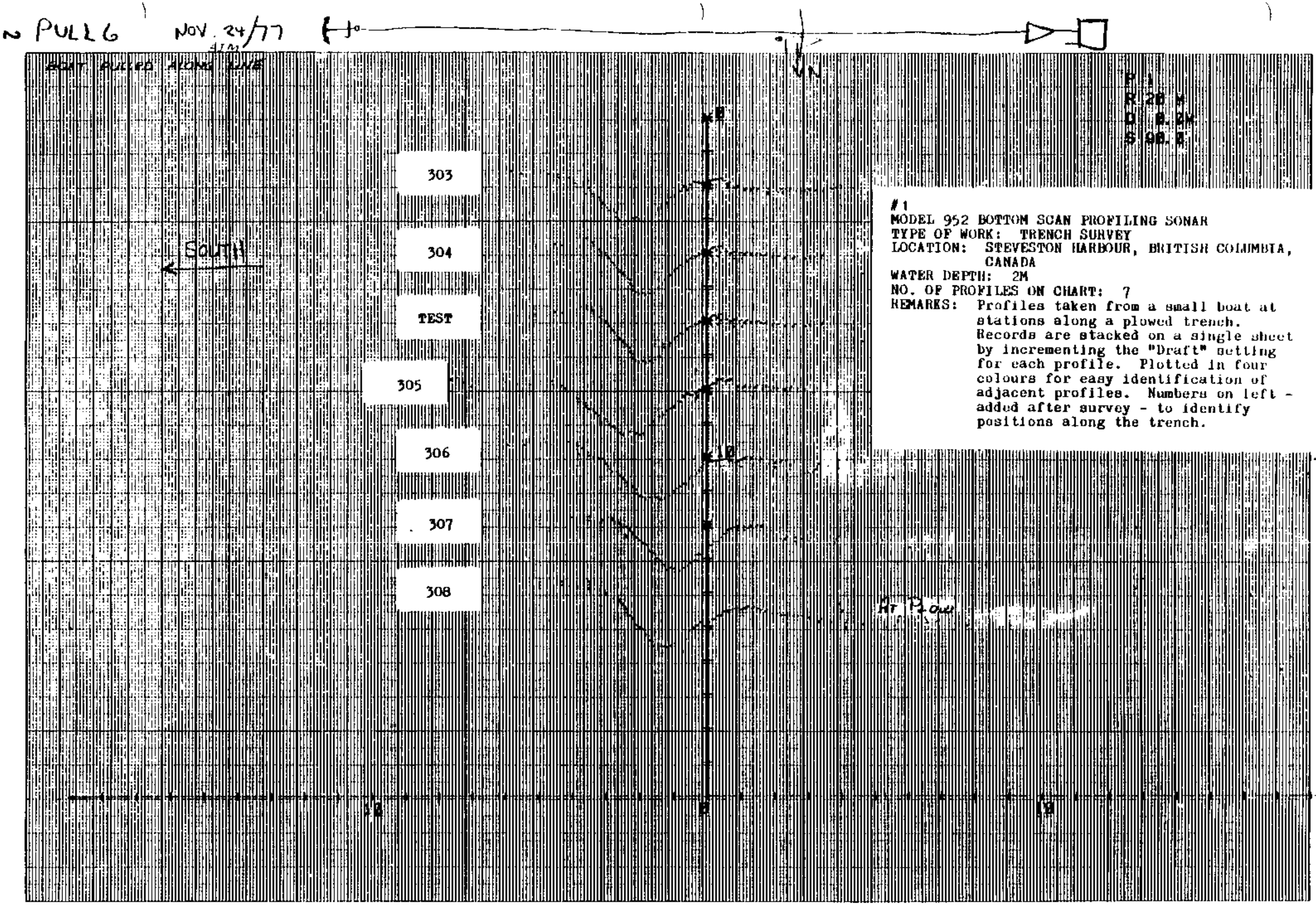
#10, Page 9 (Full Size) shows the dredging of a Glory Hole. The record is plotted in the CONTOUR mode.

#11, Page 10 (Full Size) shows a later stage from #10 with the Glory Hole enlarged at centre.

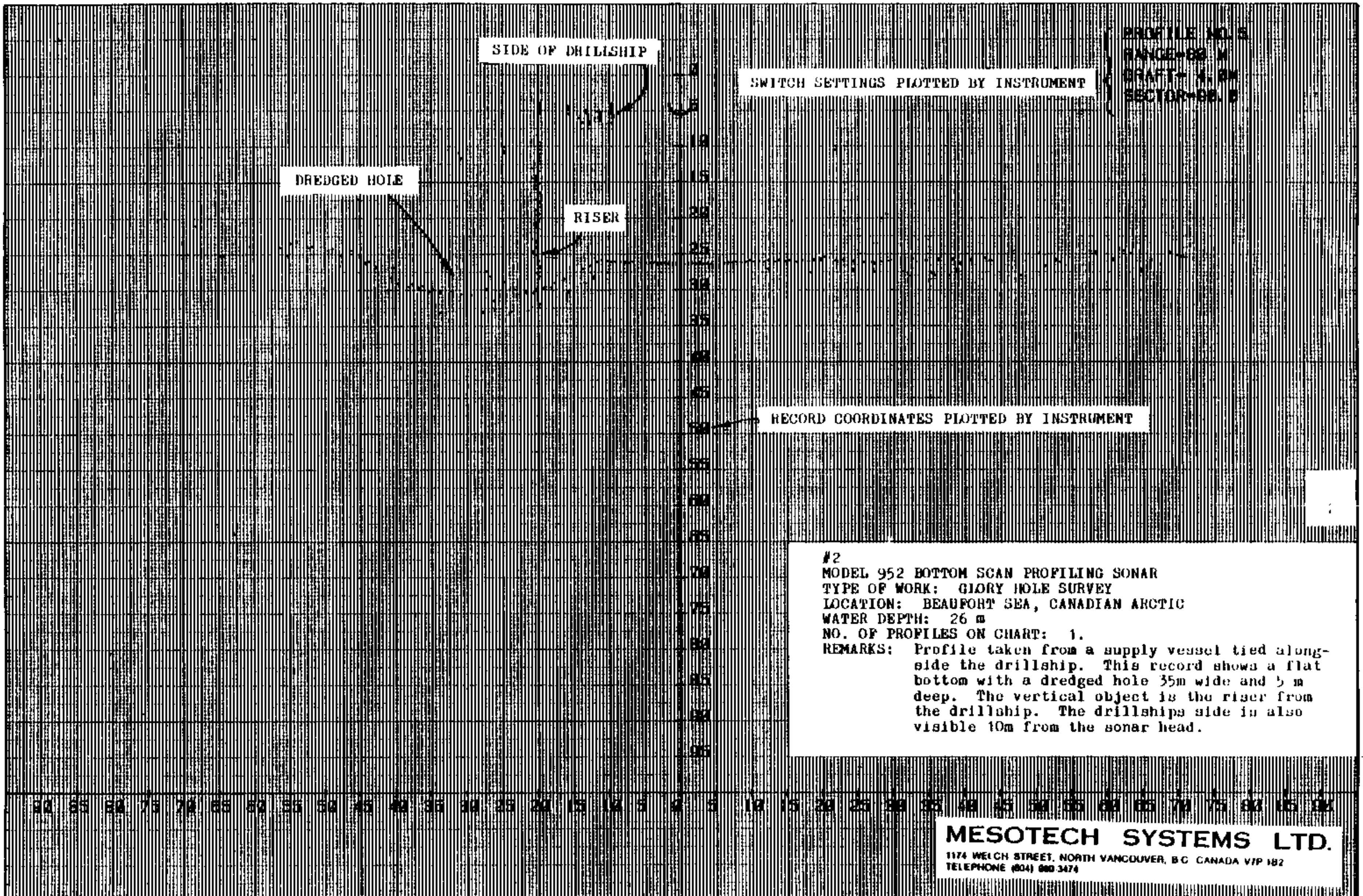
\- Page 11 952 Sonar on LR3 Submersible

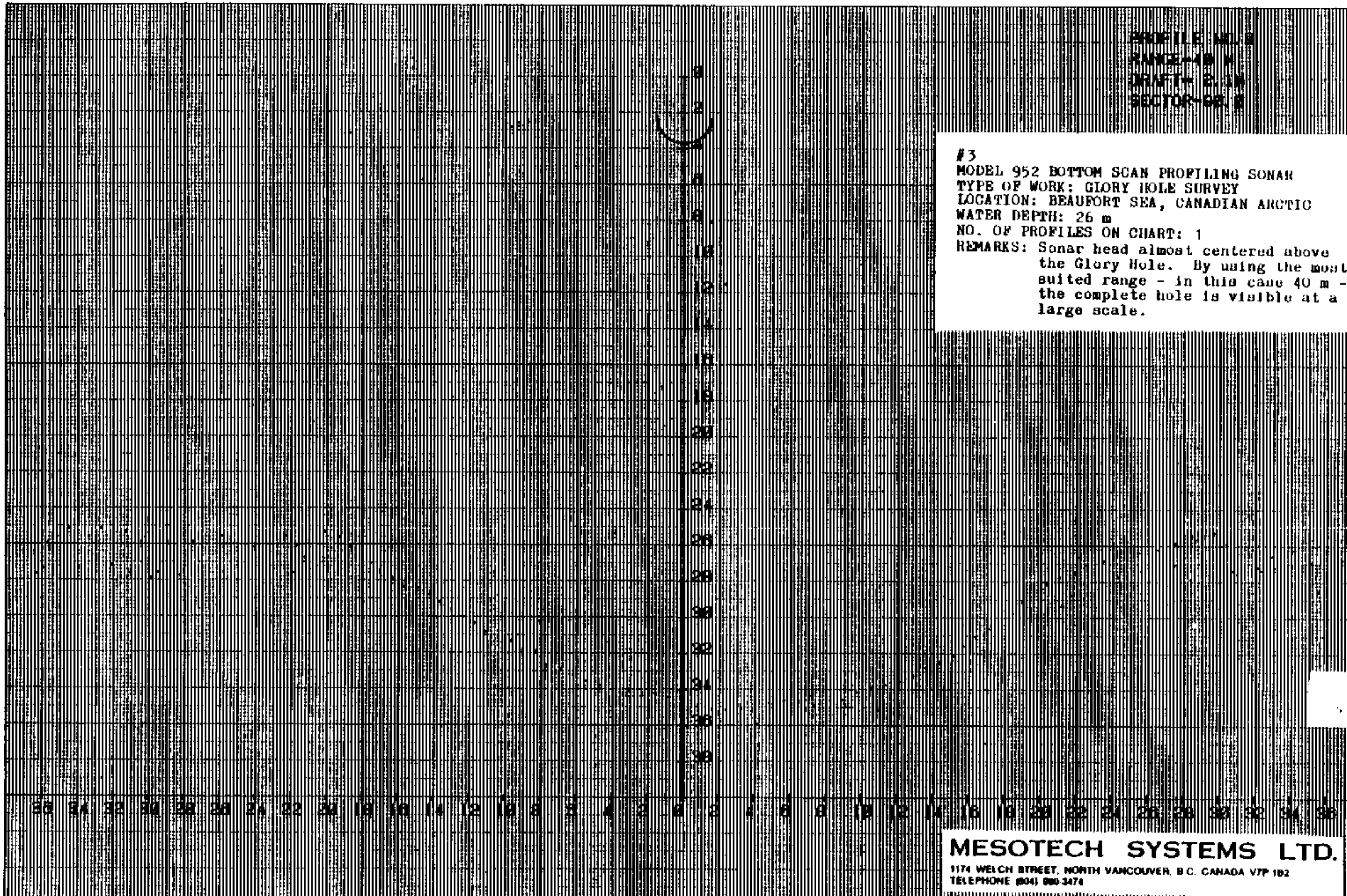
#12, Page 12 Trench profiles taken by 952/LR3

MESOTECH SYSTEMS LTD.
1174 Welch St. North Vancouver, B.C.
Canada. V7P 1B2
Tel. (604) 980-3474. Telex 04-352773.



#1
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: TRENCH SURVEY
LOCATION: STEVESTON HARBOUR, BRITISH COLUMBIA, CANADA
WATER DEPTH: 2M
NO. OF PROFILES ON CHART: 7
REMARKS: Profiles taken from a small boat at stations along a plowed trench. Records are stacked on a single sheet by incrementing the "Draft" setting for each profile. Plotted in four colours for easy identification of adjacent profiles. Numbers on left - added after survey - to identify positions along the trench.


SIDE OF DRILLSHIP
DREDGED HOLE
RISER
SWITCH SETTINGS PLOTTED BY INSTRUMENT
RECORD COORDINATES PLOTTED BY INSTRUMENT
#2
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: GLORY HOLE SURVEY
LOCATION: BEAUFORT SEA, CANADIAN ARCTIC
WATER DEPTH: 26 m
NO. OF PROFILES ON CHART: 1.
REMARKS: Profile taken from a supply vessel tied along-side the drillship. This record shows a flat bottom with a dredged hole 35m wide and 5 m deep. The vertical object is the riser from the drillship. The drillships side is also visible 10m from the sonar head.
MESOTECH SYSTEMS LTD.
1174 WELCH STREET, NORTH VANCOUVER, B.C. CANADA V7P 1B2
TELEPHONE (604) 980-3474


#3
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: GLORY HOLE SURVEY
LOCATION: BEAUFORT SEA, CANADIAN ARCTIC
WATER DEPTH: 26 m
NO. OF PROFILES ON CHART: 1
REMARKS: Sonar head almost centered above the Glory Hole. By using the most suited range - in this case 40 m - the complete hole is visible at a large scale.
MESOTECH SYSTEMS LTD.
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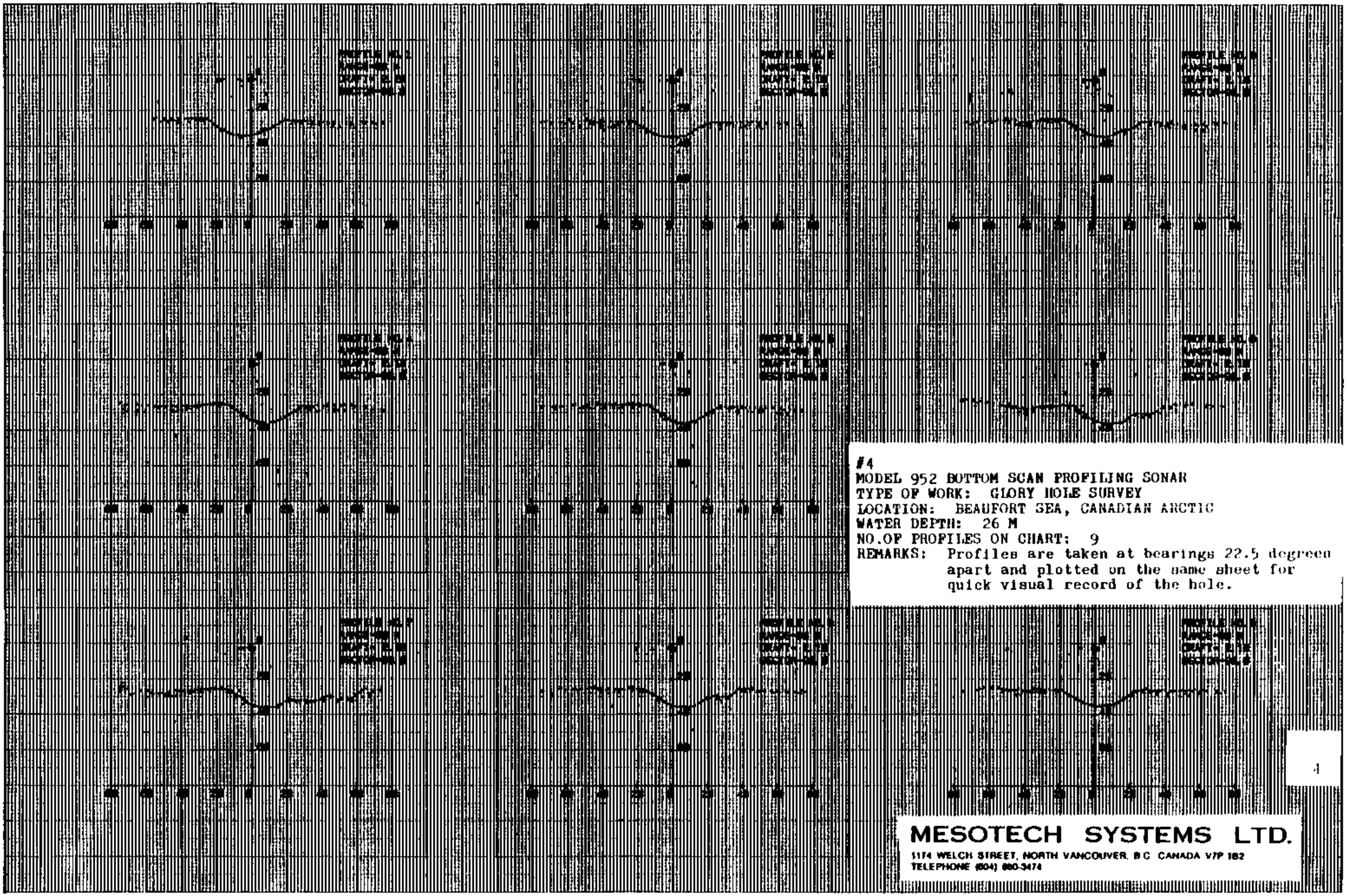

#4
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: GLORY HOLE SURVEY
LOCATION: BEAUFORT SEA, CANADIAN ARCTIC
WATER DEPTH: 26 M
NO.OF PROFILES ON CHART: 9
REMARKS: Profiles are taken at bearings 22.5 degrees apart and plotted on the same sheet for quick visual record of the hole.

4

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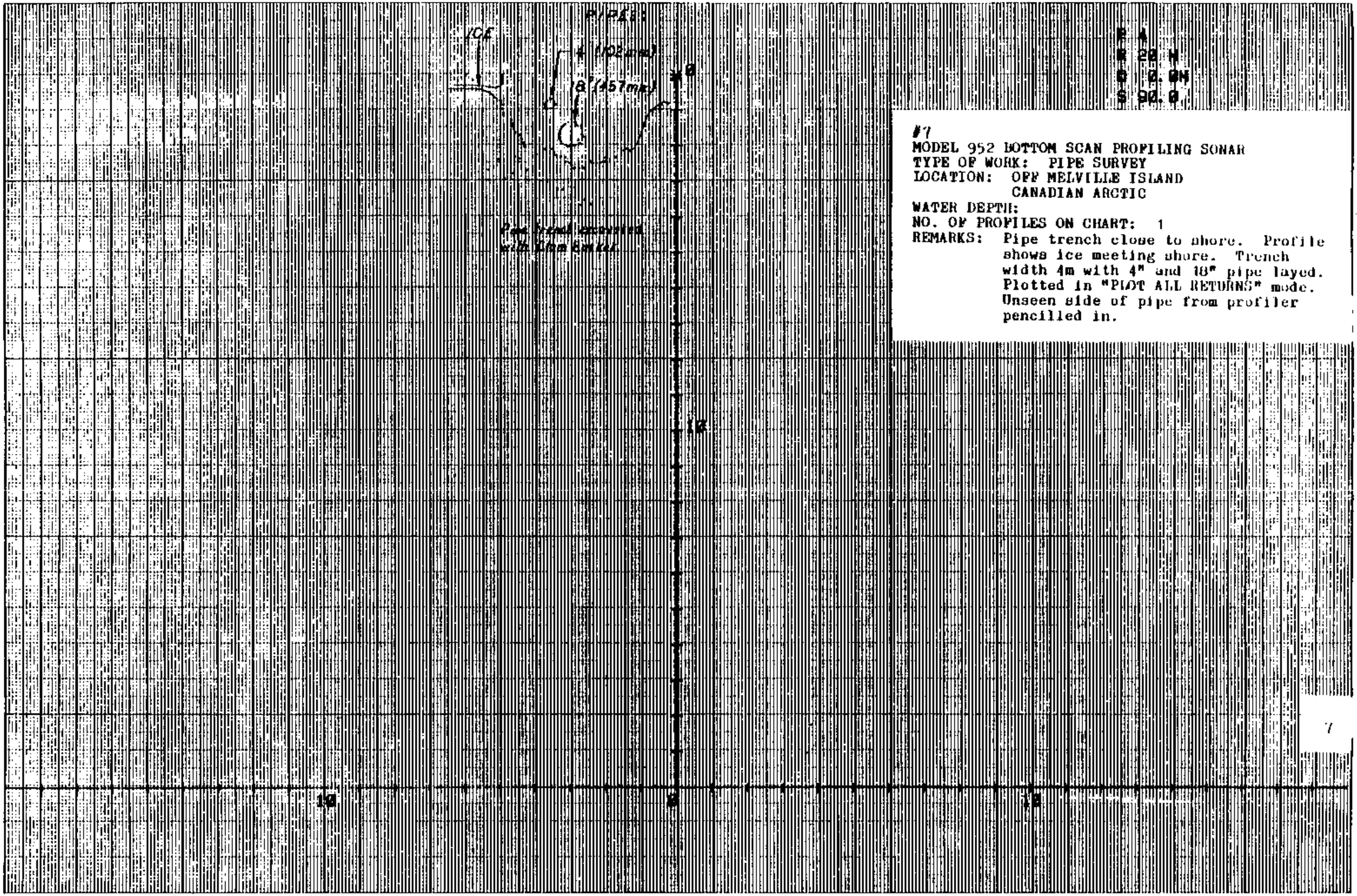
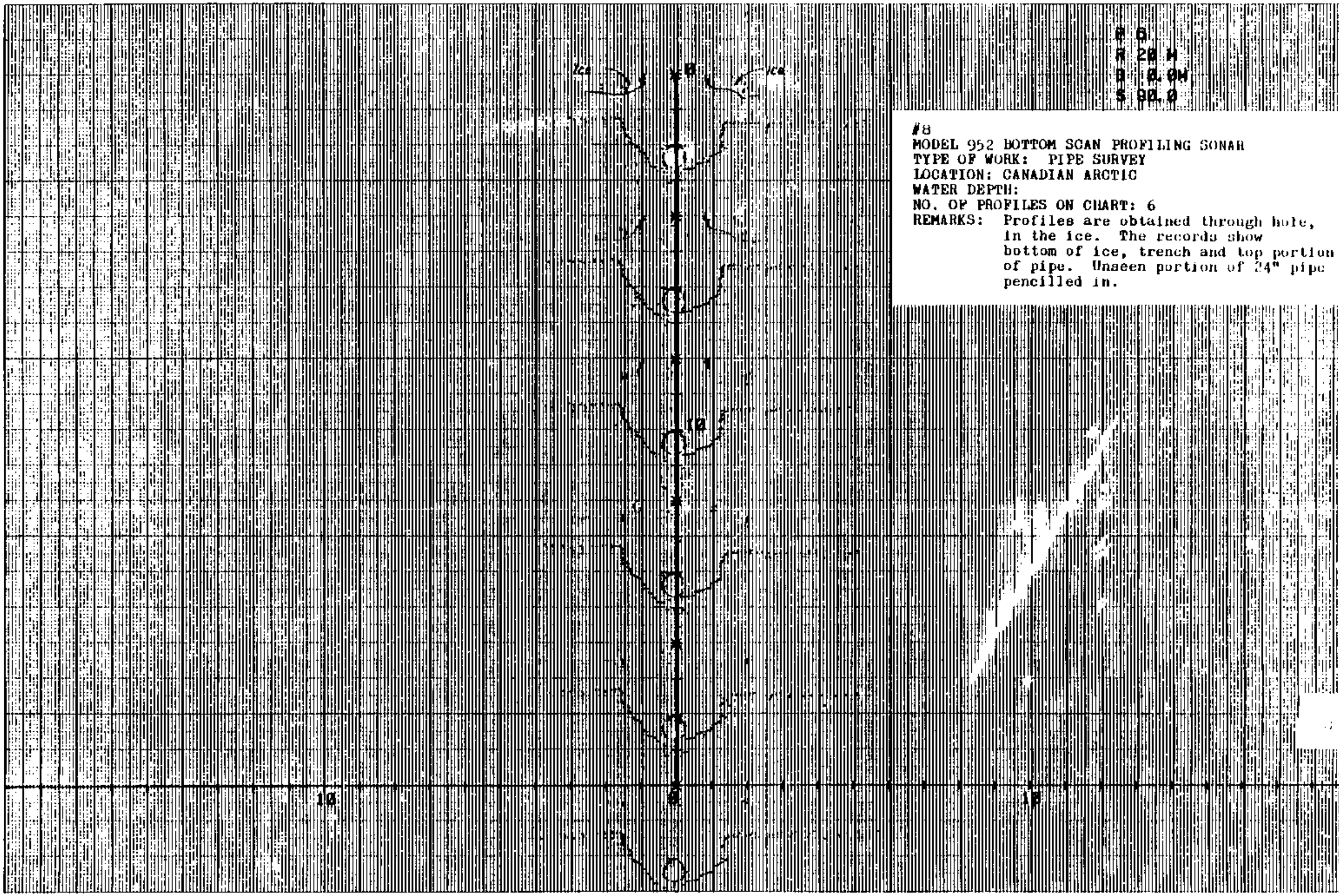
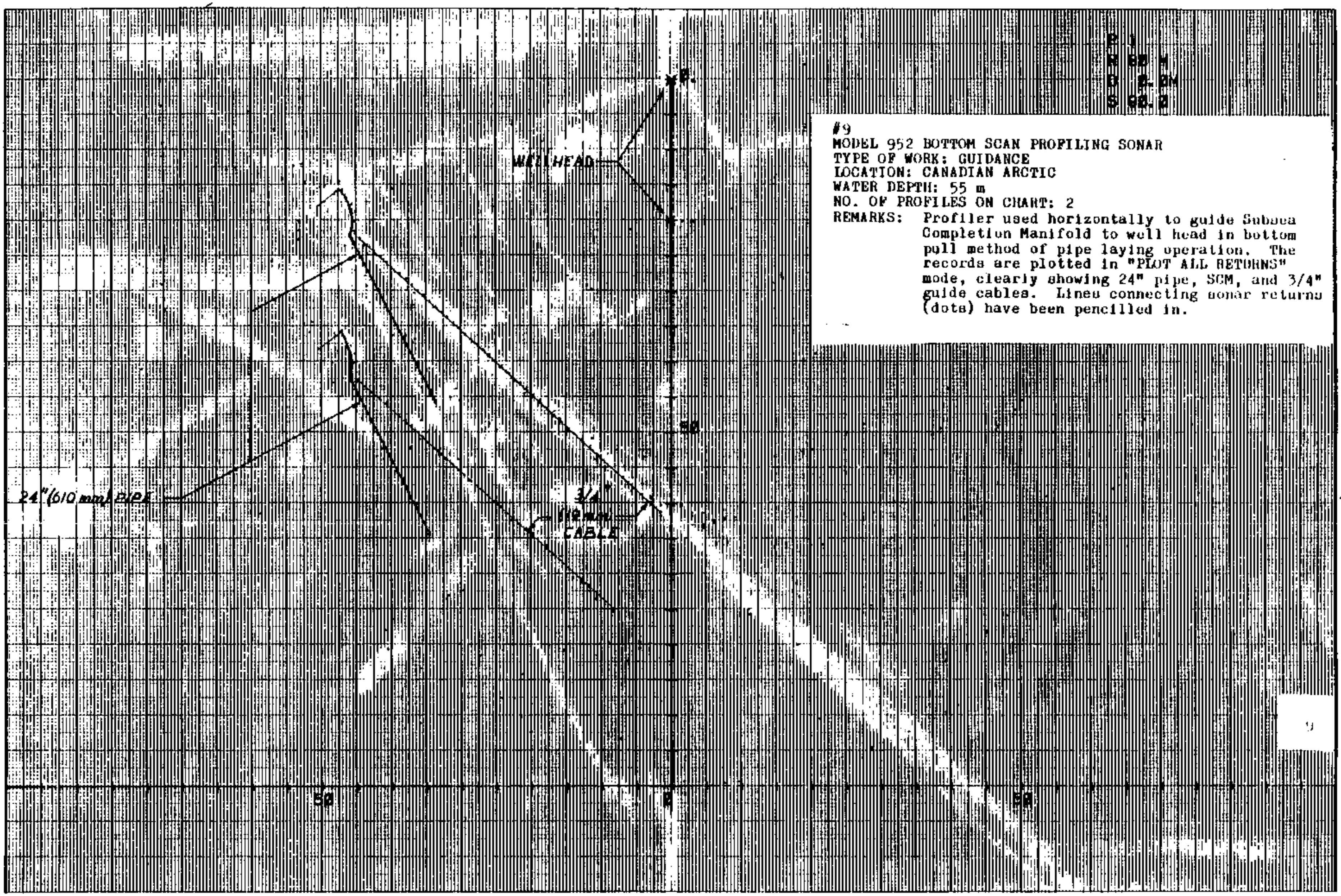


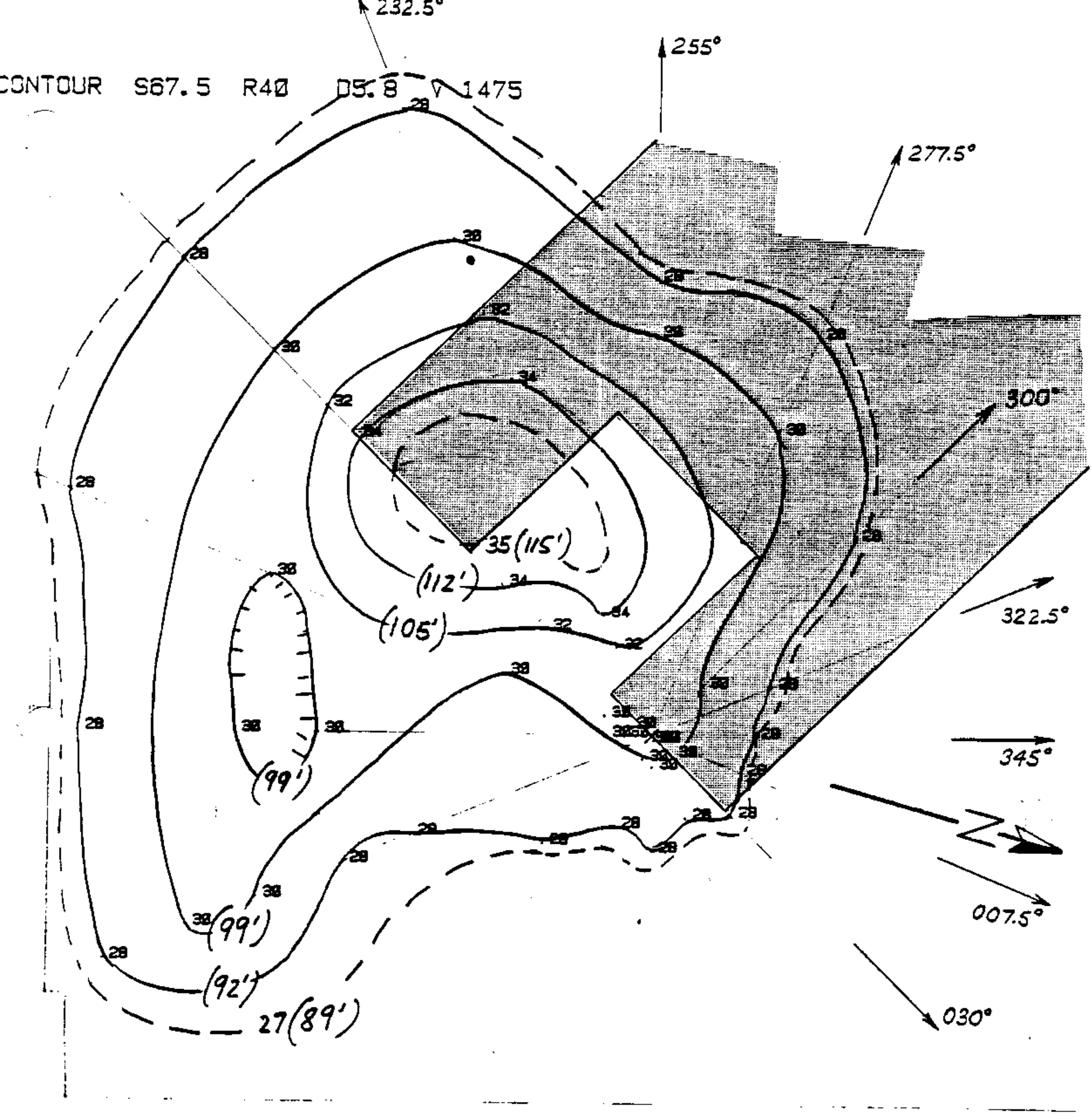
#7
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: PIPE SURVEY
LOCATION: OFF MELVILLE ISLAND
CANADIAN ARCTIC
WATER DEPTH:
NO. OF PROFILES ON CHART: 1
REMARKS: Pipe trench close to shore. Profile shows ice meeting shore. Trench width 4m with 4" and 18" pipe layed. Plotted in "PLOT ALL RETURNS" mode. Unseen side of pipe from profiler pencilled in.

6

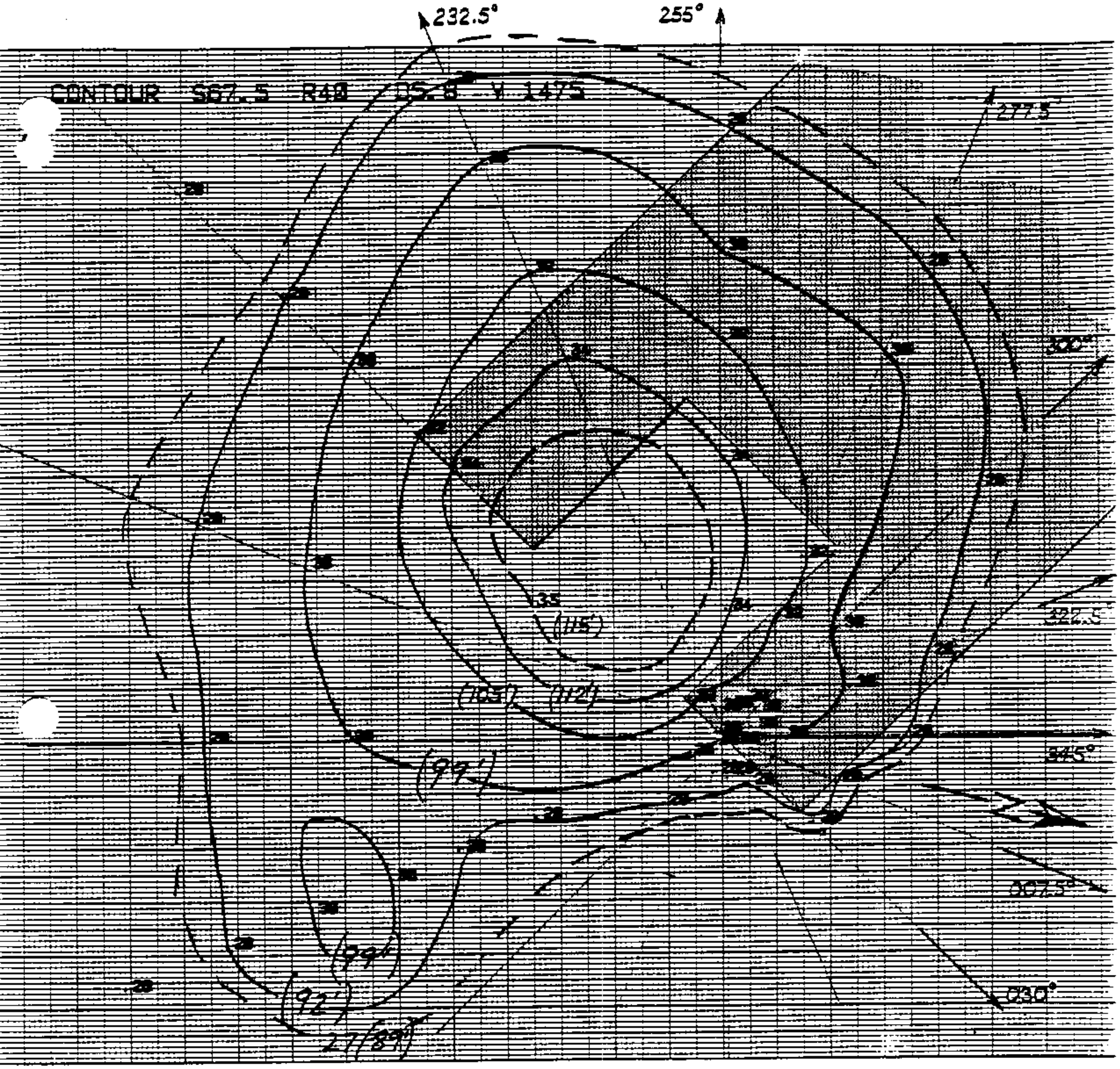

O 6
R 20 M
D 0.0M
S 90.0
#8
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: PIPE SURVEY
LOCATION: CANADIAN ARCTIC
WATER DEPTH:
NO. OF PROFILES ON CHART: 6
REMARKS: Profiles are obtained through hole, in the ice. The records show bottom of ice, trench and top portion of pipe. Unseen portion of 24" pipe pencilled in.
Ice
Ice
0
10
10
10



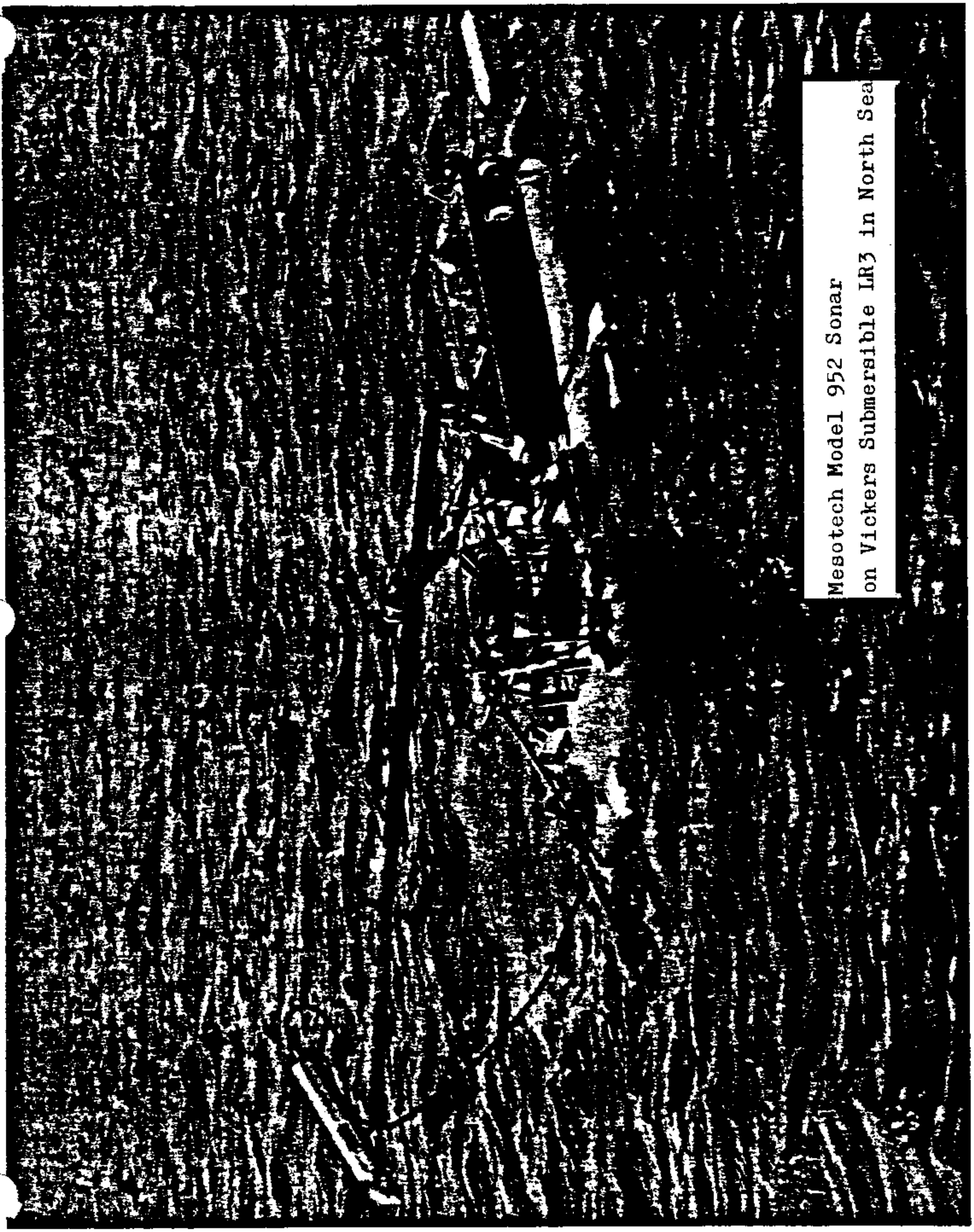
#9
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: GUIDANCE
LOCATION: CANADIAN ARCTIC
WATER DEPTH: 55 m
NO. OF PROFILES ON CHART: 2
REMARKS: Profiler used horizontally to guide Subsea Completion Manifold to well head in bottom pull method of pipe laying operation. The records are plotted in "PLOT ALL RETURNS" mode, clearly showing 24" pipe, SCM, and 3/4" guide cables. Lines connecting sonar returns (dots) have been pencilled in.



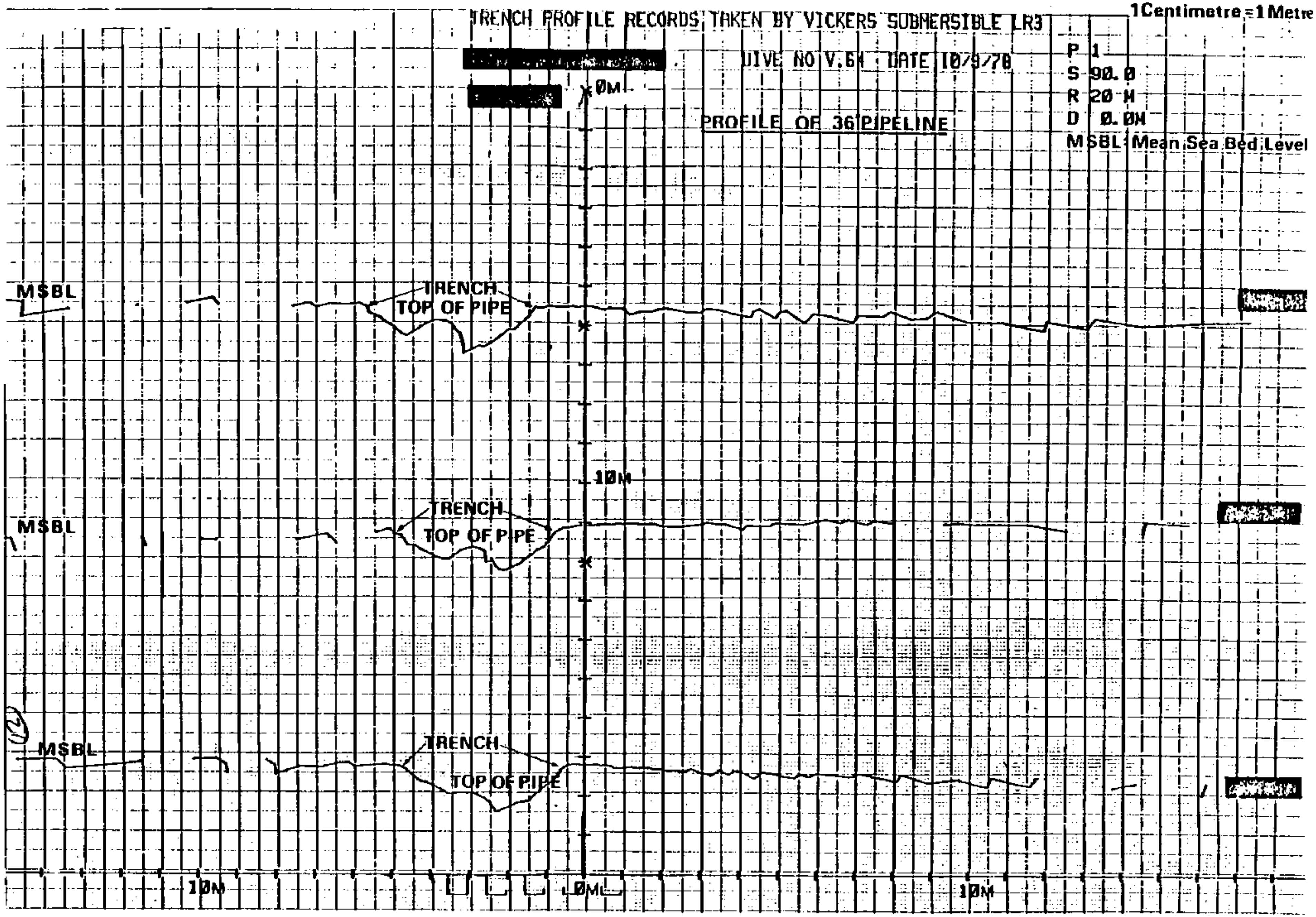
10
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: GLORY HOLE DREDGING PROGRESS MONITORING
LOCATION: CANADIAN ARCTIC
WATER DEPTH: 26.5 M
NO. OF PROFILES ON CHART: 8
REMARKS: Profiles taken from a dredging barge. Profile bearings are 22.5 degrees apart. Plotted in contour mode. Depths indicated at crossings of 2m elevation intervals. Contour lines and dredge location pencilled in.



#11
MODEL 952 BOTTOM SCAN PROFILING SONAR
TYPE OF WORK: GLORY HOLE DREDGING PROGRESS MONITORING
LOCATION: CANADIAN ARCTIC
WATER DEPTH: 26.5m
NO. OF PROFILES ON CHART: 8
REMARKS: Same hole as displayed on graph #10 after 9 hours of dredging.
Record indicates dredging progress in centre of Glory Hole.
Contour lines and dredge locations pencilled in.



Mesotech Model 952 Sonar
on Vickers Submersible LR3 in North Sea


TRENCH PROFILE RECORDS TAKEN BY VICKERS SUBMERSIBLE LR3
1Centimetre=1Metre
[redacted]
DIVE NO V.6M DATE 10/9/78
P 1
S 90.0
R 20 M
D 0.0M
MSBL: Mean Sea Bed Level
PROFILE OF 36 PIPELINE
ØM
10M
MSBL
TRENCH
TOP OF PIPE
MSBL
TRENCH
TOP OF PIPE
MSBL
TRENCH
TOP OF PIPE
10M
ØM
10M
(2)

MESOTECH SYSTEMS LTD.
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MODEL 952 SINGLE AXIS BOTTOM SCAN PROFILING SONAR

The Mesotech Model 952 Bottom Scan Profiling Sonar is a system which, from a single location records a profile of water depths along a particular line of bearing. By manually changing the orientation of the device, profiles along several lines of bearing can be obtained to provide detailed information of bottom contours.

The Model 952 is particularly useful in harbour and canal bathymetry, and during dredging operations. The ability to gather bottom profile information from a single point greatly speeds surveys and drastically reduces the amount of navigation required during a survey.

The Model 952 Sonar Head transmits a narrow beam, high frequency acoustic pulse. The narrow beam is swept through a verticle plane, and the time for each pulse return is recorded, allowing calculation of the slant range to the bottom. The bottom profile is calculated from the slant range and angle data.

The complete system consists of the Sonar Head, which is mounted underwater, and the Recorder Case. They are connected by a 100 foot cable.

The Sonar Head contains the acoustic transducer, the stepping motor which sweeps the transducer, an inclinometer which senses rolling, the sonar transmitter and sonar receiver.

The Recorder Case contains a microprocessor control system which reads the panel switches, drives the transducer stepping motor, controls the sonar transmitter and receiver, reads the inclinometer, stores the data, and records the data on an X-Y plotter.

The Model 952 can also operate as a precision recording depth sounder when the transducer is locked in the straight down position.

Specifications subject to change without notice.

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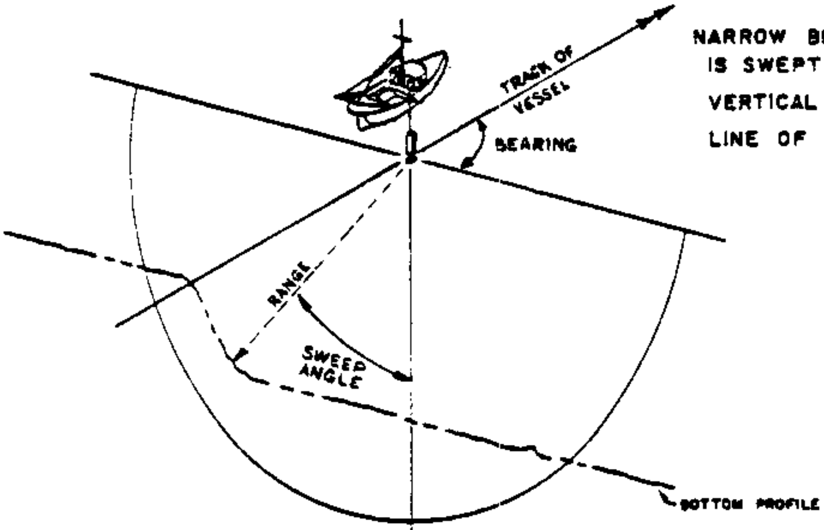
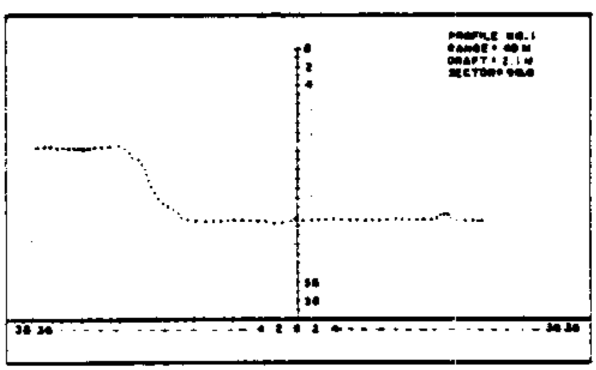
MODEL 952
BOTTOM SCAN PROFILING SONAR
METHOD OF OPERATION



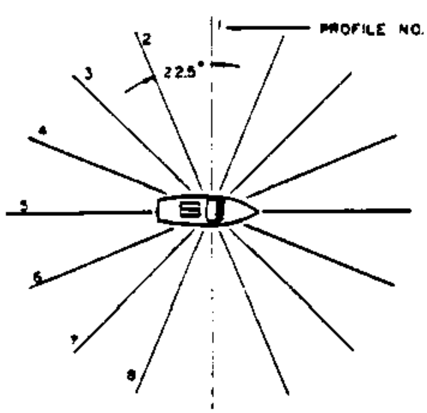
NARROW BEAM TRANSDUCER IS SWEPT THROUGH A VERTICAL PLANE ALONG A LINE OF BEARING

FIGURE 1



BOTTOM PROFILE ALONG THIS LINE IS RECORDED ON THE X-Y PLOTTER

FIGURE 2



PROFILES ARE RECORDED ALONG SEVERAL LINES OF BEARING TO MEASURE BOTTOM CONTOURS

FIGURE 3

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SPECIFICATIONS - MODEL 952

OPERATING FREQUENCY:	360 kHz
BEAM WIDTH:	1.5°
RANGE SCALES:	20, 40, 80, 160 m
MINIMUM RANGE:	1.5 m
RANGE RESOLUTION:	±0.09 m
RANGE ACCURACY:	±0.5%
SWEEP ANGLES (From Vertical):	±22.5°, ±45°, 67.5°, ±90°
SWEEP OFFSETS: (2 Axis only):	-60°, -30°, 0°, +30°, +60°
ANGLE RESOLUTION:	±0.75°
BEARING ANGLE (2 Axis only):	±90° selected in 1° steps
SWEEP TIMES: Minimum:	2.5 s (±22.5°, 20 m)
Maximum:	44 s (±90°, 160 m)
SERIAL INTERFACE:	20 mA Current Loop 9600 Baud, Full Duplex
POWER REQUIREMENTS: Sonar	115/230*V, 60 Hz, 500 VA
: Plotter	115/230 V, 60 Hz, 130 VA
SIZES: Recorder Case:	840 mm wide x 610 mm deep x 380 mm high (33"x24"x15")
1 Axis Sonar Head in Frame:	248 mm x 298 mm x 1005 mm long (9.75"x11.75"x39.6")
2 Axis Sonar Head in Frame:	increase length to 1320 mm (52.2")
WEIGHT: Recorder Case (full):	46 kg (100 lbs.)
1 Axis Sonar Head in Frame:	32 kg (70 lbs.)
2 Axis Sonar Head in Frame:	40 kg (88 lbs.)
Head/Proc. Cable, 30 m (100 ft.):	26 kg (57 lbs.)

*For 230 V operation of sonar, specify at time of order

Specifications subject to change without notice.

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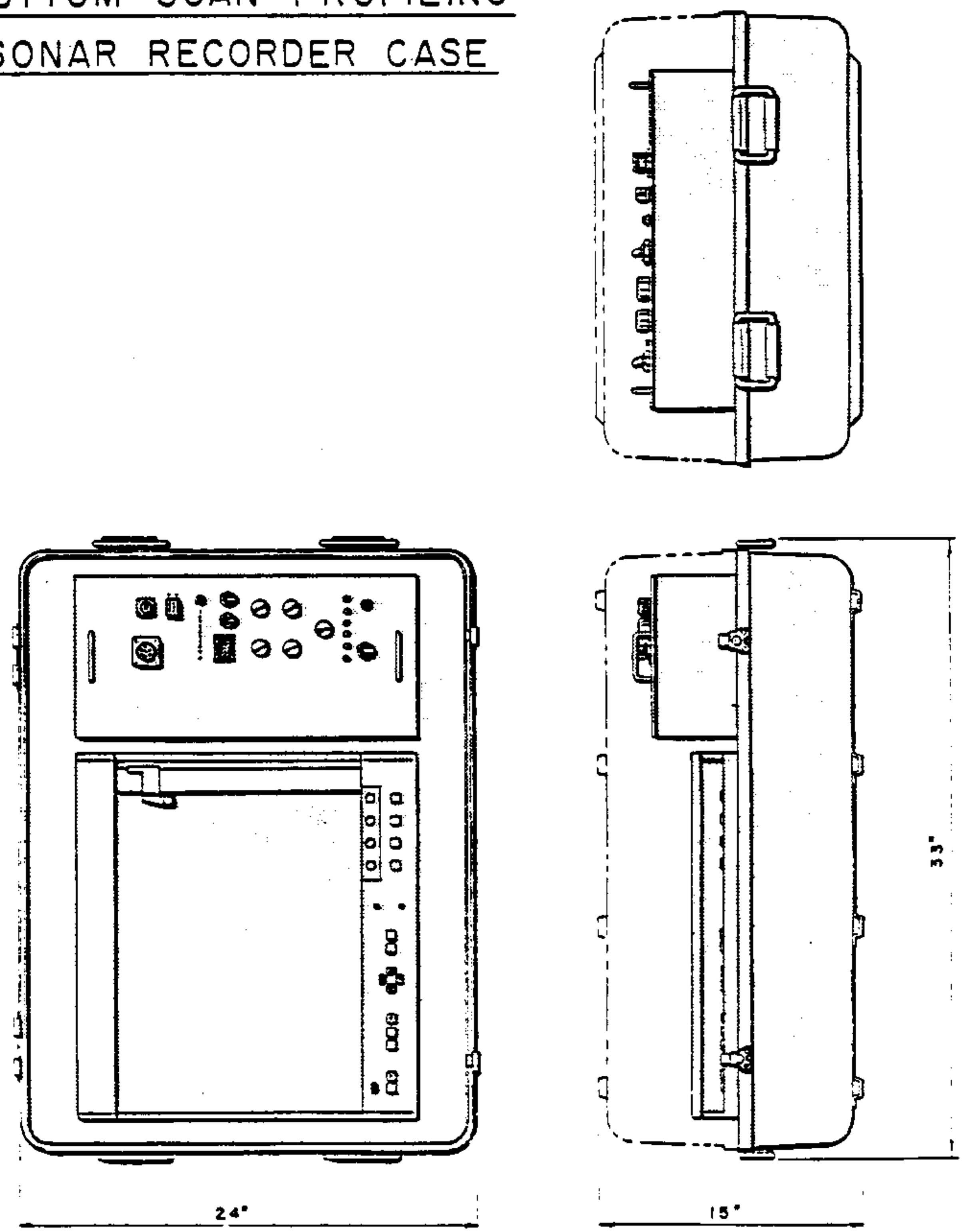
MODEL 952
BOTTOM SCAN PROFILING
SONAR RECORDER CASE



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MODEL 952 DUAL AXIS BOTTOM SCAN PROFILING SONAR

The Dual Axis version of the 952 Profiling Sonar is similar in operation to the Single Axis version. However, under processor control, using the integrated Bearing Drive, the Sonar Head can be oriented to a new bearing. The Transducer Bearing can be manually selected by operating the appropriate controls, or processor selected according to a programmable bearing step angle.

The Dual Axis Model is capable of recording up to 8 separate profiles. The stored records can be plotted separately, in an isometric format, or in hydrographic chart format enabling a contour map to be drawn.

The Dual Axis Sonar can plot these records on an HP 9872 X-Y Plotter, on a CRT Monitor (or electrosensitive graphics plotter to be interfaced with the monitor) or electrosensitive plotter, (A Model 1800 CRT Printer Driver is required).

In the continuous sweep mode the CRT constantly displays the new profile when a search is being conducted.

Specifications subject to change without notice.

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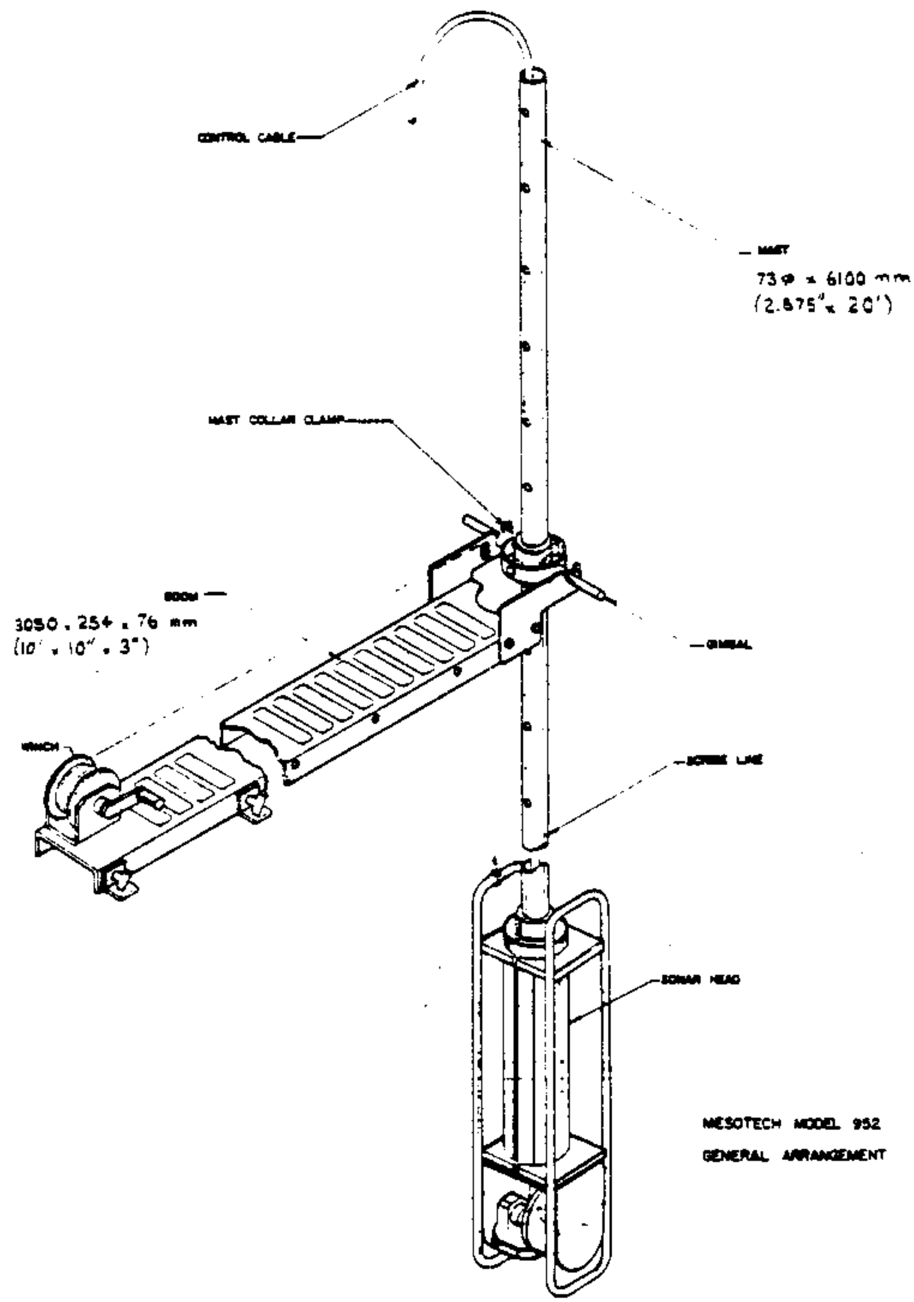

MESOTECH MODEL 952
GENERAL ARRANGEMENT

MESOTECH SYSTEMS LTD.
74 WELCH STREET, NORTH VANCOUVER, B.C. CANADA V7P 1B2
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MODEL 961 SONAR

The MESOTECH MODEL 961 SONAR is a precision surveying sonar system designed specifically for use on remotely operated submersible vehicles. It should not be confused with "obstacle avoidance" type sonars. The Model 961 uses a narrow, conical, high frequency sonar beam to make accurate measurements of underwater shapes, up to maximum range of 40 meters. The display and permanent records are clear, unambiguous, easy to interpret and in true scale. A real time, CRT display allows constant monitoring of data, and data can be stored in digital form, on magnetic tape, for later analysis.

One important application is that of making a series of closely spaced profiles across a pipeline while the vehicle moves along the pipeline. These profiles will show trench depth, width and shape and indicate pipeline exposure. The sonar also assists the vehicle in finding the pipeline, or other underwater objects.

The Sonar consists of a SONAR HEAD which is mounted on the vehicle and a SONAR PROCESSOR and CRT DISPLAY located at the operator's console. Permanent records are made using a Hewlett - Packard 9825 desk calculator and a Hewlett - Packard 9872 X-Y Plotter. The use of this standard equipment allows great flexibility in recording, displaying and analyzing the data. Mesotech supplies the software required for the Hewlett - Packard equipment.

Specifications subject to change without notice.

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SPECIFICATIONS - MODEL 961

OPERATING FREQUENCY:	360 kHz
BEAM WIDTH:	1.5°
SWEEP ANGLE/TIME:	180° in 4 s (40 m range 8 s)
ANGLE RESOLUTION:	0.7° 5,10 m ranges (256 pulses) 1.4° 20,40 m ranges (128 pulses)
RANGE SCALES:	0.5 to 5, 10, 20, 40 m
DISPLAY TYPE:	CRT, 100 dots vert. x 200 dots hor.
DISPLAY RESOLUTION:	1 dot (≈10 mm per m of Range Scale)
DIGITAL INTERFACE:	RS-232-C, 9600 baud
TIMING RESOLUTION:	10 μs (equivalent to 7.5 mm range)
CABLE REQUIREMENTS:	Coaxial cable or twisted pair, maximum length 2000 m (6600 ft)
CABLE FREQUENCIES:	143 kHz (Left Transmit) 200 kHz (Right Transmit) 250 kHz (Receive)
14.3 kHz (5 m Range) 16.1 kHz (10 m Range) 23.8 kHz (20 m Range) 15.2 kHz (40 m Range)	17.2 kHz (Scan Stop) 18.5 kHz (Scan Reverse) 20.0 kHz (Gain Step) 21.7 kHz (Gain Reset)

	SONAR HEAD	PROCESSOR	DISPLAY
POWER:	28V DC, 1.5 A	115/230V, 50/60 Hz 100 VA	115/230V,50/60 Hz 50 VA
SIZE:	173 mm (6.81in)Ø 737 mm (29 in) L	483 mm (19 in) W 133 mm (5.25 in)H 356 mm (14 in) D	222 mm (8.75 in) W 230 mm (9 in) H 399 mm (15.7 in) D
WEIGHT: In water:	16 kg (35 lbs) 2.7 kg (6 lbs)	13 kg (28 lbs) --	7.7 kg (17 lbs) --
OPERATING DEPTH:	1,000 m (3300 ft)	--	--

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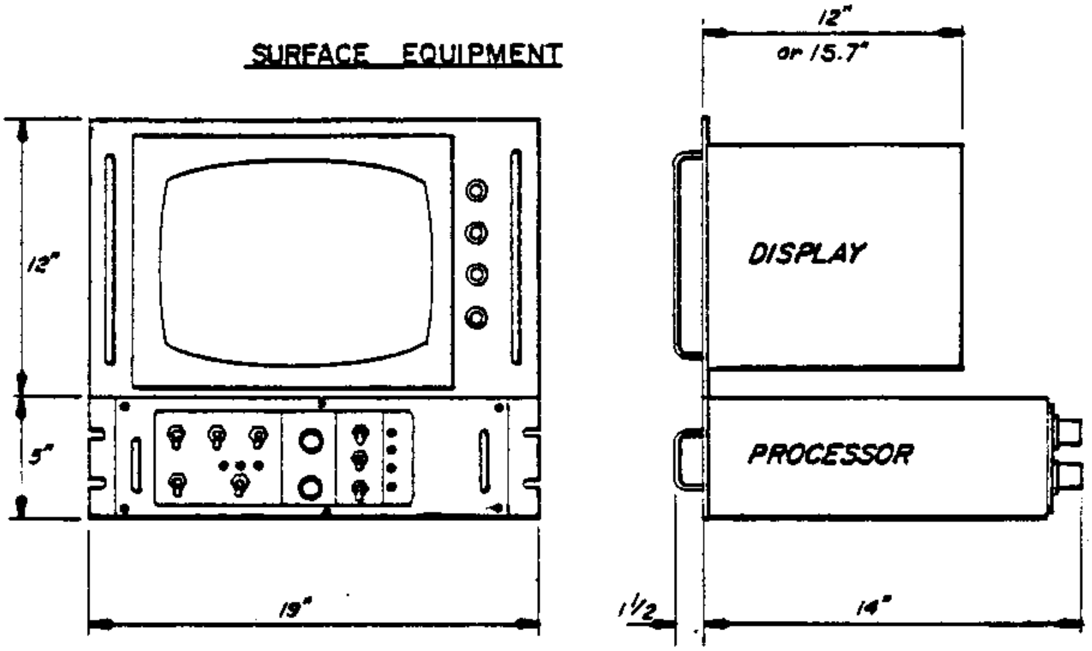
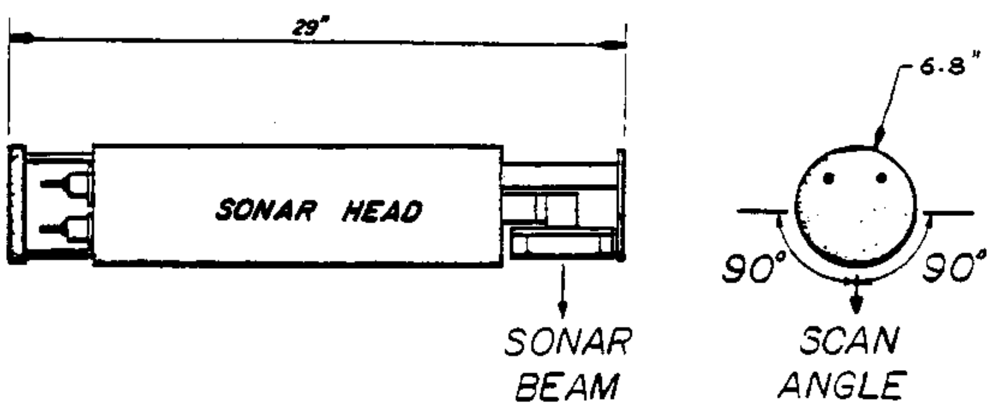
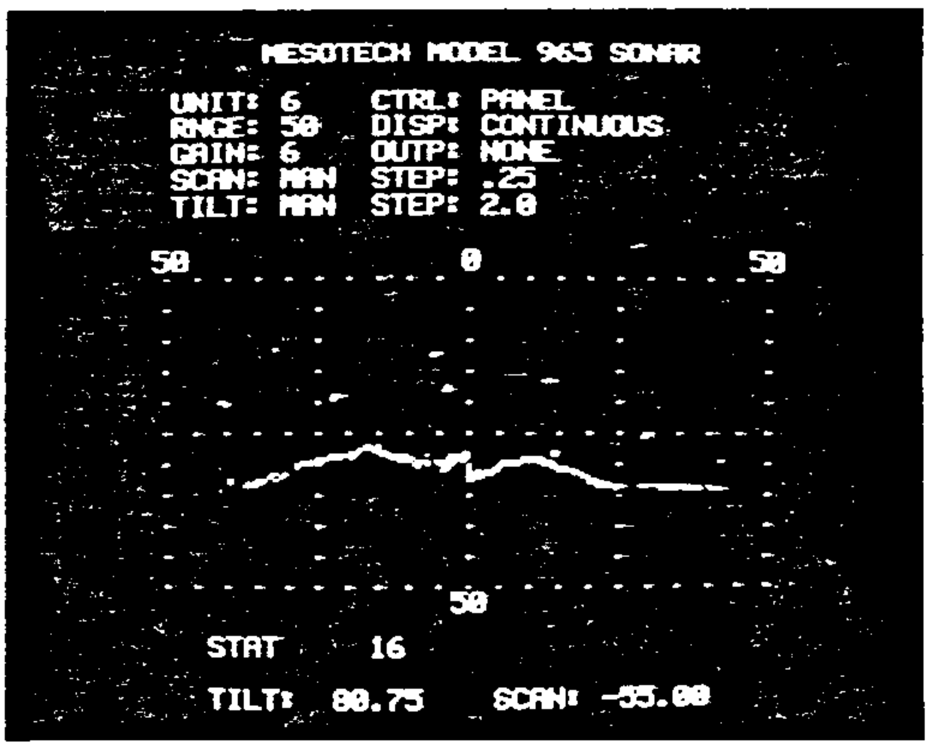
MODEL 961 PRECISION SURVEYING SONAR

SURFACE EQUIPMENT





MODEL 965
SUPER PRECISION PROFILING SONAR



MESOTECH SYSTEMS LTD.
1174 Welch St. North Vancouver, B.C.
Canada. V7P 1B2
Tel. (604) 980-3474. Telex 04-352773.

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MODEL 965 SUPER PRECISION PROFILING SONAR

The MESOTECH MODEL 965 SONAR is a super precise two axis profiling sonar, effectively an Underwater Theodolite.

It is used whenever the most accurate images of underwater terrain or objects are required. When excavating difficult material like rock, accurate monitoring and inspection can show considerable cost savings. When precise positioning is required for anti-scour mats, templates, stacks, pipelines, etc., or for civil engineering works, the Model 965 is fast and efficient.

The sonar comprises: a processor and a standard video display monitor on board, and a sonar head underwater.

The rack mountable processor contains: all the controls needed to operate the sonar, the video display generator and the interface to the external computer. Its microprocessor control offers the potential of custom programming. The external computer provides the fullest use of the generated data, e.g., three dimensional plots, and can control all but three of the front panel controls.

The pressure sealed sonar head is constructed of hard anodized aluminum alloy, epoxy coated and zinc anode protected. It contains the microprocessor based electronics for the transmitter, receiver and stepping drive motors. The highly directional transducer scans in 2 axes, "sweep" and "tilt", by means of modular drive assemblies.

Any arc may be selected in sweep or tilt from -90° to $+90^\circ$ (in 1° increments) and scanned automatically; 0° for each axis is vertically down. The transducer may also be directed manually or via the external computer. As the transducer scans, short pulses are transmitted which reflect off the surface being imaged. The time of each returning pulse is converted to a slant range, corrected for the angle of sweep and displayed as the true depth at the true range.

Time Varying Gain, Angle Compensated Gain, and manual gain controls ensure the best possible signal to noise ratio and hence displayed image. Each time the 965 is powered up, it performs a self test; tests may also be made in air without damaging the equipment.

Specifications subject to change without notice.

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SPECIFICATIONS - MODEL 965

OPERATING FREQUENCY:	750 kHz
BEAM WIDTH:	0.67°
RANGE SCALES:	25, 50 m
SWEEP ANGLE: (Limits set independently)	-90° to $+90^\circ$ in 1° steps Upper and lower limits
STEP SIZE:	0.25°, 0.50°
ANGULAR RESOLUTION:	0.1°
TILT ANGLE: (Limits set independently)	-90° to $+90^\circ$ in 1° steps Upper and lower limits
STEP SIZE:	0.25°, 1.00°, 2.00°
ANGULAR RESOLUTION:	0.1°
TIMING ACCURACY:	30 microseconds
TIMING RESOLUTION:	9.3 microseconds
DIGITAL OUTPUT:	RS232C or 20 mA current loop
BAUD RATE:	9600
CABLE REQUIREMENTS: (Sonar Head to Processor)	6 conductors, 2xNo.12 A.W.G., 4xNo.20 A.W.G., 200 m maximum
POWER REQUIREMENTS:	110/220 V, 50/60 Hz, 300 W
SIZE: PROCESSOR:	483 x 406 x 178 mm (19"W x 16"D x 7"H)
SONAR HEAD:	495 mm dia. x 1016 mm high (19.5" dia x 40" high)
WEIGHT: PROCESSOR:	21 kg (45 lbs) approx.
SONAR HEAD:	82 kg (180 lbs)

Specifications subject to change without notice.

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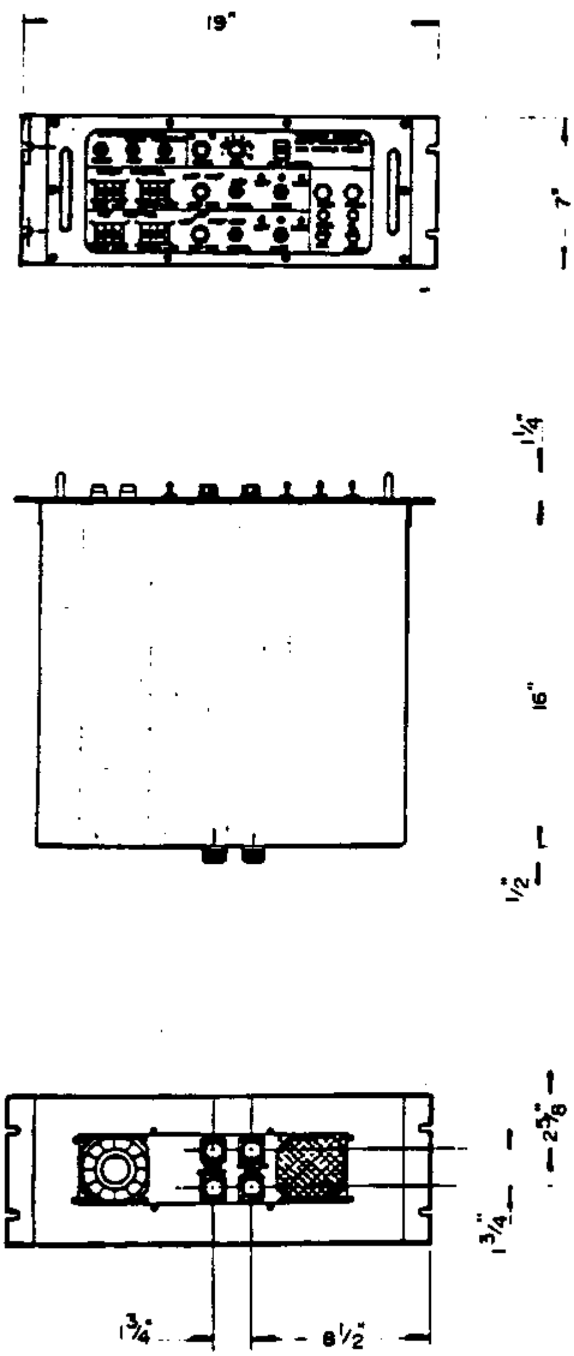
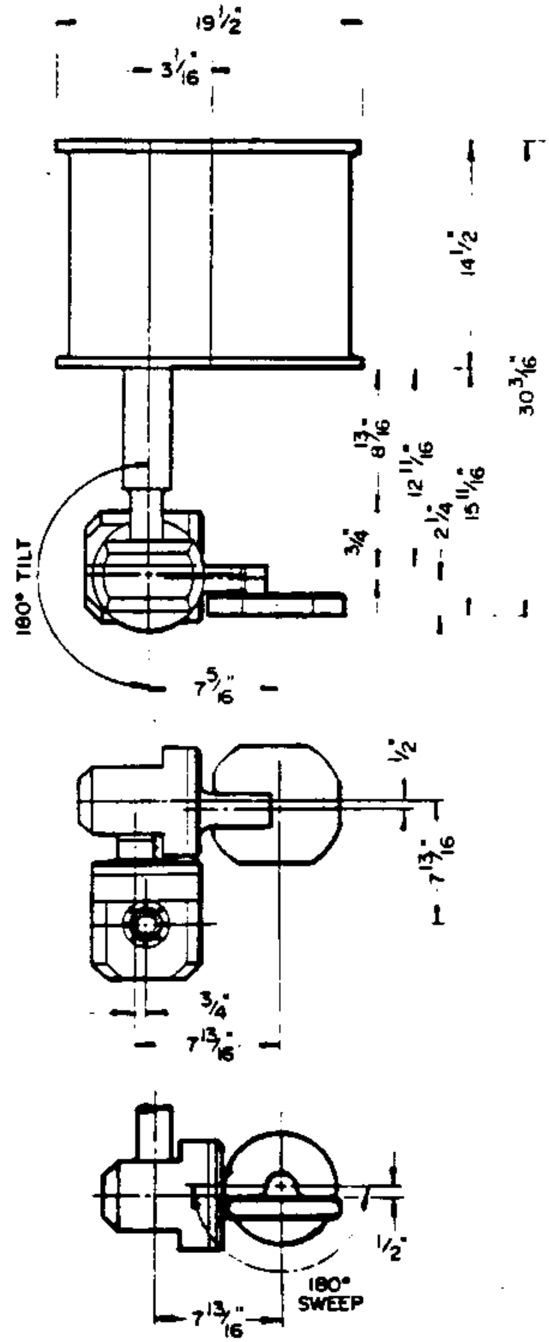
965 SUPER PRECISION PROFILING SONAR

SURFACE EQUIPMENT



SONAR HEAD

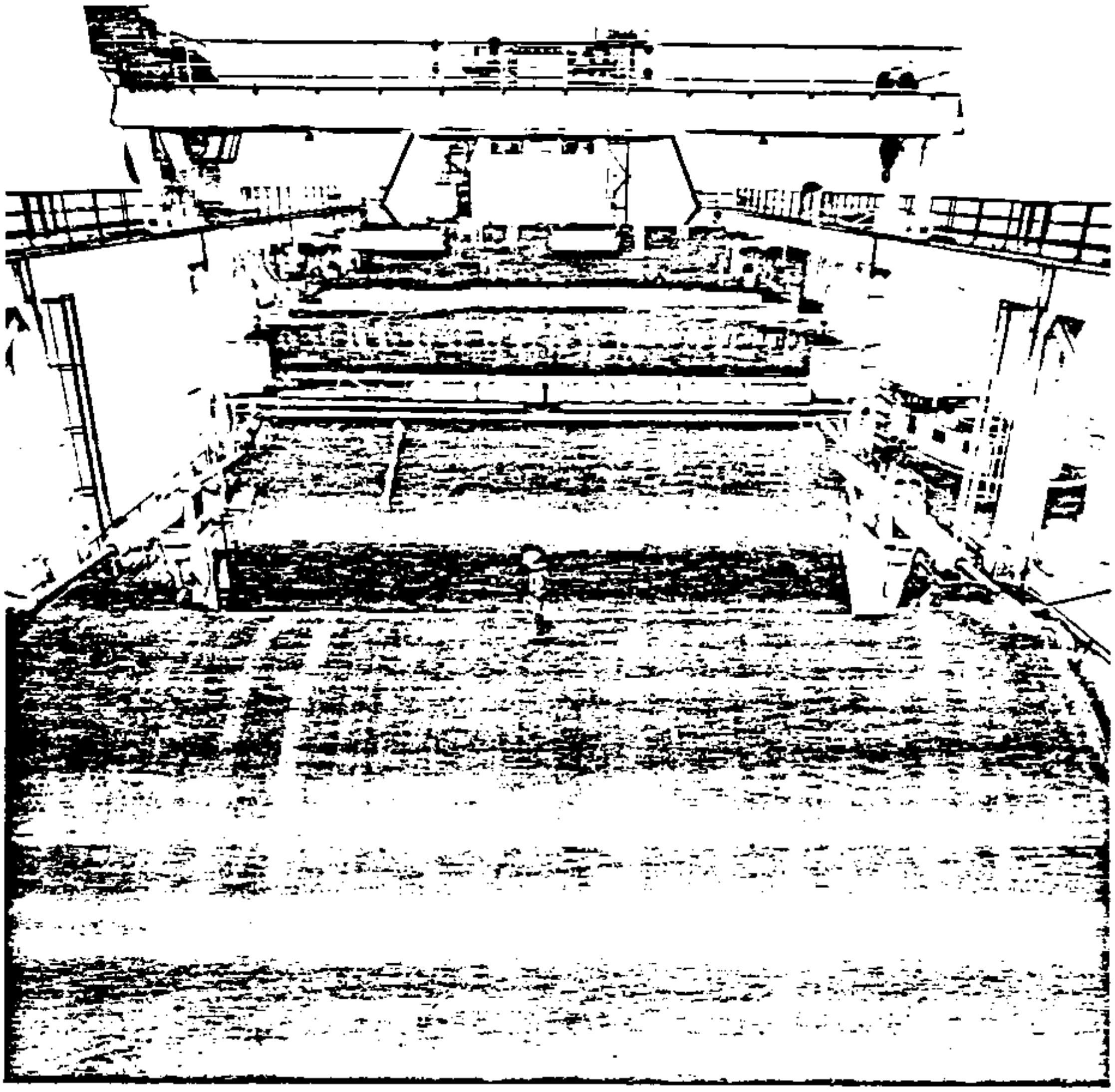


APPENDIX I

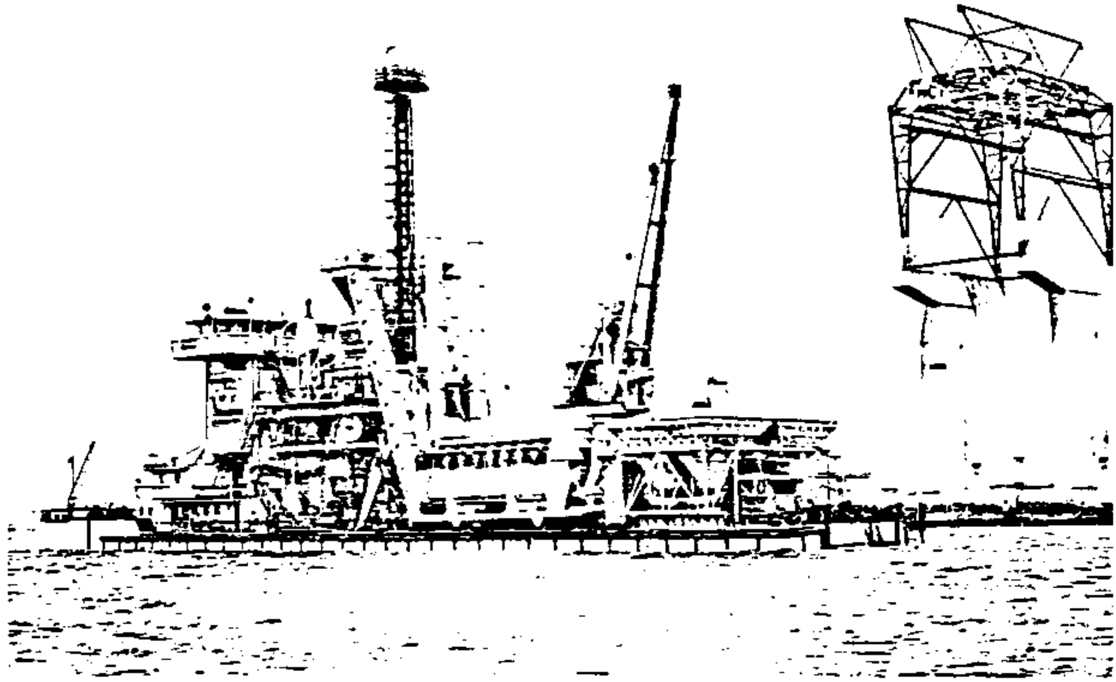
APPENDIX I

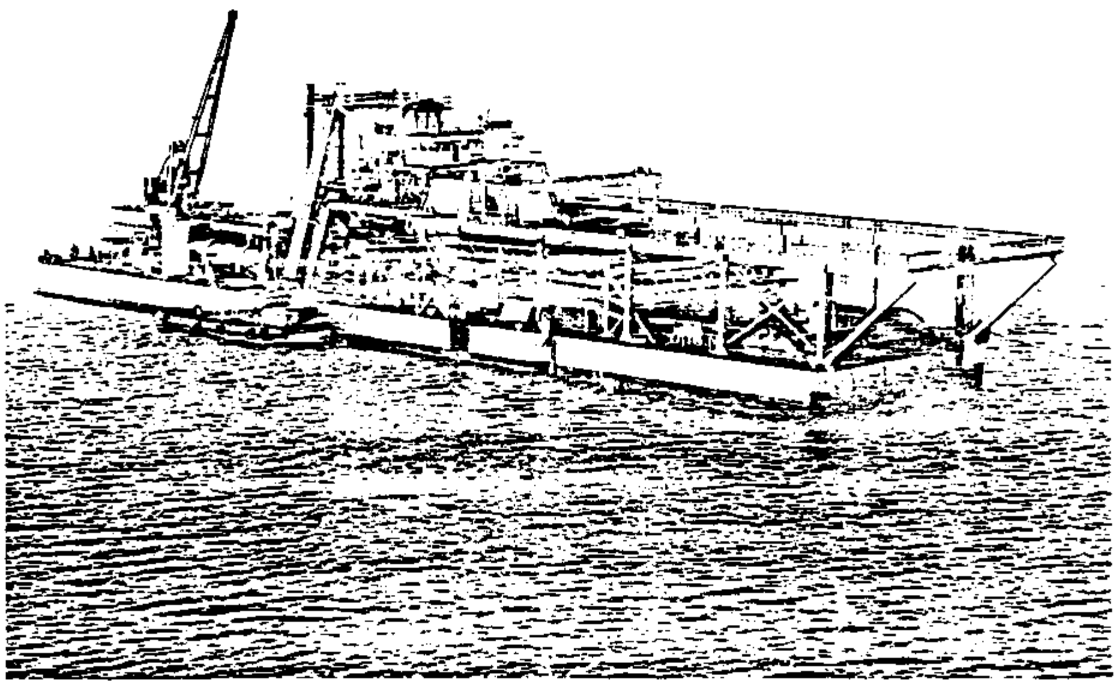
PREFABRICATED MATTRESSES WITH MASTICMAC

Appendix I illustrates the application of Masticmac mattresses with the converted "Jan Heijmans". It includes photocopies of photographs of this equipment as well as a schematic illustration of the whole operation.

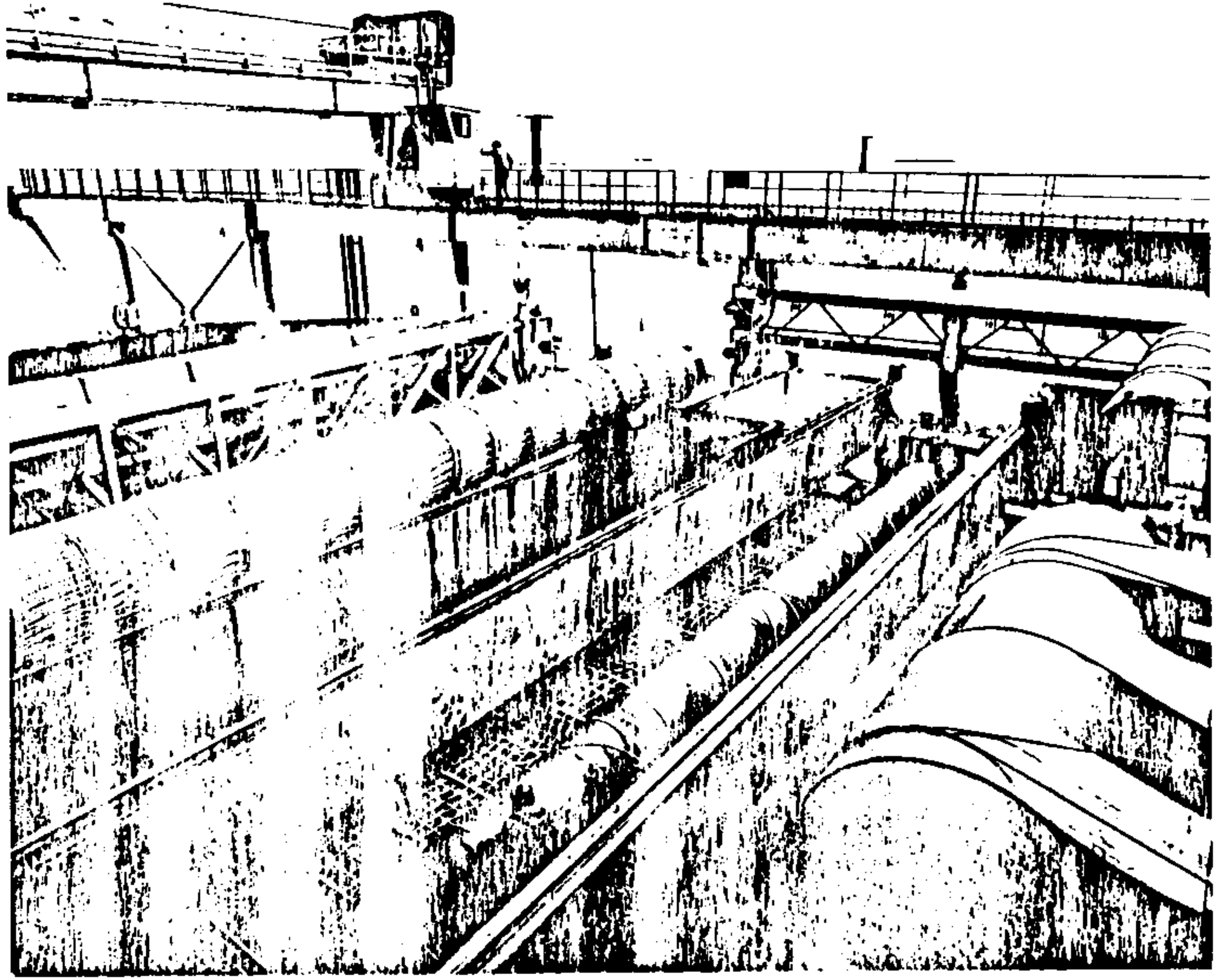


FRONTAL VIEW OF THE CONVERTED 'JAN HEIJMANS'. THE MASTICMAC MATTRESS IS BEING WOUND UP ONTO THE LARGE CYLINDER IN FRONT

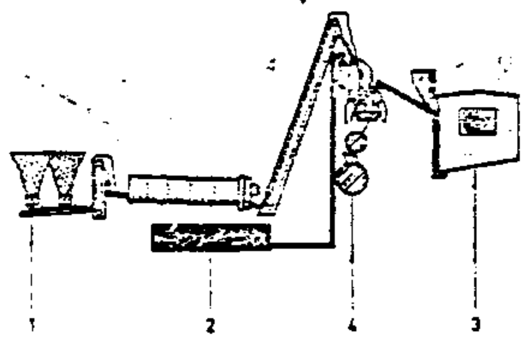




THE ORIGINAL 'JAN HEIJMANS', READY TO APPLY A SANDMASTIC CARPET IN A CLOSURE GAP (1969), AND THE SAME ASPHALTSHIP, CONVERTED TO LAY LARGE PREFABRICATED MATTRESSES WITH MASTICMAC (1974)

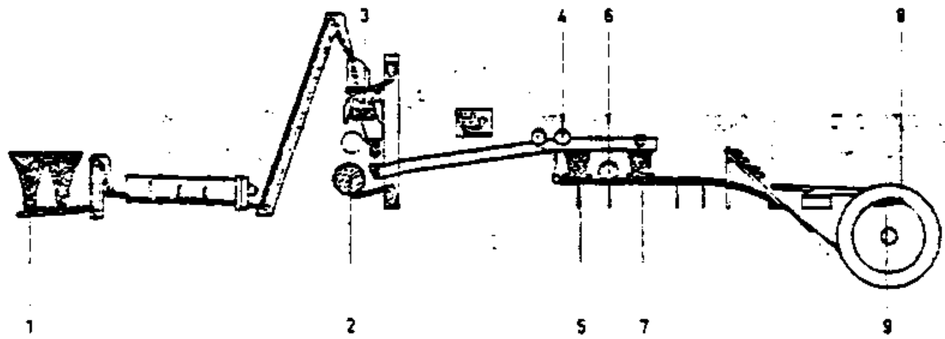


VIEW OF THE PRODUCTION PLATFORM DURING MANUFACTURE OF THE MASTICMAC MATTRESSES in lower 50 tons stirring hopper



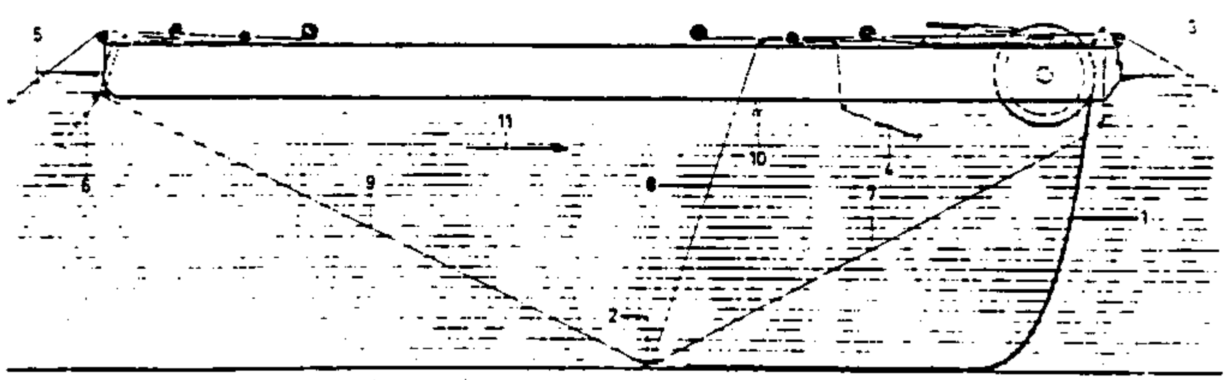
1 sand
2 bitumen
3 filler
4 sandmastic

Stage 2 Mixing of sandmastic with heated stones, transport of masticmac mix to the front, manufacture and rolling up of mattress



1 stone
2 sandmastic
3 masticmac
4 filter fabric
5 first course masticmac
6 reinforcing wire mesh
7 second course masticmac
8 edge loading
9 tailbeam

Stage 3 Unrolling and laying of masticmac mattress during neap tide



1 masticmac mattress
2 sink beam
3 bower wire
4 frontal side wires
5 stern wire
6 rear side wires
7 hoisting cables sink beam
8 guiding wires sink beam
9 rear cables sink beam
10 profiler
11 direction of laying

Fig 30 Application of masticmac mattresses with the converted "Jan Heijmans"