No. 01

BASIC DESIGN STUDY REPORT
ON
THE PROJECT FOR IMPROVEMENT OF FACILITIES FOR REPAIR AND MAINTENANCE OF FISHING VESSELS
IN
THE REPUBLIC OF MOZAMBIQUE

February 1998

Japan International Cooperation Agency
Fisheries Engineering Co., Ltd.

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PREFACE

In response to a request from the Government of the Republic of Mozambique, the Government of Japan decided to conduct a basic design study on the Project for Improvement of Facilities for Repair and Maintenance of Fishing Vessels and entrusted the study to the Japan International Cooperation Agency (JICA).

JICA sent to Mozambique a study team from December 6 to December 20, 1997.

The team held discussions with the officials concerned of the Government of Mozambique, and conducted a field study at the study area. After the team returned to Japan, further studies were made, and as this result, the present report was finalized.

I hope that this report will contribute to the promotion of the project and to the enhancement of friendly relations between our two countries.

I wish to express my sincere appreciation to the officials concerned of the Government of the Republic of Mozambique for their close cooperation extended to the team.

February, 1998

Kimio Fujita
President
Japan International Cooperation Agency

February, 1998

LETTER OF TRANSMITTAL

We are pleased to submit to you the basic design study report on the Project for Improvement of Facilities for Repair and Maintenance of Fishing Vessels in the Republic of Mozambique.

This study was conducted by Fisheries Engineering Co., Ltd. under a contract to JICA, during the period from November 21, 1997 to February 20, 1998. In conducting the study, we have examined the feasibility and rationale of the project with due consideration to the present situation of Mozambique and formulated the most appropriate basic design for the project under Japan's grant aid scheme.

Finally, we hope that this report will contribute to further promotion of the project.

Very truly yours,

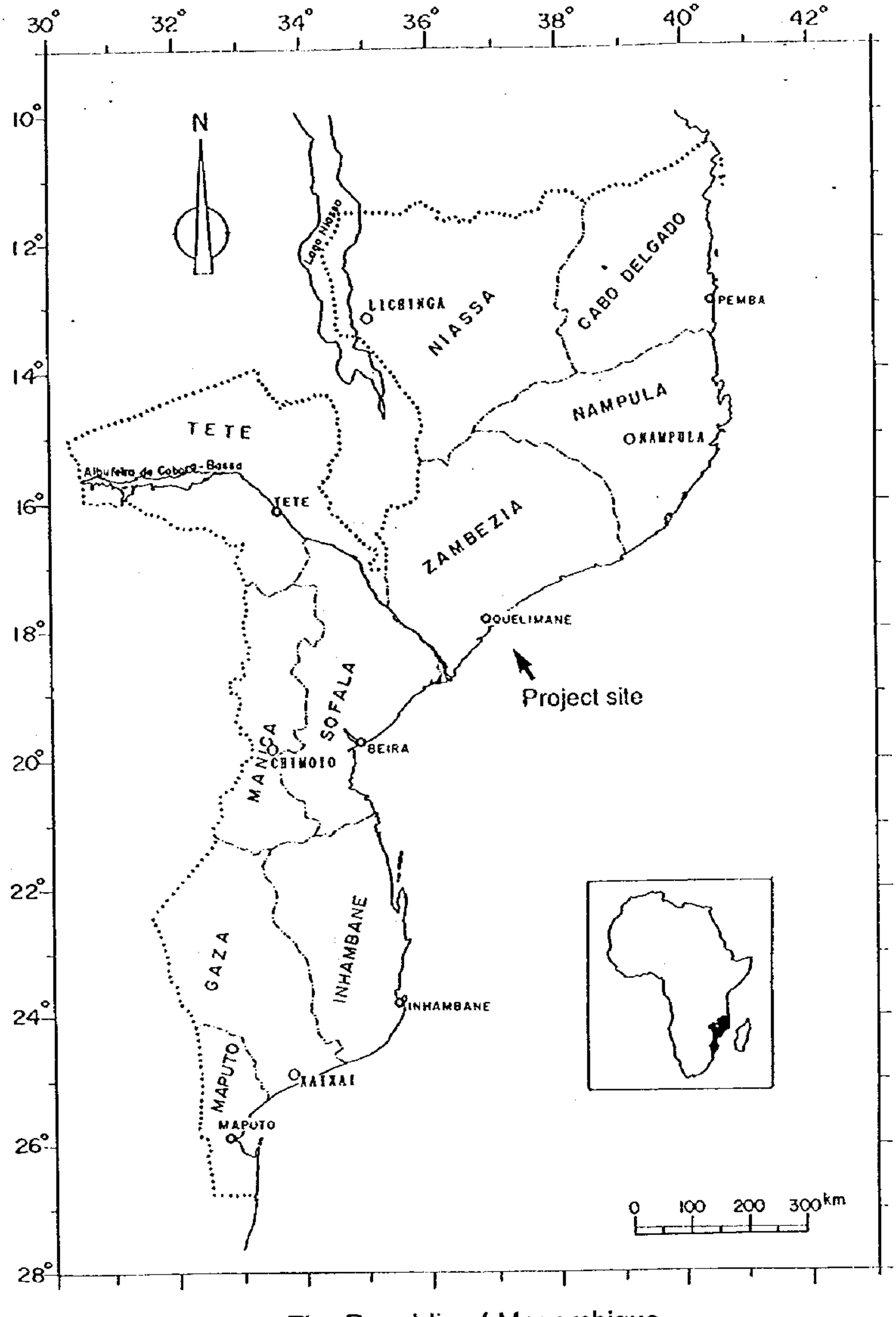
K. Watanabe

Kunihiro WATANABE

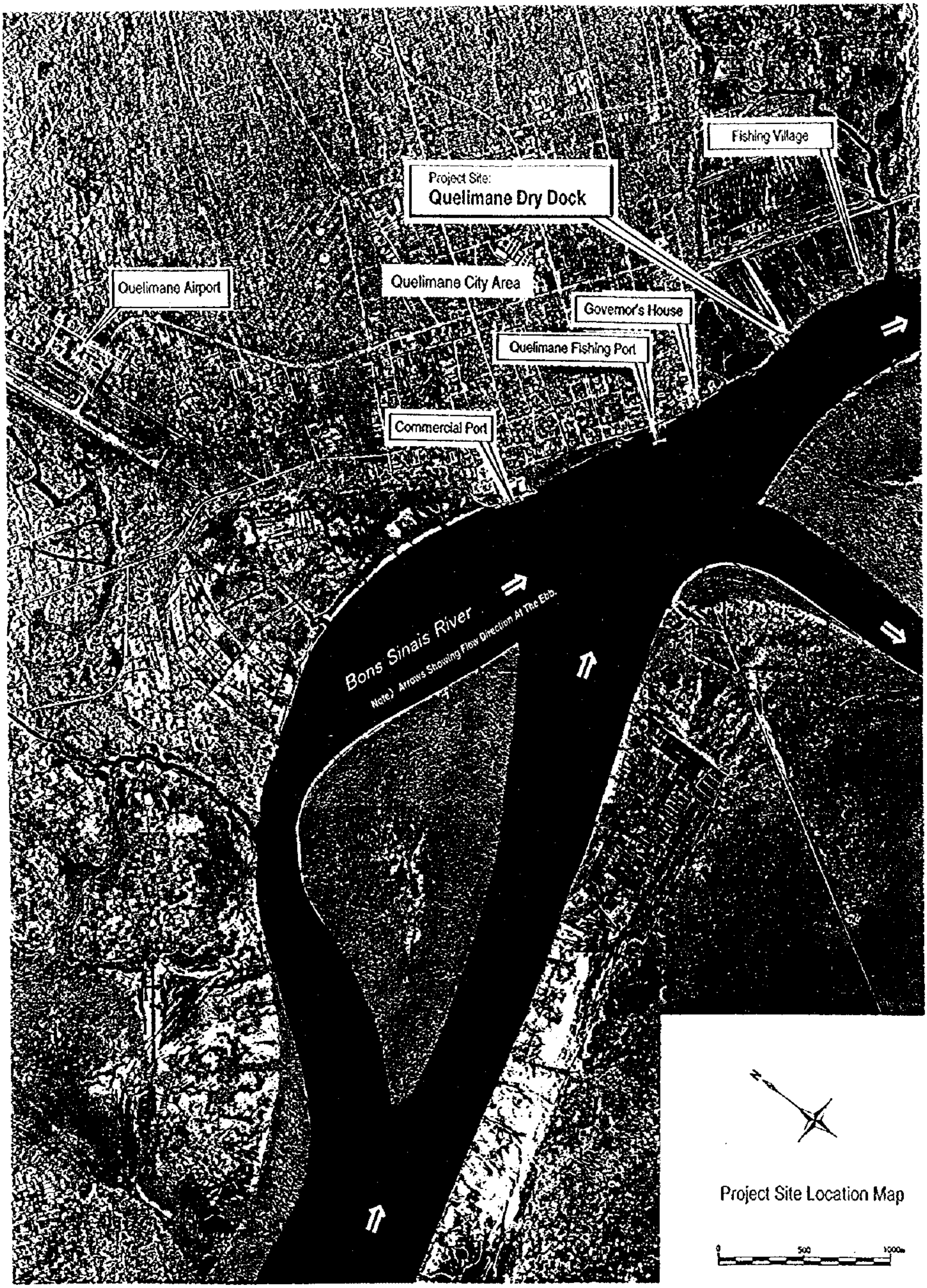
Project Manager,
Basic design study team on the Project for Improvement of Facilities for Repair and Maintenance of Fishing Vessels

Fisheries Engineering Co., Ltd.

Location Map



The Republic of Mozambique



Project Site Location Map

Contents

Chapter 1 Background of the Project

The total length of the Mozambican coastline is 2,740 km, while the area of the continental shelf in the country's waters, facing the Indian Ocean, totals about 100,000 km^2. Thanks to the presence of the Mozambique Current, which branches southward from the South Equatorial Current, a number of rivers, such as the Zambezi, which deliver nutrient salts originating on land, and the Sofala Bank, which ranges 200 nautical miles in a north-south direction, coastal waters provide an optimum environment for the reproduction of shrimp and other fish stocks, encouraging the active development of these fishery resources.

In 1995, shrimp accounted for some 43% of the total value of exports, with the balance comprising such traditional export products as cashew nuts, cotton, sugar, and wood. The fishing industry, therefore, while shouldering the responsibility for the nation's supply of animal proteins, also plays a vital role in earning foreign exchange.

While the great bulk of the shrimp catch is taken by shrimp trawlers, the Ministry of Agriculture and Fisheries in the Mozambican Government has adopted a firm policy of controlling catch effort by large fishing vessels, setting the annual TAC (Total Allowable Catch) of shallow-water shrimp by large shrimp trawlers at 6,500 tons, while also establishing a closure period for this fishery of 60 days a year, starting January 1.

At the same time, the government has been engaged in an effort to expand fish landing and repair facilities to ensure the continuance of safe fishing operations by efficient fishing vessels. However, the only repair facilities capable of conducting the legally mandated annual vessel inspections and repairs on these vessels had been located at Maputo, in the southernmost part of Mozambique, and at Beira, about 700 km north of Maputo. The authorities then had to contend with the problem that, in terms of the country's long coastline, these existing facilities were disproportionately weighted to the south.

The Mozambican Government , therefore, drew up a plan to build a new fishing vessel repair yard at Quelimane, situated in the center of the country, close to the main fishing grounds. This project was implemented under a cooperative grant-aid from Japan and completed in December, 1994 as the Quelimane Dry Dock. Operations have developed smoothly at this facility, reflecting its superior operating capabilities as a drydock dedicated to fishing vessels as well its sound operating system, based on a special staff of engineers capable of operating heavy machinery and the dispatch of a dock master as a specialist from Japan.

However, owing to erosion in the Bons Sinais River, which is assumed to have been accelerated by concentrated heavy rains in February, 1997, the structure in front of the Dry Dock has been subjected to serious scouring, as a result of which the steel sheet pile wall of the front revetment as well as the foundations of the lower portion of the concrete slab in front of the dock gate washed away, forcing a suspension of operations at the Quelimane Dry Dock in August, 1997.

Under the present situation, the Mozambican Government is concerned over the possibility of flapping out in the base portions of the steel sheet pile wall in the revetment in front of the facility or circular slip of this front revetment. In order to urgently refill the front side of the facility so as to insure structural safety while maintaining safe and efficient fishing operations, thereby sustaining economic activity in the key fishery sector, the Mozambican Government has drawn up a Plan for maintaining the Quelimane Fishing Boat Repair Facility [hereafter called the "subject Plan"], designed to restore the functions of the Quelimane Dry Dock, and has requested a grant-aid for this purpose from the Government of Japan.

Chapter 2 PROJECT CONTENTS

2-1. Project Objective

As of 1997, a total of 153 large and medium-size fishing vessels held fishing permits, with landings by these vessels in the order of 20,000 ~ 23,000 tons per year. Some 50 ~ 55% of these landings are composed of shrimp, with almost the entire catch exported, making this product the country's largest source of foreign exchange. Mozambique presently has three repair facilities to perform the yearly required inspections and repairs on fishing vessels: one at Maputo, the capital, at the southern tip of the country; a second at Beira, about 700 km north of Maputo; and a third at Quelimane, 1,100 km further north. However, the Quelimane Dry Dock, which constitutes the target facility for this project, was compelled to suspend operations in August, 1997 as a result of scouring in the lower sections of the structure induced by the progressive erosion by the Bons Sinais River.

Quelimane is located about midway up the Mozambique coast and, since its offshore waters are a key fishing ground for shallow-water shrimp and other species, many fishing vessels are based at this port. The Quelimane Dry Dock, designed exclusively to service fishing boats, was completed in December, 1994 under a cooperative grant-aid from Japan. As this is a brand-new and highly efficient dry-dock-facility, it has been serving as a core facility for Mozambique fishing vessels based at points to the north of Quelimane.

It is feared that continued interruption of operations at this repair dock will have grave repercussions on the operating efficiency of fishing vessels centered around Quelimane, resulting in a decline in economic activity within the fishery sector, which contributes so importantly to the area's economy.

The subject Plan, therefore, is designed to restore the functional capabilities of the Quelimane Dry Dock, which serves as a key facility for insuring safe and efficient fishing operations, primarily shrimp trawlers, which play a major role in the national economy through the foreign currency they earn. The restorative work will involve the construction of a protective structure to protect the front and both sides of the dock from any future erosion, thereby preventing the collapse of the revetment in front of the dock.

2.2. Basic Concept of the Project

The basic scheme of this Plan is summarized in Table 2.2.-1.

Table 2.2.-1 The basic scheme of the Plan

Objectives	As Requested	Under the Subject Plan	Plan Contents
Avoiding collapse of the structure	Emergency refill of the front section	Dealing with the problem on a permanent basis	None
Protection of the front revetment	Embankment of a mound up to -2.0 m and setting of erosion protection wall.	As per the Request	Filling with sand and covering the surface with armor stone layer
Protection of both sides	Setting erosion protection wall by driving steel pipe pile	As per the Request	Enclosing the dock structure with an erosion protection wall, using steel pipe pile over a total length of about 230m
Preventing local scouring	Not requested	Through the use of gabion baskets	Using gabion baskets to cover the front of the erosion protection wall
Countermeasures against future erosion	Not requested	Donation of monitoring equipment	Recording periodic changes in river configuration by means of a work boat and an echo sounder

The basic concepts governing the subject Plan will be as follows.

(1) The original dock functions are to be restored by protecting the dock structure at the front revetment and elsewhere, where there is a danger of collapse based on scouring in front of the dock. Since further river-bank erosion is expected, an erosion protection wall will be built around both sides of the dock.

(2) With regard to the erosion protection wall in front of the dock, allowing for the possibility that riverbed depth has fallen to -14 m, which is the present maximum depth of the riverbed, if the steel pipe piles are driven down to a depth of -21.5 m, the structure should be capable of withstanding future erosion.

(3) The mound height will be set at -2.0m, about equal to the floor slab level in front of the existing dock gate section. Operational safety will be assured by leveling the mound surface to avoid a turbulent flow.

(4) With regard to the dock sides, assuming that the average erosion speed, as obtained from an analysis of existing data, is expected to continue for another 25 years, the erosion protection wall will be built up to a range of 50 m from the existing front revetment.

(5) While it is predicted that riverbank erosion at the Project site will advance further in the future, it is difficult, under present conditions, to make an accurate determination of the state of future riverbank erosion. It will, accordingly, be necessary to make it possible to continually monitor the progress of erosion, primarily by understanding scouring conditions on the riverbed, and, when erosion develops in excess of the forecasts, take the required countermeasures. Since the river administration is uncertain about taking on such operations, we plan to provide monitoring equipment to facilitate the observation and recording of erosion by the Quelimane Dry Dock itself.

In the following section, we will discuss in detail the basic concepts outlined above.

2.3 State and Mechanism of Riverbank Erosion

2.3.1 State of Riverbank Erosion

(1) Description of the River

The Project site is located in the delta section of the lower reaches of the Bons Sinais River, about 23 km upstream from its estuary. As the river gradient is gradual, there is basic sedimentation, with the river developing a meandering course as well as shoals. The total catchment area is approximately 20,000km^2, with vast area of marsh land included.
The characteristics of the Bons Sinais River in the Plan vicinity may be described as follows :

①Owing to its very gentle gradient, shoals develop, while the river contour is quite complex, evidencing distinct meandering conditions. Shoals develop both in front of and upstream from the Project site. Accordingly, the river branches and joins and, in the section fronting on the Project site, the southern channel branched by the shoal located upstream flows into downstream.

②About 7 km upstream from Quelimane City, the Bons Sinais River narrows, with the riverbed becoming shallow, so that there is no merchant or fishing vessel traffic upstream from the Commercial Port.

③Upstream from Quelimane City, there is a conspicuous development of a river net, creating a swampy area in the vicinity. Thus, when waters rise as a result of rainfall, the river inundates the swampy area, though Quelimane City itself seldom experiences flood damage due to overflow from the Bons Sinais River.

④The rainy season runs mainly from November to February, with annual rainfall averaging about 1,400 mm from 1982 ~ 1997.

⑤The Project site lies in a tidal reaches and so is subject to influences from tidal and flow patterns. During spring tides, the water level will vary by some 4.0 m, while the phenomenon of a reverse flow (downstream → upstream) will at times change the river's usual fair flow (upstream → downstream).

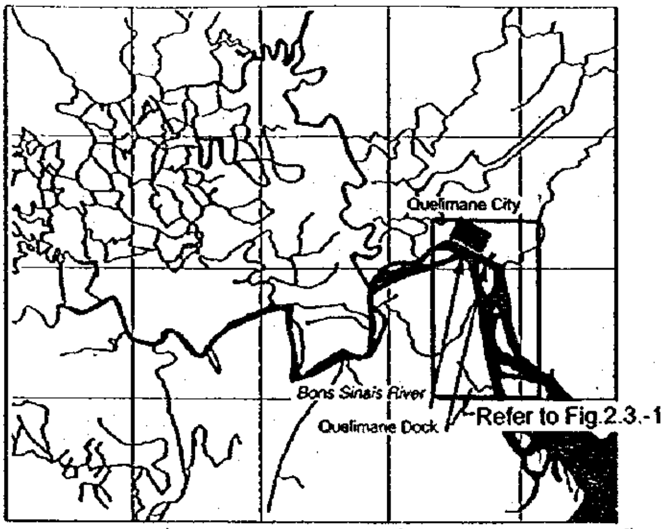


Figure 2.3.-1 River Net of the Bons Sinais River (1975)

(2) Erosion Conditions on the Riverbank

Based on findings from our field survey, it has become clear that riverbank erosion has been progressing not only in the vicinity of the dock but also over a wide area extending from the upstream section above Quelimane City down to the estuary.

Riverbank erosion is evident chiefly at river bends, with the opposite bank often evidencing sedimentation tendencies. We have concluded that this pattern is greatly influenced by the gentle gradient and meandering course of the Bons Sinais River.

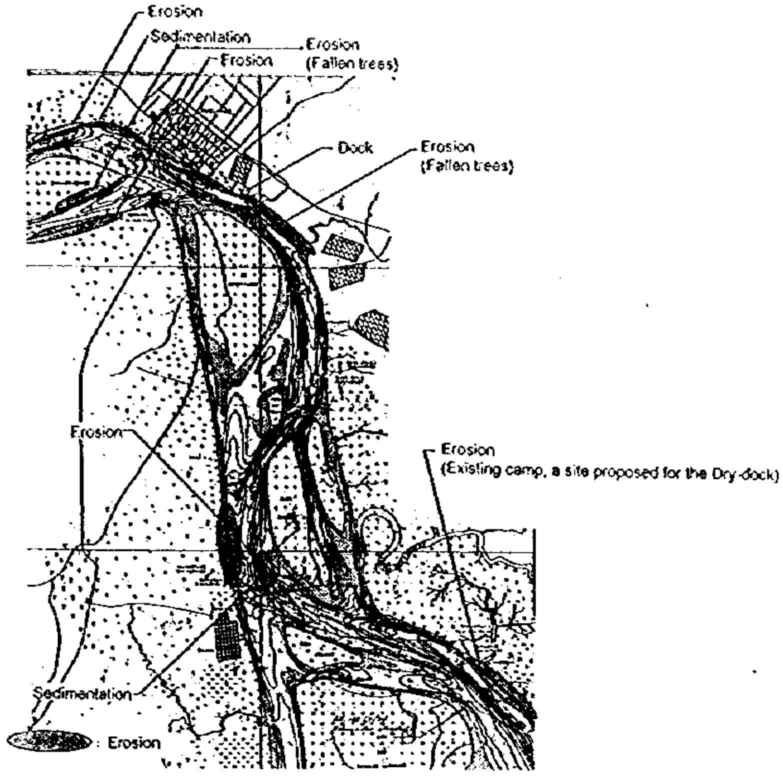


Figure 2.3.-2 Riverbank Erosion between Quelimane and Estuary

The erosion and sedimentation conditions in the vicinity of the site are shown in Figure 2.3.-3. The Quelimane Dry Dock is located at a bend of the river, and we have established that erosion has developed from a distinctive phenomenon -- viz., erosion in the bend area.

Erosion tendencies, moreover, are found on both banks of the present main channel upstream from the Project site, whereas, on the contrary, upstream from the city center along the former main channel, a sedimentation tendency is evident.

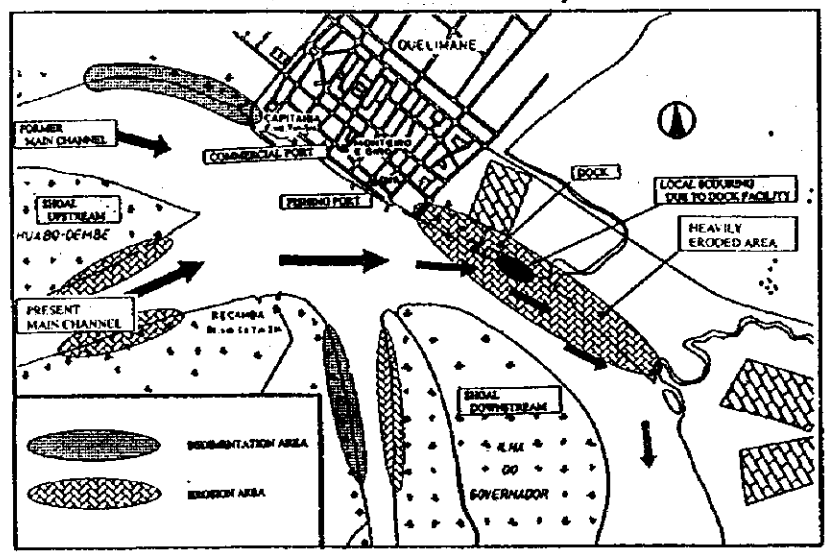


Figure 2.3.-3 State of Existing Riverbank in the Vicinity of the Project site

As is clear from the result of riverbed depth sounding, local scouring has developed in front of the dock, which may be attributed to the presence of a structure (i.e., a phenomenon identical to scouring based on piers in the river channel), as opposed to erosion on the riverbed.

(3) Advancement of Riverbed Erosion :

Figure 2.3.-4 shows the general development stages of the river meander. Erosion due to meander and attenuated locations has developed while moving these locations to the middle of the bends at the meander stage.

Riverbank erosion in the vicinity of the Quelimane Dry Dock is a phenomenon of the river bend, starting upriver from the dock, with maximum erosion found at the fishing village, and this erosion is even now advancing, including that at dock locations. This condition, similar to the general phenomenon of river meander, may be said to not only evidence progressive erosion, but also indicates that the most severely eroded locations are gradually moving to the downriver side.

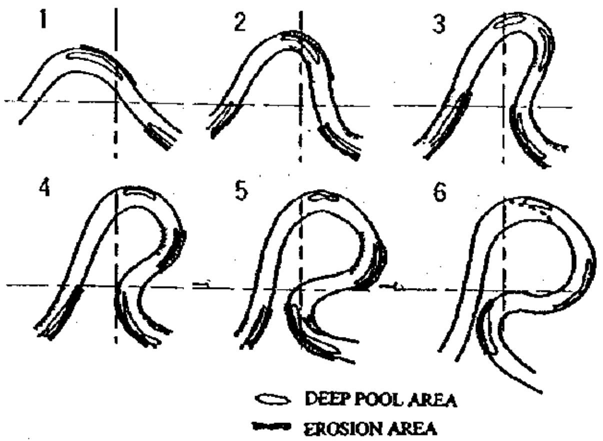


Figure 2.3.4 Developmental Stages of the Meander

2.3.2 General Causes of Riverbank Erosion

Generally speaking, the factors giving rise to deepening of the riverbed-- i.e. the development of deep pools reflect the following.

① shoals

② bends in the channel

③ changes in river width

④ small waves on the riverbed

⑤ structural elements

⑥ other factors .

In the river, these factors overlap, producing scouring, which, in turn, becomes a cause of a riverbank erosion. However, in tidal river where, as in the Quelimane area, maximum tide differentials reach 4.0 m, since the effect of 「① shoals」, and 「④ small waves on the riverbed 」 is believed to be minor, the causes may be assumed to involve the remaining factors -- i.e.., 「② bends in the channel」, 「③ changes in river width 」, and/or 「⑤ structural elements」.

Since riverbank erosion at the Project site may, as already explained, be divided between: 「riverbank erosion at bends」 and 「local scouring in front of the structure」, we shall discuss below the causes of erosion along with the erosion mechanism.

(1) Riverbank Erosion at River Bends

a) Erosion mechanism of riverbank

The Bons Sinais River is a tidal river with a complex configuration, formed by shoals, with the latter developing on the upriver side and directly in front of the dock. Based on simultaneous flow volume observations at the Project site, the main current path at present is south side in the upstream sector and dock side in the downstream.

Looking at flow direction and velocity in the waters in the vicinity of the dock, the flow direction component toward the left-bank at the bend section, where the dock is situated, is quite large. Accordingly, at the riverbank line both upriver and downriver from the dock, erosion has developed from the emergence of a secondary current based on both flow concentration and the bend in the river.

The mechanism of this erosion is based on flow concentrations at bend areas, resulting from a direct inertial flow into the area. The velocity increases, along with the transport capacity of the materials making up the riverbed, which causes scouring. In addition, since the flow line is bent at the bend area, producing a centrifugal force that causes the riverbed depth of the outer bay to rise, a pressure differential is created in a traverse direction across the river, producing a secondary flow toward the inner bay, which in turn moves the soil and sand into the inner bay, again resulting in scouring.

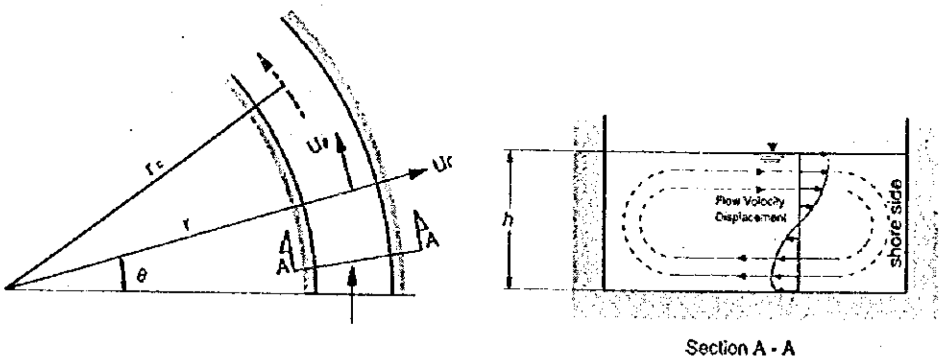


Figure 2.3.-5 Secondary Current in the Bend Channel

The factors causing this sort of flow concentration in the vicinity of the dock facility, thus accelerating the riverbank erosion, reflect the following natural conditions :

①the dock's location at a bend in the river meander.

②the large flow component toward this bend area.

③the constant influence of tidal action on fair flow and reverse flow.

④In addition, during periods of heavy rainfall, the volume of water accumulation in the river channel increases, thereby increasing velocity during fair-flow periods.

b) Development Sequence of the Riverbank Erosion :

The riverbank erosion in the dock vicinity has developed with the dock as the upstream extremity and the fisherfolk village as the mid-point. Based on our field observations and interview, this erosion may be attributed to the following factors :

①In recent years, the channel containing the shoals upstream from the Project site has shifted its main flow towards the south side channel, causing the current to directly strike the riverbank line, with the upstream edge of the erosion at the dock.

②Along with changes in the flow channel, river width and riverbed depth have both increased, leading to an increase also in retained water volume within the river channel. This, in turn, has resulted in an increase of flow volume in the direction of the bend.

③Furthermore, as a result of the relatively heavy rainfall that has developed in recent years, the influx of rain into the river has increased, causing an increase in the retained water volume in the river channel. This is believed to have been a factor aggravating the erosion. In the course of our interview at the fisherfolk village, we were told that, during the 1996-97 rainy season, many houses were carried away as a result of riverbank erosion. Although the fisherfolk village was formed 12 years ago, this was the first accident of this nature. Since a relatively sizable amount of rain fall over a short period during this particular rainy season, it is presumed that the disaster was caused by an increase in retained water volume in the river channel which served to aggravate the erosion.

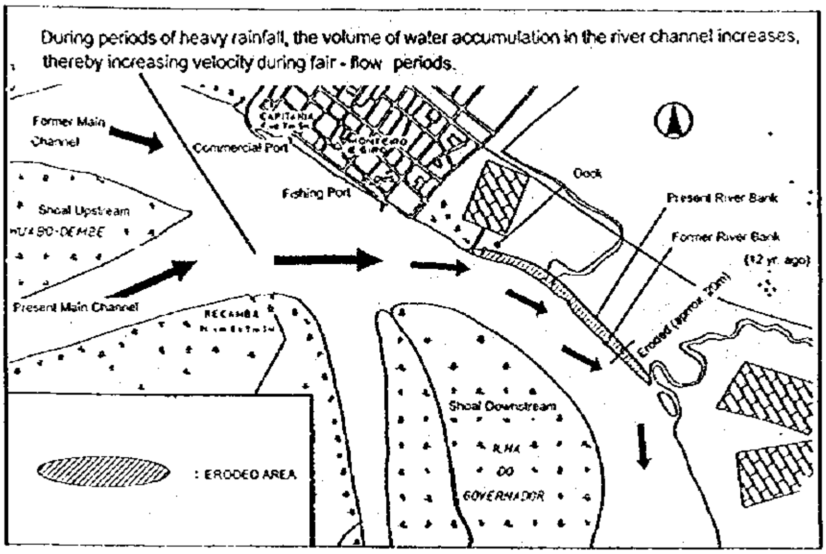


Figure 2.3.-6 Development Sequence of Riverbank Erosion

(2) Local Scouring in Front of the Structure

The local scouring due to the structure has resulted from an increase of velocity around the structure in the middle of the current as well as the displacement of riverbed composite material, based on the current flowing toward the riverbed as a consequence of the turbulent flow action (e.g., eddies and secondary currents). Judging by the results of the riverbed depth sounding and observations of flow direction and velocity, the most severe locations and range of this local scouring in the existing structure were in the protective steel sheet pile wall in the front revetment and at the corners of the centered portion of the dock, which receive the direct impact of currents in both directions during fair flow and reverse flow periods.

The following figure shows a conceptual chart of the local scouring.

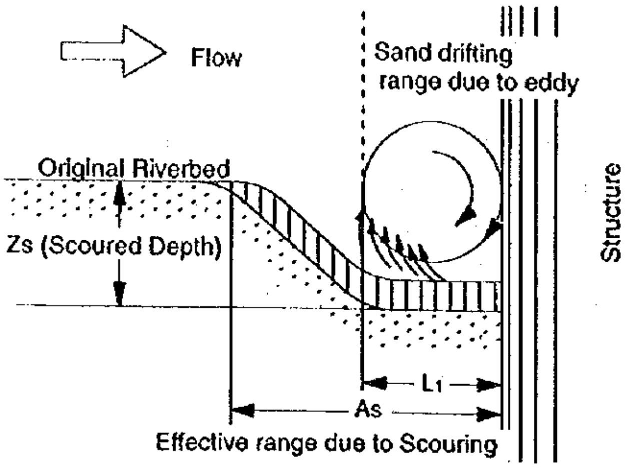


Figure 2.3.-7 Conceptual Chart of Local Scouring

2.3.3 Future Projections of Riverbank Erosion

The causes of the riverbank erosion in the vicinity of the dock include both natural conditions i.e., erosion at the bend of the river and local scouring due to the structure itself, and, in our judgment, this erosion is likely to continue in the future. While it is, therefore, essential that a reasonable attempt to forecast future erosion conditions be established as a design condition for the subject Plan, in view of the limited availability of river data in the area, it will be necessary to rely on forecasting methods based on the available source materials.

Generally speaking, the following methods are available for forecasting and evaluating riverbank erosion :

① Evaluation techniques based on secular data trends

② Evaluation techniques based on past research

③ Evaluation based on numerical computations

④ Evaluation based on hydraulic model tests

Among the above forecasting methods, Method 1(using secular data trends) would be somewhat feasible, but, with regard to Method 2 (using past research findings), research history is as yet inadequate on riverbank changes due to tidal action in tidal rivers, such as that in the survey area. As to Methods 3 (numerical computations) and 4 (hydraulic models), the lack of local topographical and hydraulic data would present serious obstacles to establishing forecast conditions. Accordingly, our plan is to make future forecasts of riverbank erosion -- i.e., deepest riverbed height and speed of riverbank erosion -- on the basis of Method 1 (secular data trends).

(1) Deepest Riverbed Height

"Deepest riverbed height" can refer to either the riverbed height, as determined from river characteristics, or that determined on the basis of local scouring of the structure. The riverbed height will be estimated here on the basis of river characteristics, as the local scouring could be suppressed by the protective measures.

There is a possibility that the riverbed height in front of the structure may become deeper than that based on river characteristics, but this phenomenon would be local in nature, and its progress could be controlled by construction work designed to cope with scouring.

With respect to the deepest riverbed height, based on a comparison of the riverbed depth sounding findings and past navigation charts, the stable riverbed depth in the bend section will be set at -14 m for the following reasons:

①Based on the riverbed depth sounding charts, as prepared in September, 1992 and July, 1997, the deepest sections of the riverbed surface are about -13 ~ -14 m. Over the past five years, riverbed height has been found to be virtually stable.

②Based on a comparison between July and December, 1997, it has been confirmed that, while the isobath contour has indeed extended to the downriver side, the deepest riverbed height has not fallen below -14 m.

(2) Riverbank Erosion Speed

As is clear from the results of river cross-section sounding, the riverbank erosion is advancing in tandem with the steep pitch of the riverbank, based on erosion of the bank base and an accompanying succession of collapses. Since this riverbank erosion is believed to have resulted not just from the influence of normal tidal action but also as an intermittent phenomenon that has suddenly advanced during periods of rainfall and spring tides, it is possible to estimate the speed of this erosion on the basis of average values over a relatively long period. As a result, the erosion speed on the riverbank in the vicinity of the dock can be evaluated as an average annual value derived from a comparison of secular data trends. The speed of riverbank recession will be set at an average of 2.3 m/year for the following reasons:

①Based on the riverbed depth sounding charts prepared in September, 1992 and in July and December, 1997, we have tentatively calculated the average distances between the respective isobath contours (-10, -7, -5, -3, ±0, and +2 m) and the base lines in front of the existing revetment.

Over the 5.3 year period from September, 1992 to December, 1997, the receding distance of the various isobath contours has ranged from a low of 10.8m to a high of 12.3m, thus the difference in erosion speed, based on depth, was small (only 1.5m). It may, accordingly be said that each depth contour evidenced about the same erosion speed. An annual value of 2.30m was obtained as the receding speed of the riverbank line, based on erosion over the same period. (cf. Appendix 5-4.)

②Based on the interviews conducted at the fisherfolk village downstream from the dock, we were told that the riverbank had receded about 20m over the past 12 years. Converted to an annual average value, this figure works out to approximately 1.67 m/year.

③Based on a comparison of the aerial photographs taken in June, 1996 with those taken from an aircraft during our December, 1997 survey, and rough estimates of the receding conditions on the riverbank in the vicinity of the fishing village, the annual erosion speed on the riverbank is believed to be running 2.0 ~ 2.6 m/year.

(3) Scope of the Erosion Protection Work

Based on the above considerations, the deepest riverbed height has been calculated at -14 m and the erosion speed at 2.3m / annum.

With regard to the face of the existing revetment, based on a front riverbed depth for the erosion protection work of -14m, a mound will be constructed to provide the required stability to the existing revetment. With respect to the dock sides, although, at the present time, no riverbank erosion has reached to date, the surface level of this section will be reduced as a result of future erosion progress. Thus, the scope of the construction project for a protective structure that will not affect the structure of the main dock body will be determined in a manner similar to that for the steel members used in the existing dock facility. That is to say, since the margin vis-a-vis corrosion of the steel components has been set at 30 years, considering the time that has already elapsed since completion of the facility, the scope of the protective structure will be set as required to withstand erosion for a period of 25 years following the restorative work.

The conditions, as above described, are shown in the following illustration.

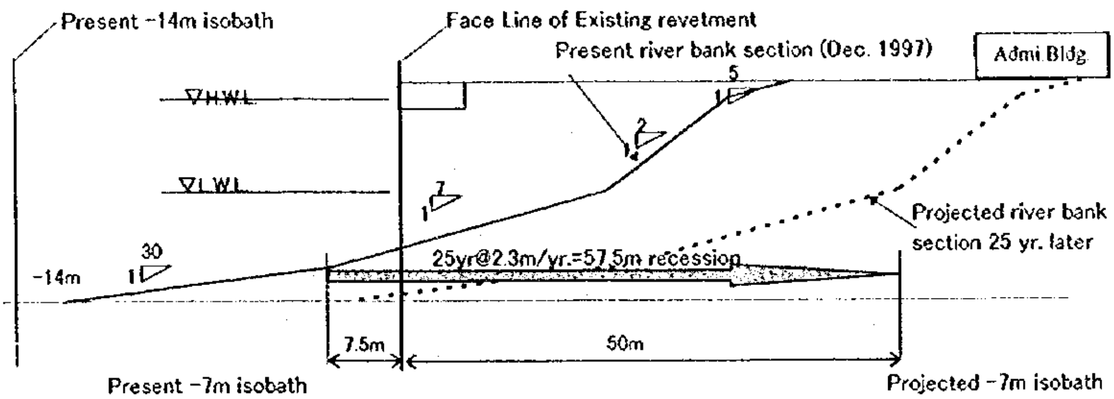


Figure 2.3.-8 Riverbank Section under Present Conditions vs. a Projected Section in 25 years

2.4. Design for the Optimum Project Plan

2.4.1. Design Guidelines for the Erosion Protection Work

Design guidelines for the basic restorative construction will be established on the basis of an evaluation of construction methods and structural forms for the erosion protection work.

(1) Comparative Evaluation of Construction Methods

Generally speaking, erosion control methods for river banks, as shown below, include :

Table 2.4.-1. Types of Erosion Control Methods for River Banks

Erosion Control Methods	Types of the control methods
1) Erosion Control Work on the Bank	Bank protection work
2) Work to Control Flow Direction	Spur dike (groyne) work
	Groundsill work
3) Combination of Bank Erosion Control and Flow Direction Control	Bank protection plus spur dike works
	Bank protection plus groundsill works

The bank erosion control method involves direct shore protection of the eroded section, using an artificial structure. As a result, this method not only has immediate effect but also certainly facilitates achievement of the desired results.
Facilities used in this work comprise shore protection work and foot protection work to guard against scouring.

Flow direction control, on the other hand, involves diverting the watercourse to the center of the river to isolate the bank from the erosion while stimulating sedimentation in front of the bank. The facilities for this method include either spur dike or groundsill works. With the former, problems exist in terms of both immediate effectiveness and reliability, since results are strongly influenced by the characteristics of the particular river. Furthermore, as the riverbed protection work should extend across the river, this method is not economically advantageous in the case of wide rivers.

Taking into account the special characteristics of the above erosion control options as well as the specific conditions required for bank erosion control in the survey area -- i.e., the fact that the target facility to be protected is a fishing boat repair facility and thus localized --, and considering also the certainty of immediate benefits from the work, we have concluded that the bank erosion control method is superior to flow control for purposes of this project.

In general, since revetment destruction is usually triggered by scouring of the base portion, in the case of the Plan river as well, it will be necessary to include foot protection work designed to reduce flow power at this point and so alleviate sudden scouring by directly covering the riverbed.
Accordingly, in the erosion protection work in front of the Quelimane Fishing Boat Repair Facility, we plan to install foot protection work to protect the structure exposed to the river flow.

(2) Comparative Evaluation of Structural Forms

The following three structural alternatives can be considered for the erosion protection work in light of the present erosion conditions, prospective future erosion, soil conditions, and the structure of the existing revetment.

①Mound type

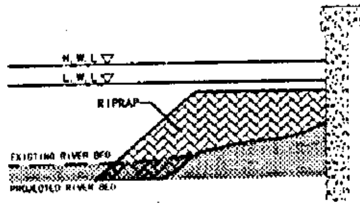
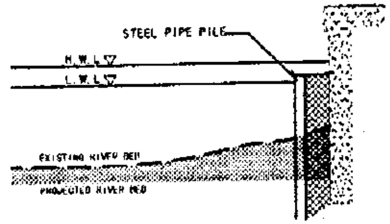
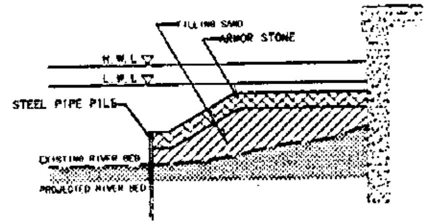
②Wall type

③ A combination of the mound and wall types

In the mound method, the scoured portions of the revetment in front of the dock are refilled with rubble, and a mound is constructed to protect the structure against future erosion. In the wall method, pipe piles and sheet piles are driven in the vicinity of the existing revetment to support the active earth pressure at the dock. When the two methods (mound and wall) are combined, the pipe piles are driven at a certain distance from the existing revetment to prevent further erosion, while a mound is built up in front.

We have carefully evaluated the pros and cons of the above three methods, and the results of this assessment are summarized in the Table 2.4.- 2.

Table 2.4.-2 Evaluation of Structural Methods for the Erosion Protection Work

Conditions	Mound Method		Wall Method		Combination of Mound & Wall	
Overall Structural Chart						
Characteristics	A mound is built up based on the introduction of stone. This method is frequently used with a favorable river bed surface.		Pipe piles and sheet piles are driven in the vicinity to protect against further erosion. Depending on water depth, the work can become structurally large-scale.		This is a combination of the mound and wall methods. Depending on conditions, by blending the two techniques, their respective defects can be offset.	
Impact on Existing Facilities	None, since stone is introduced.	○	Since piles must be driven in the vicinity, the work will have a major impact on the existing structure.	×	As the piles are driven at a distance from the existing facilities, the impact of the driving operation can be suppressed by firstly the introduction of rubble in front of the revetment.	△
Safety	Should erosion continue in the future, there is a danger that the toe of slope will give way, creating a safety problem, however it will be no problem when surplus rubble stone amount are placed at toe and slope.	○	While there is a danger of scoring in the vicinity of the piles, this problem can be solved to some extent by foot protection work. .	○	Despite the danger of scoring in the vicinity of the piles, this problem can be solved to some extent by foot protection work.	○
Construction Conditions	Since a large quantity of stone will be introduced, while dredging will also be required, heavy-duty construction equipment will have to be used. A certain margin will be required in the amount of stone used, making this method inferior in terms of construction reliability.	△	Since large-caliber steel piles of 1,800 mm will have to be driven, large-scale construction equipment will be required. Non-vibration piling method may be required, thus resulted in the need of many types of construction machinery. Construction reliability can be achieved.	△	The required caliber is smaller than that required for the wall method alone, making it possible to use medium-scale construction equipment. Construction reliability can be achieved.	○
Material Conditions	Materials must be procured from third countries. A large quantity of stone will be required.	△	Materials must be procured from third countries. There is no need for building stone.	×	Materials must be procured from third countries. Since locally gathered sand can be used in filling work, it will be possible to hold down the volume of stone used.	△
Maintenance Conditions	Maintenance and repairs will be required for the toe of the slope bottom.	△	Maintenance requirements will be modest.	○	Maintenance requirements will be limited.	○
Economy	This method is generally economical; however, in the Plan area, difficulty in obtaining local stone material will be a negative constraint.	△	In view of the deep water, rather large structural members will be required, making this method uneconomical.	×	Quite economical.	○
Work Period	12 months at the minimum, varying on the method of stone transportation	△	Pile driving requires specific method and longer work period needed than other two methods.	×	Estimated at approximately 9.5 months, being shortest among three method.	○
Overall Evaluation	There will be some safety problems with respect to development of future erosion. Inferior from an economy standpoint, owing to the need for dredging operations.	△	Major impact on the existing revetment. While safety can be maintained against any future erosion, this method is inferior from an economic standpoint.	△	Economically advantageous, while safety is also assured vis-a-vis any future erosion.	○

At some locations in the Project area, riverbed depth reaches -10 m, even directly in front of the dock, while soft soils remain in the riverbed. Since it is assumed, from the river conditions, that bank erosion continues to advance, with respect to the mound method, it is likely that the slope toe portion is vulnerable to scouring, which will create a need for future maintenance, while safety problems will remain.

With the wall method, while the structure itself is simple, in as much as this system involves piling operations in the vicinity of the existing revetment, which is already in a structurally perilous state, the impact of the construction work will be considerable. Also, since riverbed depths run deep, the length for the self-supporting sections will also be substantial, requiring the use of large-caliber steel pipe piles. Constraints will also arise with respect to the procurement of materials and construction equipment, thereby creating economic problems.

With a combined mound/wall approach, although the principal materials and construction equipment will have to be sourced from third countries, since local sand can be utilized in the fill operation, while the scale of construction equipment will be smaller than in the case of the wall system alone, the combined method will be most economical.

Based on the above assessment, the combined mound/wall method has been deemed appropriate as the structural system for the erosion protection work.

2.4.2 Basic Plan

(1) Design Conditions

Based on the results of our survey of natural conditions, the design conditions adopted for the subject Plan will be as follows.

a) Oceanographic Conditions

・Tidal Levels

H. W. L.	C. D. L.	+4.60 m
M. S. L.	C. D. L.	+2.70 m
L. W. L.	C. D. L.	+0.70 m

・Max. Flow Velocity 150 cm/sec

b) Soil Conditions

Soil conditions have been set as follows.

+2.50 m		
	Cohesive Soil	
	Unit Weight	γ = 1.6 tf/m3 (atmospheric)
		γ ' = 0.6 tf/m3 (underwater)
	Cohesion	C = 0.7 tf/ m^2 (according to uni-axial compression test)
-5.50 m		N value = 1
	Sandy Soil	γ = 1.8 tf/m3 (atmospheric)
		γ ' = 1.0 tf/m3 (underwater)
-7.50 m		N value = 14, $\phi = \sqrt{12N} + 15 = 28°$
	Sandy Soil	γ = 1.8 tf/m3 (atmospheric)
		γ ' = 1.0 tf/m3 (underwater)
-13.0 m		N value = 30, $\phi = \sqrt{12N} + 15 = 34°$
	Cohesive Soil	γ = 1.8 tf/m3 (atmospheric)
		γ ' = 1.0 tf/m3 (underwater)
-27.0 m		N value = 20, (C = 4.85 tf/ m^2)
	Sandy Soil (foundation layer)	γ = 1.8 tf/m3 (atmospheric)
		γ ' = 1.0 tf/m3 (underwater)
		N value = 40, φ = 40°

c) Seismic force

Seismic force is not taken into consideration.

d) Existing Revetment Design Conditions

- Crown height of the existing revetment C. D. L. + 6.0 m
- Surcharge (normal condition) 1.0 tf/ m^2

e) Design riverbed C. D. L. -14.0 m

f) Applicable Standards

Technical Standards for Design of Port Facilities (1989) (supervised by the Ministry of Transport, Japan)

Standard Design Method for Fishing Port Structures (1990) (supervised by the Fisheries Agency, Japan)

Japan Industrial Standards (JIS)

g) Construction Materials

①Filling Materials

Material	Averaged N Value	Unit Weight (tf/m3)		Internal Friction Angle (°)
		Moisture	Underwater	
Filling Sand	5	1.80	1.00	27.5
Rubble	40	1.80	1.00	35

② Steel Material

Material	Allowable Stress (kgf/cm²)
Steel Pipe Pile (SKK 400)	1,400
Steel Sheet Pile (SY 295)	1,800

③ Corrosion Rate of Steel

Corrosive Environment		Corrosion Rate (mm/year)
Sea Side	Above H. W. L.	0. 3
	H. W. L.~ L. W. L.−1.0 m	0.1 ~ 0.3
	L. W. L.−1.0 m~the sea bed	0.1 ~ 0.2
	Below the sea bottom	0.03
Land Side	In marine atmosphere	0.1
	In soil (above residual water level)	0.03
	In soil (below residual water level)	0.02

④ Concrete

Material	Unit Weight	Allowable Stress
Reinforced concrete	2.45 t/m3	Fc = 24 N/mm²
Plain concrete	2.30 t/m3	Fc = 18 N /mm²

(2) Assessment of Mound Shape

a) Crown Height and Width

The base of the existing front revetment has been subjected to scouring, while the passive earth pressure at this revetment, which has a steel sheet pile structure, has fallen, raising fears of imminent collapse. This critical situation has made it necessary to build a mound by filling sand into the front portion of the revetment so as to raise the passive earth pressure. In order to restore structural stability to the existing steel sheet pile revetment, the required crown height of the mound will be determined on the basis of the types and embedment length of the steel sheet piles used in the present revetment, while the crown width of this mound will be determined on the basis of a value that will satisfy the safety factor for embedment of the steel sheet piles (at least 1.50).

The results of our assessment, based on these condition's are shown in Table 2.4.-3. For the calculation of the mound section, refer to Item 6, Appendix-8.

Table 2.4.-3 Mound Crown Height and Width

Position	Mound Height	Mound Width	Safety Factor
Front central portion	D.L. − 2.0 m	17.5 m	1.51 > 1.50
Both front edges	D.L. ± 0.0 m	17.5 m	1.51 > 1.50
Both side sections	D.L. + 2.5 m	10.0 m	1.53 > 1.50

The types and embedded depths of the steel sheet piles used in the front revetment all differ in the 3 sections shown in the above table, since the assumed river bottom sections at the time the design was made were all different. As a result, the mound crown heights and widths, as required to restore passive earth pressure, are structurally different.

In addition to the structurally required sectional shape, it will also be necessary to consider : (1) the flow for preventing local scouring ; (2) safety of vessel operations when entering the Dry Dock ; and (3) ease of construction and durability.

To prevent local scouring, it is essential that a rough flow be avoided, and so, from this standpoint, it would be advantageous to establish uniform mound heights and widths. From the standpoint of fishing vessel maneuvers too, safety would be enhanced by having a river bottom section in front of the revetment. And, in terms of construction ease as well, if the finished height of the river bottom section differs by location, sloping of mound would be required, complicating the construction work.

Based on the above considerations, it is clearly desirable that the mound height and width be uniform, in which case it would be most sensible to set at -2.0m, level with the depth of the concrete slab in front of the dock gate.

However, structurally speaking, mound height at both ends and sides of the existing revetment must be 0 m and + 2.5m, respectively. For this reason, we have concluded that it would be most advantageous to maintain uniform mound height and width across the entire section, by driving new steel sheet piles next to the existing revetment so as to reinforce it at both edges and sides.

b) Evaluation of Mound Construction Materials

Let us next consider the main materials required to construct the mound during the erosion protection work.

These materials must satisfy the following two conditions.

1. Stability vis-a-vis river flow speed
2. Effectiveness as passive earth pressure.

Following is a comparative evaluation of the materials satisfying the above conditions:

Table 2.4.-4 Comparative Evaluation of Alternative Mound-building Materials

	Sand (including sandbag)	Mortar Fill (in sandbags)	Rubble
Specifications	Local river sand γ =1.7 ton/m3	Mortar in sandbag by mixing river sand with cement γ =1.8 ton/m3	Spreading rubble γ =2.3 ton/m3
Stability against Current	Unstable	While this material is superior to sand (sandbags), it would lend no stability on the slope.	Stable
Effectiveness as Passive Soil Pressure	Effective	Effective	Highly effective
Source	Locally available	Locally available	South Africa
Delivery	No problems	No problems	Would be brought in by private vessel
Price	Inexpensive	Expensive	Expensive
Evaluation	It would be inappropriate to seek the desired stability on the basis of sand alone. However, by combining sand with scour protection mats, this material could serve as filling material.	By increasing the weight of mortar-packed sandbags, this material would be usable, but quite expensive, considering local construction conditions.	Most suitable for project. Volume can be reduced by use of fill material (sand)

Based on the above evaluation, we have concluded that it would be most appropriate to use sand as the fill material for the mound and rubble as the armor layer to protect the mound surface.

c) Armor stones

While sand will be used as the fill material, since the design value for maximum river velocity at the facility has been set at 150 cm/second. It will be necessary to protect the sand fill by spreading stones over the mound surface. In this case, the requisite stable weight of the armor stones will be determined by the following formula:

$$dg = \frac{v^2}{2\,gy^2\,(Sr - 1)\,(\cos\ \alpha - \sin\ \alpha)}$$

Wherein:

- dg: size of the rubble (m)
- Sr: specific gravity of rubble (γ/1.03) = [2.65]
- v: flow velocity on rubble surface (m/sec) = [1.5m/sec]
- g: gravity acceleration (m/sev) = [9.8 m/sec]
- α: slope gradient (°) = [26.565]
- y: Isobath constant = [0.86 - 1.20]

Based on the above formula, the required weight of the armor stones will be as follows:

dg = 0.11 - 0.21m/piece or greater→ 25cm or greater (15,625cm^3 or greater when converted to a cube)

Thus the stone weight comes to; w = 15,625cm^3 x 2.65 = 41.4kg → 50kg/piece or more.

d) Scour protection mat

If we use rubble, as stipulated above, there is no danger of the stones covering the surface being washed away. But, to prevent an efflux of the interior sand fill, a scour protection mat will be required between the sand fill and the armor stones.

Based on the above considerations, the shape of the mound for the erosion protection work will be as shown in Figure 2.4.-1

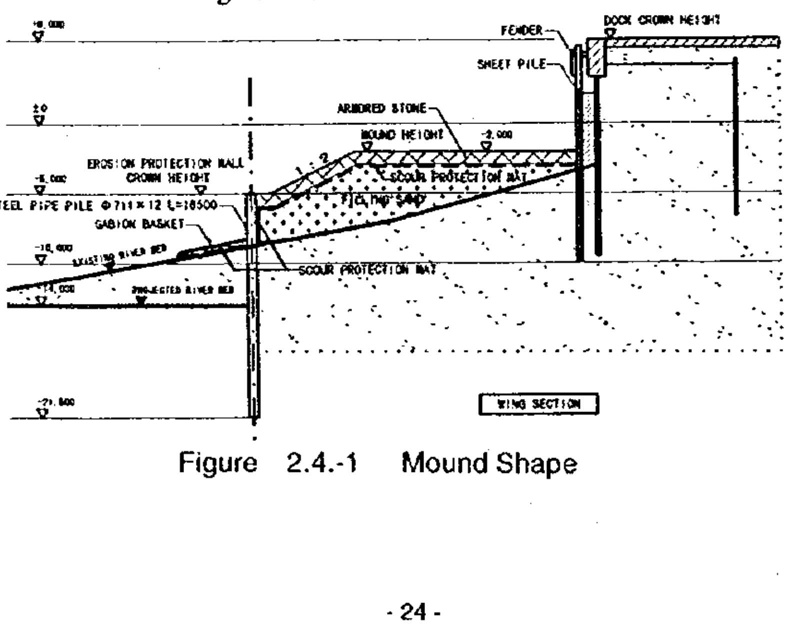


Figure 2.4.-1 Mound Shape

e) Stability of the revetment

On the assumption that a mound will be constructed, to restore safety to the existing revetment, in order to confirm the safety of the revetment, we have verified the safety factor of the bank as a whole, based on circular slip.

As shown in Table 2.4.-5, four sections have been stipulated for the verification process. The safety factors for circular slip have been computed for each section. The structural stability analysis is shown in Appendix 6.

Table 2.4.-5 Safety Factor for Circular Slip by Section

Target Section in the Revetment	Safety Factor
① Section on upstream side	1.422
② Section in front of gate	1.361
③ Section in front of gate	1.334
④ Section on downstream side	1.416

Based on the above findings, the safety factors computed for the four sections will all be at least F = 1.3 and so satisfy the design criteria.

(3) Erosion Protection Wall

As examined in Section 2.4.1, the erosion protection work will involve a hybrid structure, combining mound and wall construction, but an erosion protection wall will be required to protect the base section of the slope toe portion from erosion. An erosion protection wall will also be needed on both sides of the dock to protect the dock structure from future erosion encroachment. We shall consider below the structure and required coverage area of these erosion protection wall.

a) Structural Method for the Erosion Protection Wall

Possible materials for the erosion protection wall include steel pipe piles, steel sheet piles, and concrete pipe piles.

The essential condition, from a structural standpoint, is that the structure must be self-supporting even if the riverbed depth were to fall to -14 m, which is the presumed riverbed surface. Based on procurement considerations, it would also be advantageous to be able to procure uniform quality materials within a short period of time. And, from the standpoint of construction ease, the materials must facilitate underwater construction work while maintaining construction precision.

Steel sheet and concrete pipe piles have weaker stiffness for bending. The required self supporting distance above the -14m level or the projected riverbed level will reach to 9m, provided that these materials can be driven into -5m level in the water. In our judgement, steel sheet and concrete pipe piles would be difficult to use as wall material for limited structural tolerance against bending moment. Steel pipe piles can resist against large bending moment and thus commonly used for deep water quay or deep self-supporting walls from the structural points. In terms of construction efficiency, they would also be suitable for driving operations in water using flat barges. From a procurement standpoint, these material can be sourced from neighboring South Africa. Based on the above considerations, it is felt that steel pipe piles would be appropriate as the material for the erosion protection wall.

In designing the erosion protection wall, we refer at first to the soil conditions of the site, which are shown in Section 2.4.2 and in Appendix-8 at the end of this report.
The steel pipe piles receive no surcharge as the erosion protection wall material, however it must protect the mound toe portion as a self-supporting wall when the river bed would have been eroded to a depth of -14m. The design process should start with calculation of total active earth pressure derived from the virtual riverbed level, followed by calculation of stress of pile based on Chang's formula as an exposed pile from surface. Then the coefficient has been decided, which were determined by lateral resistance of ground, pile width and stiffness against bend. The required embedded length of pile was calculated with this coefficient and we have verified if the deformation at the pile head is less than 10cm or not. These design procedures generally follow the Standard Design Method for Fishing Port Structures (1990) (supervised by the Fisheries Agency, Japan).
From the standpoint of reducing soil volume for mound embankment, large diameter piles being driven as close as the existing revetment will have an advantage. Larger caliber piles also reduce the required number of piles to form a certain length of protection wall. However, the maximum diameter of pile will actually be limited by the construction conditions including availability of pile driving equipment and expertise. For setting the most appropriate pile diameter and thickness, we have examined two types of steel pipe pile, ϕ700 x 12t, which have been used in large quantities in the foundation section of the existing dry dock structure and so have already proved themselves in construction work, and a smaller ϕ600 x 14t pipe.
As a result of the comparison, the required weight of steel material per meter of wall is 4.670t for ϕ700 pile and 5.247t for ϕ600, and the required number of pile per meter of wall comes to 1.136 pieces for ϕ700 and 1.282 pieces for ϕ600, with the width of joint between piles set at 180mm. From the above examination, the ϕ700 piles demonstrate

advantage in construction of erosion protection wall requiring less steel weight and pile driving work. The maximum depth of pile head underwater is -5m with the pile driving method using a pile coupling tube.

As described later in Section 3.1.5, the source country for steel pipe piles will be South Africa, the diameter of the steel pile shall be described as ϕ 711 x 12t, in compliance with the South Africa standards (ϕ 28 inches).

Accordingly, the specifications of the steel pipe piles for the erosion protection wall will be ϕ 711 x 12t, with embedded depth to -21.5m. The Appendix-8 shows the above examination process and results.

b) Range of the Erosion Protection Wall

As concluded in Section 2.3.3., the range of the erosion protection wall has been set to cope with erosion over the next 25 years, even assuming that the present erosion speed continues throughout this period.

While the soil at both sides of the dock have thus far been erosion-free at present, the erosion protection wall will become necessary to protect the main dock structure in the event that the soil in these sections will be washed away, owing to future erosion progress. If erosion protection wall are also built on the sides, using steel pipe piles of the same specifications (ϕ 711 x 12t) as those for the wall in front of the existing revetment, the limiting factors both in structure and in work method are as follows.

- Self-supporting length of pipe pile becomes 7 ~ 8m from the pile top under the existing soil conditions.
- Pile head level in subsoil section will be 5m from the surface even when driven by using a pile coupling tube.

From the applicable work method, the top level of the erosion protection wall will be determined by the present ground level, and the highest pile head level will thus be $\pm$0m, as the ground level is about +5.0m at the highest point around the dry dock. At this point, the pile can be self-supported until the ground level is lowered to -7m, as the self-supporting length of the pile is 7m. The erosion protection wall, therefore, will be built on both sides of the dock at a range that anticipates a riverbed depth of -7m or deeper after 25 years of erosion. In the event that the present -7m isobath contour, which runs about 7.5m offshore of the existing revetment, advances toward the shore side at a rate of 2.3m/year x 25 years -- i.e., by a total of 57.5m over the period --, its location would shift 50m from the front revetment of the dock.

We have next examined the stability of the slope formulated between the end of the erosion protection wall and the edge of the working apron of the dock after 25 years of erosion. From the survey results, the gradient of riverbank between ±0m and -7.0m, which receives constant erosion by tidal actions, shows a slope of 1:2. This slope is considerably steep but still stable against circular slip. With the existing soil conditions and erosion mechanism, we may judge that the same angle in this section may be kept during the progress of the riverbank erosion, then the slope of the ground at the end of the erosion protection wall should be less than 1:2, or 25.6 degree. In the meantime, in the "Standards for Erosion Control Technology on Rivers (Design Edition) of Ministry of Construction, Japan (1993)" suggests that normally 30 degree of riverbed gradient to be assumed when the erosion is in progress. It is therefore necessary to confirm that the slope will be less than 25.6 degree or 1:2, in order that the slope will be stable.

Difference of ground level at the end: 10m(Existing ground level C.D.L+5.0m - ground level at the wall end after 25 years' of erosion C.D.L-5.0m) (-7.0m contour line runs actually at the extreme fore position of the last section of the wall and the level at the rear pile is estimated at -5m) (See 0-0 Section, Basic Design Drawings in Section 2.5)

Horizontal length between dock boundary and the erosion protection wall:
21.35m

The slope between these two points will become less than 1:2 or 26.5 degree, thus the slope is considered stable. The Appendix 9 illustrates these examination results.

On these basis, we have specified a length of 50m for the erosion protection wall on each side of the dock.

The existing ground level at 50m from the dock front is approximately +5m. Under the present conditions, the wall head will remain in subsurface layer. The top of the wall will only appear above the surface when future erosion progresses to the stage that the surface soil is washed away. At this moment, there is no difference in ground levels between the inner (dock) side and outer (river) side of the erosion protection wall, however, as erosion proceeds, only the outside ground would be washed away, since the inside soil is protected by the wall. If no corrective action were then taken, the exterior earth soil pressure -- i.e., the pressure on the passive side -- would drop, leading to the ultimate collapse of the erosion protection wall toward the outer side. In order to prevent this to be occurred, as soon as the pile head appears on the ground, it will be necessary to install a foot protection work by placing gabion baskets, etc. on the outside of the wall, to deter further progress of erosion.

(4) Scour Protection Work

Since the riverbed surface is exposed on the outer side of the erosion protection wall, while part of the current striking the wall turns downward, the embedded area for the steel pipe piles is subject to scouring action, making scouring control work essential. Since erosion of the riverbed continues to advance, foot protection, based on gabion baskets, is appropriate for this purpose, as this will facilitate the task of following changes in the riverbed contour.

The scope of this scouring control work, using gabion baskets, will be determined both by analogical reference to the results of the hydraulic model experiments relating to scouring and local observation of scouring conditions at the site.

a) Local Scouring Conditions

Based on the riverbed depth sounding, the areas in front of the dock which have experienced heavy scouring are the corner sections of the protective sheet piles. The scouring extends about 10 m offshore, covering virtually the entire area of protective sheet piling in both the upriver and downriver direction.

b) Findings from Hydraulic Experiments at Similar Structures

The following results were obtained from the hydraulic model experiments relating to the temporary coffering of an estuary barrage. (Cf. Report (III) on the Kinokawa Barrage Hydraulic Model Experiment, March, 1988, Civil Engineering Laboratory, Ministry of Construction). This experiment was carried out on the conditions shown in Fig. 2.4.-2.

Experiment model:

Flow rate: 4,000m^3/sec.(constant)

Scale: 1/50

Flow period:5 hours

Riverbed soil size: d_{60} = 0.24mm (actual particle size d_{60} = 0.25 - 20mm)

(Particle size of soil in Ouelimane is d_{60} = 0.10 - 0.15mm, as shown in Fig. 3 on Appendix-7.)

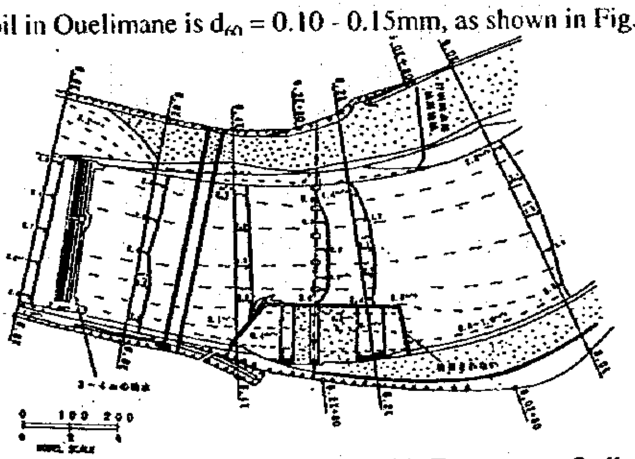


Figure 2.4.-2 Experiment on Riverbed Change with Temporary Coffering

- The largest scale scouring developed in the corner sections upstream from the temporary cofferdam (velocity: 3.5 m/s).
- While a scouring tendency was originally evidenced at the edges and corners downstream from the cofferdam, this later became a sedimentation tendency based on scoured soil and sand on the upstream side.
- At the straight portion of the cofferdam, no evidence of scouring was found, rather sedimentation tendency is apparent.

As a countermeasure to this scouring problem, despite the considerable scouring depth, in view of the fact that it was limited to a narrow range while the cause was found to be a local flow concentration in the vicinity of the cofferdam, it was proposed that a rubble mound or sand mats be installed over a 10 m wide area, as shown in Fig. 2.4.-3.

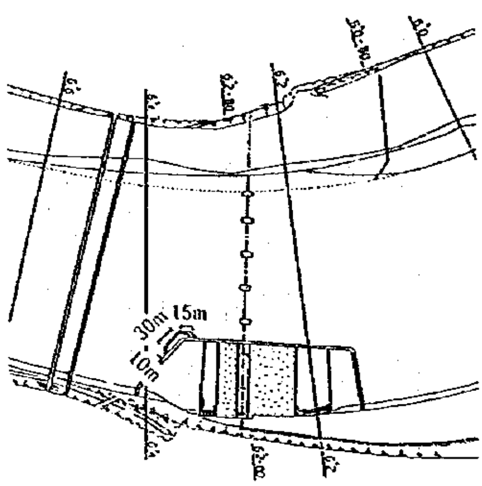


Figure 2.4.-3 Proposed Countermeasure Based on Experiment Results

c) Scour Protection in the Corner Sections (Refer to Appendix-7)

The experiment results show the largest scouring develop in the corner sections, and countermeasure of 10m width sand mats was proposed (with a velocity of 3.5 m/s). Though the river characteristics at Quelimane differ (e.g., in terms of flow volume, riverbed material, and tidal currents), scour protection in the corner sections of the erosion protection wall should be considered. The local scouring conditions at the site indicate the heavy scouring area extends to about 10m offshore, as shown in Appendix-7. Meanwhile, in the "Standards for Erosion Control Technology on Rivers (Design Edition) of Ministry of Construction, Japan (1993)" it is recommended that the following formula to be applied to determine the width of foot protection work for shore protection.

$B = Ln + \Delta Z / \sin\theta$

where;

B: Width of foot protection work

Ln: Width of flat area in front of the shore (to be greater than 2m)

ΔZ: Difference between level of foot protection work and the maximum river depth

θ : Gradient of riverbed slope being eroded (normally 30 deg.)

In this Plan, if we take Ln = 2m, ΔZ = 4m (difference between the existing depth of -10m and projected future depth of -14m), θ = 30°, the width of the foot protection work comes to B = 10m.

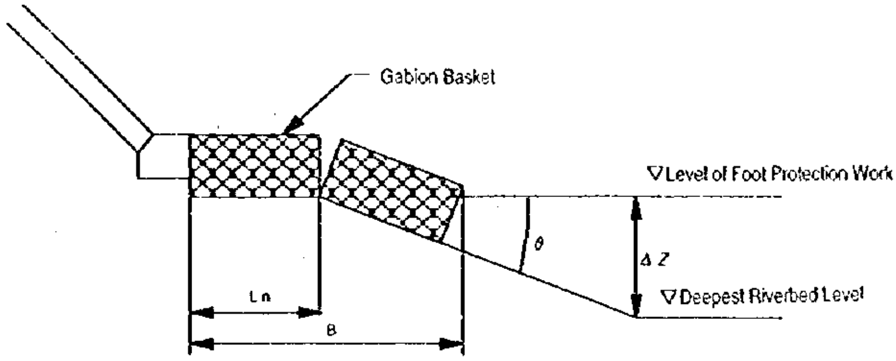


Figure 2.4.-4 Width of Foot Protection Work

In case of groundsill work, though also considered as river structures, the required length is considered on the basis of the section of hydraulic jump and following regulated flow area formed in the downstream, however this method is not applicable to the current without hydraulic jump, as is the case in the Project area..

On the basis of the experiment and the formula, both of which suggest 10m width of foot protection, we plan to set up 12m width of foot protection work by placing two gabion baskets of 6m length at the corner sections of the erosion protection wall, where there is a probability of local scouring. We also plan to apply scour protection mat beneath the gabion baskets.

d) Scour Protection in the Straight Sections

According to the Kinokawa experiment, it would be less probability of local scouring at the straight section of the cofferdam, and the area received accumulation action, rather than erosion. However, the planned area in front of Quelimane Dry Dock, where it is exposed to the constant tidal action and moreover is situated at the bend section of the river, receives flow of both fair and reverse directions, and also an increased velocity resulted from the inertial flow (refer to Fig. 2.3.-3). It is therefore desirable to supplement foot protection work by gabion baskets at the straight section of the erosion protection wall. The width of the protection must cover the area of turbulent flow with eddies and generally this area is much smaller than in the corner section. We have concluded that 6m width foot protection work to be installed at the straight section. The scour protection mat is also applied underneath of gabion baskets.

Appendix-7 illustrates the state of local scouring, expected area of scouring due to installation of protective wall, etc.
In addition to the countermeasure mentioned so far for the local scouring, it is inevitable to take precatious maintenance work after completion of the work by conducting periodical check of riverbed conditions, which is fully described later in Section 2.4.3 Monitoring Plan.

(5) Restoration of the Existing Structure

Restorative work will be carried out in sections that have been directly damaged by the ongoing erosion.

a) Repair of the Slab in front of the Dock Gate

Since a portion of the lower foundation for the concrete slab (500 mm thick) in front of the dock gate has been washed away, cracks have developed in the concrete slab, resulting in a subsidence. In addition, in the section directly in front of the gate, as the edge of the floor has risen to the surface, the gate no longer fully opens.
In the subject Plan, we plan to drive H-shaped steel piles into the gate-receiving portion at the tip, while taking down the remaining slab in other sections, lay pre-cast concrete slabs on the surface, leveled with pebbles, thereby restoring the front section of the dock gate to its original state.

b) Repair of Joint Separations

The method we have adopted to repair the joint separations between the concrete and buttress or the pump room structure on the upper section of the revetment, both up and downstream, will be to press elastic filler into the joint sections, as a sealing agent against sucking out of the soil and sand with grout to be injected to the gap.

c) Repair of Cracks in the Concrete of the Upper Revetment

Cracks in the coping concrete portion of the existing revetment are particularly evident in the self-supporting side walls on the upriver side, and we plan to repair cracks throughout the concrete section, including those in the coping concrete portion. The repair method we have chosen is to inject an epoxy adhesive, which has a fine performance record as a bonding agent for repairing of concrete cracks.

2.4.3 Monitoring Plan

Erosion still continues at the river in the vicinity of the Quelimane Fishing Boat Repair Facility. Thus, even after the erosion protection work is completed under the subject Plan, we see a distinct need for a program of continuing surveys, as outlined below, aimed primarily at monitoring scouring conditions on the riverbed. Considering the dearth of data on the Bons Sinais River, a regular program of monitory and recording river conditions in the area of the Plan facility will, in fact, also be helpful in designing other plans. However, since we have not identified any public agency engaged in such observations, for the time being, this monitoring activity will have to be carried out by the Quelimane Dry Dock itself, on the basis of the monitoring manual to be produced in the subject plan. We have determined, in this connection, that, if suitable equipment is provided, this activity would be well within the capability of the existing dock staff.

(1) Survey to Monitor Riverbed Scouring Conditions (Refer to Fig. 2.4.-5)

① Survey items :	River cross-section survey
② Survey objective :	Monitoring changes in the condition of the riverbank in the vicinity of the dock.
③ Survey frequency :	3 times a year-- before, midway through, and following the rainy season:
④ Survey method :	As shown in figure 2.4.-5, 3 cross-sections, as measured during the survey of Dec. '97, (in front of the dock, upriver, and downriver sections to the opposite riverbank), and 2 riverbank lines, 50m from the both edges of dock, at approx. 0 m contour line as of Dec. '97 survey. The traverse sections should be surveyed by a boat with an echo sounder, cruising at a constant speed along with the line extended from the two targets established at the fixed points on land, and the recording of echo sounder shall be taken for both (go & return) ways. Two riverbank lines to be measured at low tide by a level at 5m interval.
⑤ Data reduction :	Changes in riverbed conditions will be monitored by arranging the cross section contour plans in time series.

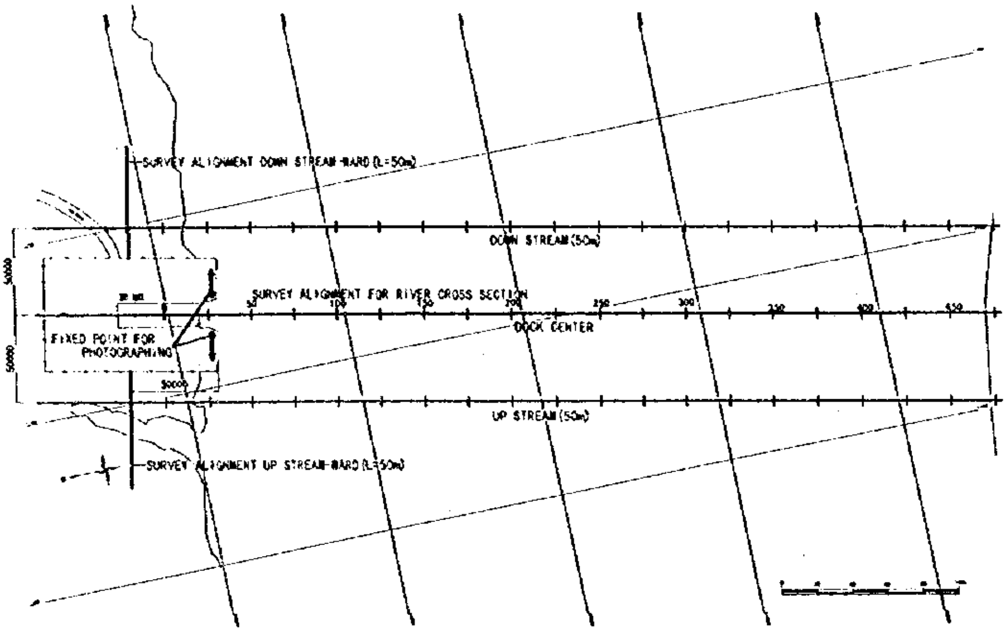


Figure 2.4.-5 Established Survey Lines and Photographing Points

(2) Survey to Monitor Erosion Conditions on the River Bank (Refer to Fig. 2.4.-5)

① Survey items :	Fixed point photography
② Survey objective :	Monitoring changes in river bank conditions over a wide area, both upstream and downstream from the dock.
③ Survey frequency :	Annually, in March and September, at lowest tide during spring tide
④ Survey method :	Fixed point photographs to be taken, from the outside corner of both wings of the dock, in upstream and downstream direction in parallel with front revetment base line.
⑤ Data reduction :	Changing conditions at the riverbank will be monitored by organizing the photographic findings in time series.

(3) Survey to Monitor Conditions at the Dock Front Structure (Refer to Fig. 2.4.-6)

① Survey items :	Measuring horizontal and vertical displacement at the front revetment
② Survey objective :	Monitoring changes in configuration at the dock facility
③ Survey frequency :	Once a month (at low tide at springs) for 2 years after completion, thereafter depending on the first 2 years' results.
④ Survey method :	As shown in Figure 2.4.-3, horizontal distances to be measured with a steel tape on 12 lines as indicated in the plan. Elevation of 8 points to be measured by a level.
⑤ Data reduction :	Change in horizontal and vertical distance on the revetment to be monitored by organizing the observation findings in time series.

(4) Survey on Water Level

① Survey items : Survey of water levels in front of the dock

② Survey objective : To gain a grasp of flooding conditions from a rise in water levels.

③ Survey frequency : At high tide in spring tide during the rainy season (from Dec. to Mar.), and approx. 1 week observation after each heavy rainfall.

④ Survey method : Reading staff-gauge in front of the dock.

⑤ Data reduction : Organize the survey findings into time series for comparison with tide table. At the time of heavy rain, data on daily rainfall at Quelimane Airport to be also recorded.

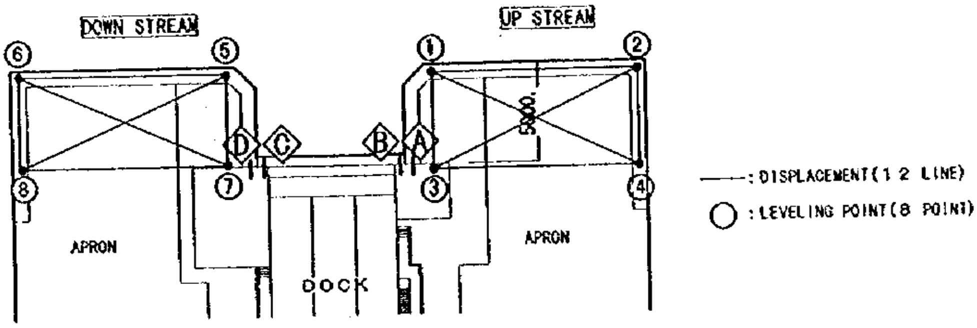


Figure 2.4.-6 Monitoring Measurement Points

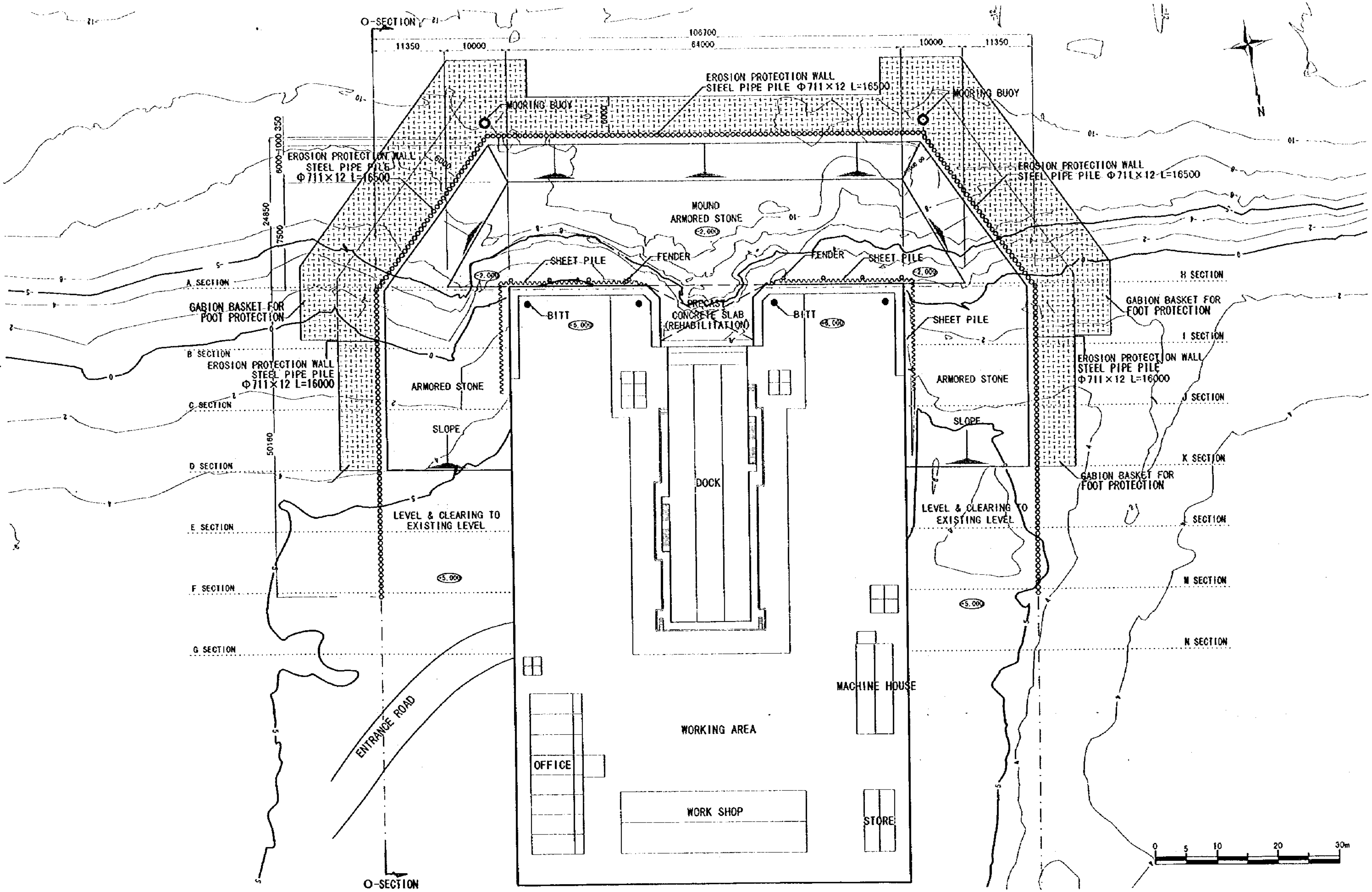
2.4.4 Monitoring Equipment

The monitoring program would entail mainly navigational observations on a fixed course from a small boat, riverbed depth sounding using an echo sounder, data processing, and photography. However, since these activities are all well within the purview of ongoing dock operations, it has been concluded that, by mobilizing equipment and personnel at the facility, program costs can be fully funded out of its regular operating budget.

Under the subject Plan, we also plan to donate the following items of equipment, which would, we believe, be difficult to procure in Mozambique, for use in these monitoring surveys.

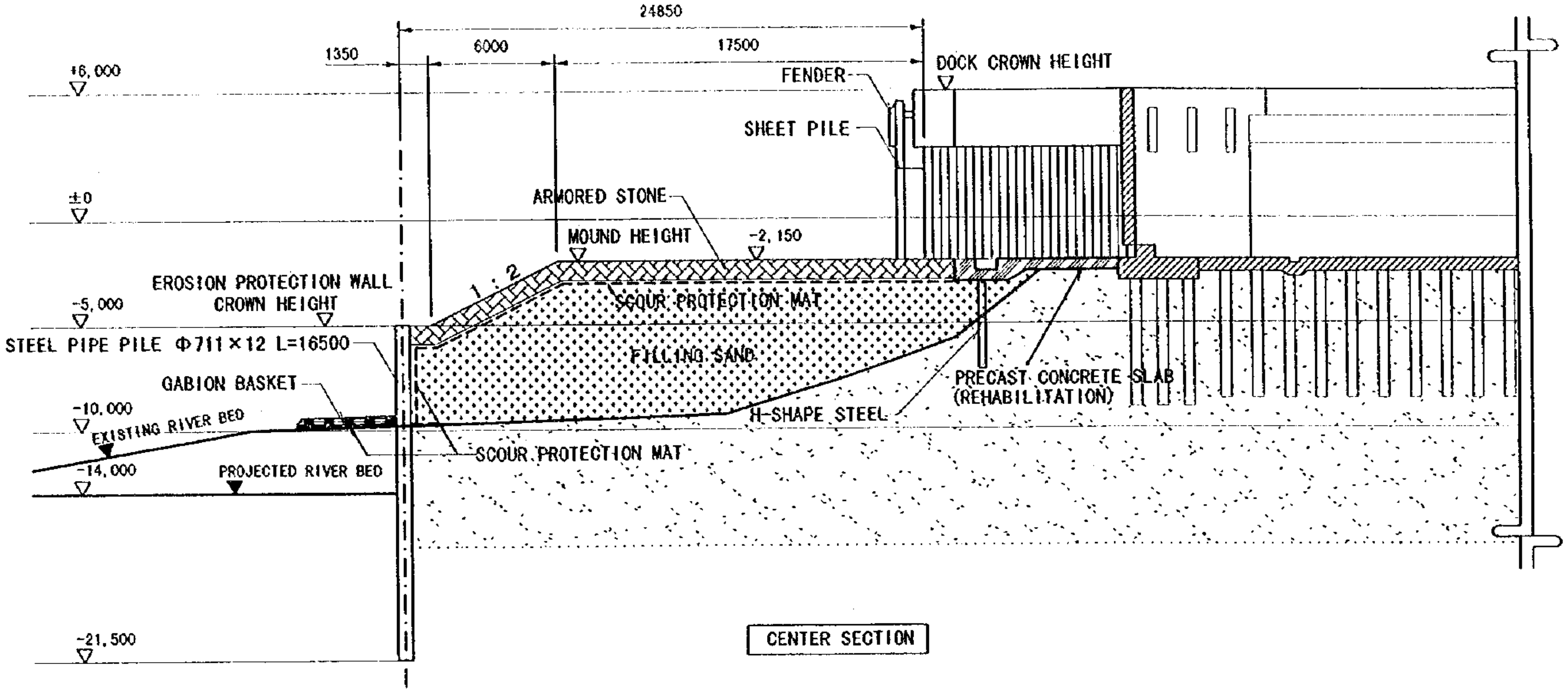
Equipment Item	Specifications	Number of Units
(1) Echo sounder	1 element, 0-50m, 200 kHz., W = 150mm, dry recording paper	1
(2) Level	Automatic, 3-class level, Leveling rod (2nd class: 5 m)	1
(3) Work boat	FRP hull, 60 ps. diesel inboard engine, total length: 8 m	1

2.5. Basic Design

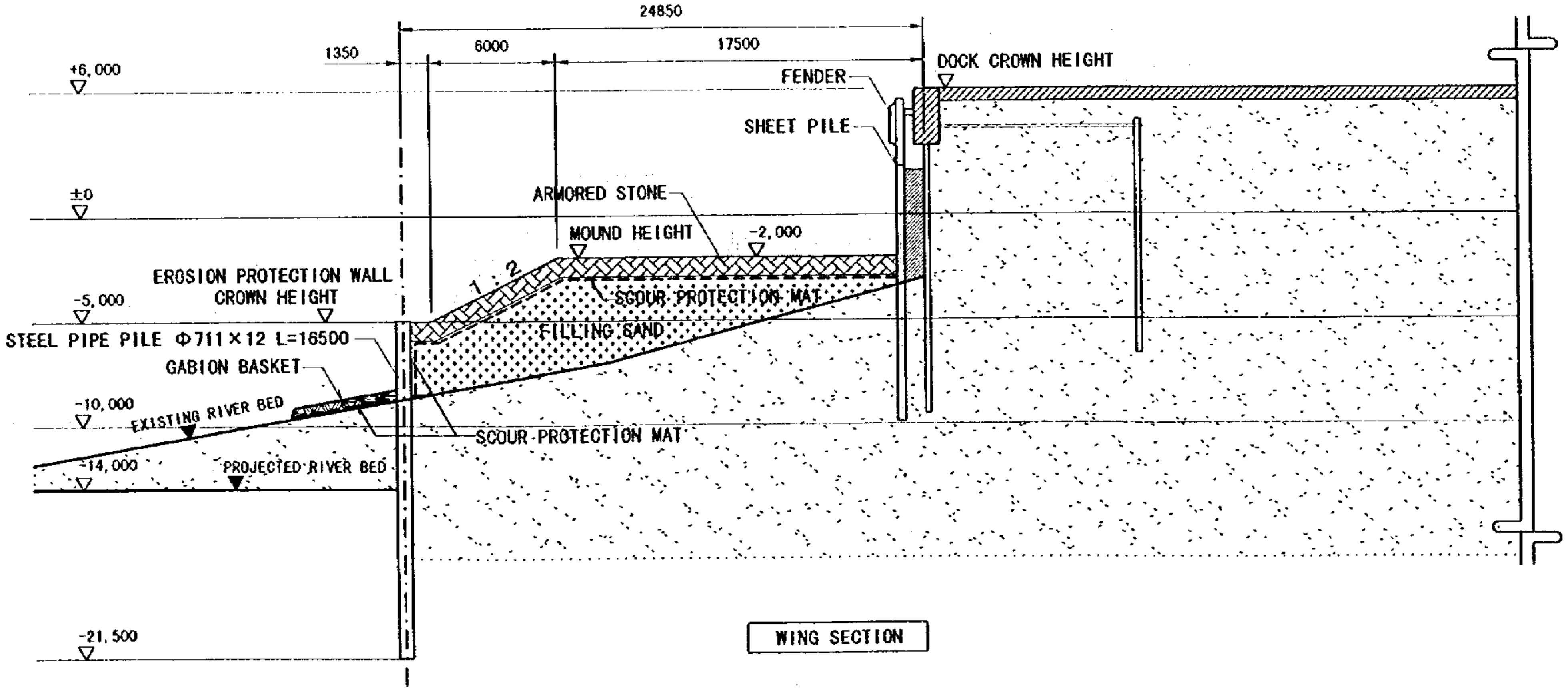


O-SECTION
106700
64000
11350
10000
EROSION PROTECTION WALL
STEEL PIPE PILE Φ711×12 L=16500
MOORING BUOY
N
MOUND
ARMORED STONE
SHEET PILE
FENDER
PRECAST CONCRETE SLAB (REHABILITATION)
BITT
GABION BASKET FOR FOOT PROTECTION
EROSION PROTECTION WALL
STEEL PIPE PILE
Φ711×12 L=16000
ARMORED STONE
SLOPE
LEVEL & CLEARING TO EXISTING LEVEL
DOCK
A SECTION
B SECTION
C SECTION
D SECTION
E SECTION
F SECTION
G SECTION
H SECTION
I SECTION
J SECTION
K SECTION
L SECTION
M SECTION
N SECTION
ENTRANCE ROAD
MACHINE HOUSE
WORKING AREA
OFFICE
WORK SHOP
STORE
0 5 10 20 30m

PLAN

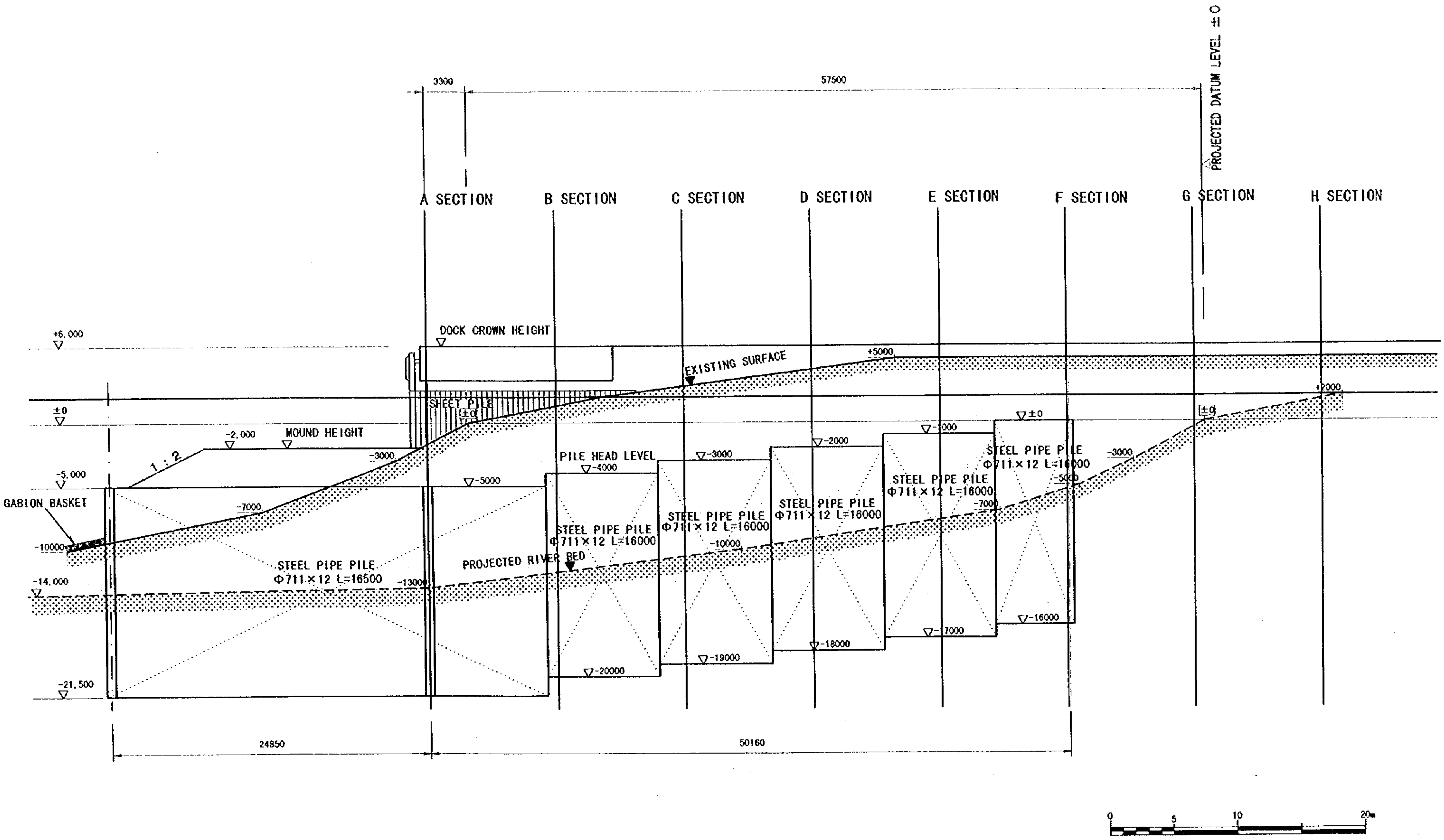


CENTER SECTION



WING SECTION

SECTION



3300
57500
PROJECTED DATUM LEVEL ±0
A SECTION
B SECTION
C SECTION
D SECTION
E SECTION
F SECTION
G SECTION
H SECTION
+6,000
DOCK CROWN HEIGHT
EXISTING SURFACE
+5000
SHEET PILE
±0
+2000
-2,000
MOUND HEIGHT
1 : 2
-3000
-5,000
-5000
PILE HEAD LEVEL
-4000
-1000
-2000
GABION BASKET
-7000
-10000
STEEL PIPE PILE
Φ711×12 L=16500
Φ711×12 L=16000
-14,000
-13000
PROJECTED RIVER BED
-20000
-19000
-18000
-17000
-16000
-21,500
24850
50160
0
5
10
20m

O—O SECTION

Chapter 3 Implementation Plan

3.1. Implementation Plan

3.1.1. Construction Concept

Considering that the subject Plan is to be implemented under a grant-aid from the Government of Japan, the construction plan will be prepared in accordance with the following guidelines.

① The project is extremely urgent.

② Advance discussions will be required with local organizations with respect to the procedures that must be carried out by the Mozambique side with a view to minimizing the time required for customs clearance.

③ The work to be carried out under this project is designed to protect the existing facilities. Thus, during the construction phase as well, particularly during the piling operations, careful attention must be paid to any changes in these existing structures.

④ While the Plan involves mainly marine construction, in view of the over 4 m differential between high and low tide in the construction area, due consideration will be paid to the local tide conditions.

⑤ Special consideration will be given to the possibility of resuming provisional use of the dock by December, 1998.

3.1.2. Special Considerations Required during the Construction Phase

(1) Selection of Construction Machinery

Availability of the construction machinery is limited in Mozambique and thus these must be sourced from third countries. Special care must be paid in selecting the minimum number and type of construction machinery by establishing fully examined construction network program.

(2) Source of Steel Pipe Pile

Procurement period of steel pipe piles will have a direct impact on the total construction period. Careful preparation of procurement plan including the delivery and transportation route will be indispensable for securing necessary construction period.

(3) Source of Stone Materials

Large amount of stone materials will be required in the Plan. The construction period is limited, and large batch of demand on stone materials is still difficult to be met in Mozambique. It is judged that procurement of stone materials from a third country, i.e. South Africa is most practical.

(4) Selection of Sand Source Site

For source of sand for filling of the mound, following three sites have been examined. As shown in Table 3.1-1, we have selected sand from site no. 3, considering quality, available volume and the transportation conditions. For filling material of sandbags, which will be used for protection of the existing revetment prior to the piling work, sand from the site no. 1 can be used.

Table 3.1.-1 Selection Criteria for Sand

Place	Quality	Distance	Remarks	Evaluation
1. Quelimane City	Pit sand (rather fine) (△)	Approx. 6km, surfaced (O)	Suitable for filling on land, but not in water	△
2. Zara Coast	Sea sand (fine sand) (×)	Approx. 20km, partly unsurfaced (△)	Clay content is low, but too fine for use in water	×
3. Licuare River	River sand (proper particle) (O)	Approx. 30km, surfaced, partly under improvement (△)	Distance is unfavourable, but road conditions generally good	◎

(5) Construction Work on Tidal River

The majority of the construction work for the erosion protection wall and a mound in front of the existing revetment are to be conducted under water. The maximum tidal difference reaches to 4.0m and river flow changes in fair and reverse directions at each tide, thus special attention should be paid to the tidal change during whole construction period.

(6) Influence to the Navigation Route

There is a navigation route in the area approximately between 40m to 150m offshore from the existing dock revetment. The construction work must be carried out with emphasis on safety to the navigation of vessels to and from Quelimane port.

3.1.3. Scope of Works

The Plan construction work will be executed around the existing facilities, and no major undertakings by the recipient country is expected. However, since the work area may neighbor with the navigation route, the relevant authorities should make a public notice to the navigating vessels with regard to the work area and expected period for the construction, as precaution measures against safety.

Issuance of necessary permits and approvals with respect to the construction work will not pose any problem. however it may be necessary to obtain confirmation of all concerned regional authorities in Quelimane on the area of the erosion protection structures to be constructed in the river. The detail design drawings etc. will be submitted to the Engineering Laboratory of Mozambique, Ministry of Public Works for approval.

Responsibilities assumed by both Mozambican and Japanese side for the implementation of the Plan are shown in section 3.1.7.

3.1.4 Construction Supervision

Following signing of the construction contract, the consultant, pursuant to its consultant contract with the Mozambican Government, will approve execution plan, shop drawings and conduct material inspections in Japan; and will dispatch civil engineers to the Mozambique during the main construction stages, such as setting of the work, start of erosion protection work and mound formation work. The consultant will also exercise construction control by dispatching for short periods, as required, engineering and facility supervisors and will conduct inspections, attend conferences, and provide guidance, while also maintaining close liaison with concerned agencies of the Mozambican Government as well as the Japanese Embassy and the regional JICA office in Zimbabwe. In addition, the consultant engineer will participate, as required, in factory and pre-shipment inspections of equipment and materials produced in Japan and third countries and will exercise close supervision to forestall equipment problems after arrival in the project site.

(1) Basic Concept of Construction Supervision

The consultant will supervise construction schedule to ensure that the construction work is completed within the prescribed construction period and will supervise and provide appropriate guidance to the contractor to ensure that the quality standards specified in the contract are secured and that the construction is carried out safely. However, due to characteristics of the work to be carried out on and underwater, and in the area of existing structurally unstable revetment, particular priority must be given to the safety of the construction work.

The following considerations should be borne in mind in connection with the main aspects of the supervisory process.

a) Safety of the Existing Structures

The Plan aims at prevention of collapse of the existing revetment. Thus the protection work for the revetment should be initiated before starting pile driving work, which will generate vibration and shock. The contractor shall make regular measurements on displacement of the existing revetment during the pile driving work to take necessary measures against any possible danger apprehended from the existing structure. The consultant is also requested to confirm the safety of the existing structure in collaboration with the representative of the contractor at the site.

b) Schedule Controls

The contractor will be required to submit monthly reports covering the following items, and the consultant with compare actual progress with the scheduled progress outlined in the contract. Should there be any indication if a delay in a particular phase of the work, a warning will be given to the contractor demanding that suitable corrective measures be taken, with ongoing guidance provided to insure that the work is completed within the construction period.

①Confirmation of the amount of construction work completed

②Confirmation of actual material equipment delivered

③Confirmation of actual number of engineers, foremen and workers

c) Quality Control

Supervision will be exercised, based on the following controls, to determine whether or not the quality of materials and equipment, as shown in the contract documents (such as technical specifications and the detail design documents) is being maintained by the contractor. Where quality control is found to be questionable, the contractor will be asked to correct, amend, or adjust such deficiencies.

①Examination of the mill certificates of steel materials.

②Examination and confirmation of quality of scour protection mat.

③Confirmation of unit weight, size and quality of stone material.

④Examination and confirmation of materials for concrete.

d) Quality Control of Workmanship

Since the majority of the construction work will be executed underwater, much difficulty will be expected in the inspection of the workmanship after completion in highly turbid water. Thorough discussion with the contractor before start of the work should be made to produce better workmanship. The contractor will be required to keep every record including photographs of construction work and procedures for the entire phase of the construction.

①Examination of the work execution plan

②Check of the shop drawings and actual workmanship

③Confirmation of points of care for each shop of the work

④Check of construction records by the contractor and confirmation of the work quality by reviewing these data.

e) Safety Controls

As the construction work will be carried out on the river, through consultations and cooperation with the site manager of the contractor, close field supervision will be exercised during the construction period to prevent casualties or accidents. The following points will be borne in mind in connection with field safety controls.

①Establishment of safety control regulations and appointment of safety administrators.

②Prevention of casualties through regular inspections of construction equipment.

③Designation of operating routes for construction vehicles and equipment and strict enforcement of slow driving rules.

④Providing welfare facilities for workers and strict enforcement of holiday rules.

(2) Construction Firms

The contractor will prepare execution plan and shop drawings for approval purposes in Japan, based on its construction contract with the Mozambican Government, and then carry out the procurement of equipment and materials. In order to complete construction in accordance with contract specifications within the construction period, the company responsible for construction work will dispatch construction supervisors to the project site throughout the construction period.

As supply of engineers and foremen will be realized in South Africa, thorough considerations must be made before deciding the manning plan for the work.

Based on the scale and nature of the Plan facilities, the number and categories of persons from the consultant and contractor that are to be dispatched to the Mozambique in connection with project implementation are shown in Table 3-1.3.

Table 3-1.-3 Number and Categories of Personnel

Categories of Personnel	No. of person (times)	Term	Responsibility
1. Consultant			
Supervising engineer	Five times	short period	project supervision
2. Contractor			
1) General manager (Civil Engineer)	1 (Japan)	whole construction period	overall site management & coordination
2) Technical staff (Civil Engineer)	1 (Japan)	ditto	construction work management
3) Coordination staff	1 (Japan)	ditto	construction work management
4) Technical staff	2 (Moz./SA)	ditto	assistance to site management
5) Operator	2 (SA)	as required in construction period	for heavy equipment
6) Mechanic	1 (SA)	ditto	for machinery general
7) Pile driving technician	4 (SA)	ditto	pile driving work

3.1.5. Procurement Plan

(1) Procurement Plan

It will be difficult to procure within Mozambique the principal materials and construction equipment required for this project. However, local supply will be feasible in the case of sand for filling, cement, and reinforcing bars.

Procurement plan for major materials and machinery are shown in Table 3.1.-4 and Table 3.1.-5.

Table 3.1.-4 Procurement of Key Materials

Main Construction Materials		Source
(1) Steel pipe piles	φ711 x 12 mm, L = 16,000 ~ 16,500 mm	South Africa
(2) Joints for above	PT type joint	Japan
(3) Steel sheet piles	IV type	Japan
(4) Rubbles for armor layer		South Africa
(5) Sand for fill use		Mozambique
(6) Reinforcing bar		Mozambique
(7) Cement		Mozambique
(8) Scour protection mat		South Africa
(9) Gabion basket		South Africa

Table 3.1.-5 Source of Principal Machinery

Main Items	Number	Source
(1) Crawler Crane 50 ton	1	South Africa
(2) Pile driving hammer CD45	1	South Africa
(3) Vibro-hammer	1	South Africa
(4) Flat barges:		
a) 300 TON	1	South Africa
b) 200 TON	2	South Africa
(5) Tugboat	1	South Africa
(6) Power generator		
a) 100 KVA	1	South Africa
b) 50 KVA	1	South Africa
(7) Excavator 0.6 m3	1	South Africa
(8) Bulldozer	1	South Africa
(9) Concrete Mixer 0.3m3	1	South Africa
(10) Crawler Crane 35 ton	1	Mozambique
(11) Truck Crane 20 ton	1	Mozambique
(12) Welders	3	Mozambique
(13) Truck 4 ton	1	Mozambique
(14) Forklift	1	Mozambique

(2) Transport Plan

The required equipment and materials will be transported to Mozambique (Quelimane) through routes as shown below and customs clearance at Quelimane port is possible for these materials.

①Ocean Transportation

a) Route in the Mozambique	Maputo → Quelimane
b) Route from third countries	South Africa (Durban) → Quelimane
c) Route from Japan	Japan→ South Africa (Durban) → Quelimane
	Japan→ Beira → Quelimane

②Land Transportation

Transportation from Quelimane port to the Project site will be made by land over approx. 3 km surfaced road.

3.1.6. Implementation Plan

The implementation plan for the subject project comprises the implementation plan, including tenders, along with engineering work, including piling operations for the protective wall and the formation of a mound, which are designed to protect existing facilities. The project site houses existing facilities that were completed in 1994 under a grant-aid from Japan, and so the basic infrastructure is now in place, including roads, power, and water supply to the project site, so that no problems are anticipated in this connection.

Manpower (including technicians) and materials and equipment will be difficult to obtain in Mozambique, requiring procurement from third countries. And, in view of the nature of the project and the need to comply with the designated construction period, in designing the implementation plan, ample time must be allowed in the procurement plan for assembling labor and equipment.

In preparing the implementation plan, the implementation stages have been grouped, according to the nature of the respective phases, into those that must precede the main construction, those that can proceed concurrently, and those that can be undertaken independently. After also taking into account the temporary facility plan, the procurement plan for materials and equipment, and construction costs, we have developed an optimum construction period. The contents of the main construction phases are as follows:

(1) Temporary/preparatory works	Procurement of equipment and materials for the main construction
(2) Protective works	Filling sand bugs into front portion of the existing revetment before piling
(3) Piling	Piling of steel pile and sheet piles
(4) Filling	Filling with stone and sand materials for forming a mound (including scour protection mat)
(5) Restoration works	Construction of slab in front of dock gate
(6) Repair works	Repair of superstructure concrete, etc.
(7) Appurtenant works	Installation of mooring buoy

Based on an evaluation of the above stages, the periods required for the implementation plan and construction work under this Plan will be as shown below:

(1) Implementation plan: 3.0 months

The implementation plan will include all stages from the signing of the consultant contract through preparation of tender documents, contractor Pre-Qualification for tenderers (PQ), tenders, and signing of construction contracts.

(2) Construction: 9.5 months.

The construction schedule, incorporating the above findings, is shown in Fig. 3.1.-1.

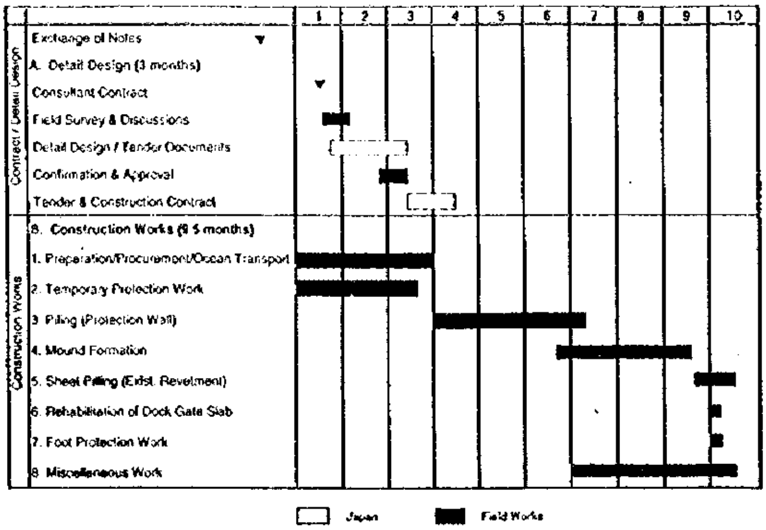


Table 3.1.-1 Construction Schedule

3.1.7. Obligations of Recipient Country

Share of responsibilities assumed by respective government is as shown in Table 3.1.-2.

Table 3.1.-2 Scope of the Project Responsibilities

	The scope of the project responsibilities	Japan	Mozambique
1.	Securing permits and approvals for construction activity in the river area as well as other permits and approvals required for Project implementation.		○
2.	Prompt customs clearance and payment of or obtaining exemptions from customs duties for all equipment and materials imported into Mozambique in conjunction with this Plan.		○
3.	Arranging exemptions from all local taxes and surcharges for the Japanese nationals and his sub-contractor performing project services in Mozambique.		○
4.	Issuance of authorizations to pay and payment of remittance charges, based on arrangements with the Japanese foreign exchange bank.		○
5.	Construction of a protective wall for the existing revetment, mound, and incidental facilities.	○	
6.	Implementation of monitoring operations following the completion of Project construction.		○
7.	Procurement of equipment necessary for carrying out monitoring operations.	○	

With respect to the construction responsibilities assumed by the beneficiary country, information will be provided to the Ministry of Agriculture and Fisheries, including the import item list and the list of machinery to be brought in and withdrawn, necessary to complete procedures for customs duty exemptions or appropriation of necessary budget. For the materials to be procured in Mozambique for the Plan project, no tax exemption will be expected since no such levies as value-added tax have yet been imposed in Mozambique.

3.1.8. Operation and Maintenance Costs

The Plan facilities, under existing conditions, comprise an erosion protection structure built either underwater or in the earth and, as such, will require no regular maintenance program. However, with the continuing riverbank erosion, the existing ground on both sides of the dock is falling, and at the stage when the top of the erosion protection wall becomes exposed, it is expected that there will be a need for a refill operation, with either sandbags or rubble, so that the slope between the dock structure and the protective wall is not scoured. Accordingly, this work should tentatively cost and allocated in equal portions to annual maintenance budgets.

The Plan anticipates that riverbank erosion at the Project site will continue at the present erosion rate for the next 25 years, and so we have calculated the maintenance costs that would be incurred if repairs are made by filling sandbags into the slope surface midway through this period -- i.e., after 13 years.

Table 3.1.-4 Total Maintenance Expenses over 13 Years

Item	Amount	Value (in millions of MT)
Sand for sandbags	3,800 m3	874.0
Sandbags	127,000 bags	438.2
Labor for sandbag production and filling	1,300 man-days	89.7
Total		1,401.9

The total repair costs for the slope surface, as shown above, have been estimated at 1.40 billion MT. If this amount were distributed equally over the 13-year period, the annual maintenance burden would come to 108 million MT.

Actually, since repairs of slope surface are required in accordance with riverbank erosion conditions, it is difficult to think in terms of generating equal annual payments for this purpose. Nevertheless, an annual maintenance assessment of 108 million MT would correspond to only about 4.3% of the total direct costs incurred by the Quelimane Fishing Boat Repair Facility during 1996 (2.52 billion MT). Considering the 610 million MT operating surplus, excluding depreciation, in that year, once the dock functions are fully restored, dry-docking revenues will resume, and, so long as future operations run as smoothly as in 1996, we believe that it will be feasible to defray maintenance costs for the slope without special budgetary provisions.

Chapter 4 Project Evaluation and Recommendations

4.1 Validation of Plan Appropriateness and Project Benefits

The Quelimane Fishing Boat Repair Facility has been functioning as a core facility supporting safe and efficient operations by fishing vessels in Mozambique, particularly those based in the middle and northern parts of the country.

Based on implementation of the subject Plan, the functions at this facility will be fully restored. The direct benefits that can be expected from this project will be as follows.

(1) Improving Operating Efficiency and Safety

Based on Plan implementation, fishing vessels based at Quelimane and points north will no longer incur the high fuel expenses that are presently required to sail to Beira, Maputo, or the nearest foreign repair yard in Durban, South Africa to undergo annual vessel inspections.

Fuel prices in Mozambique are higher than on the international market, reflecting the fact that the fuel must be imported, consuming precious foreign exchange. 29 large- and medium-size fishing vessels are based north of Quelimane, with a combined engine horsepower of about 19,000 ps, not including auxiliary engines. Assuming that, owing to the shutdown of the Quelimane repair facility, these vessels must now sail to Maputo to undergo inspection, the round-trip journey will consume 400 ~ 500 kl of fuel with main engines. Plan implementation will thus mean a significant saving in fuel costs for both main and auxiliary engines, contributing to higher operating efficiency.

(2) Contribution to the Regional Economy

Restoration of functions at the existing fishing boat repair yard will emphatically support the Mozambican Government's program for developing the fishing industry via medium-size vessels. With a total length of not over 20m and main engine horsepower of 300 ps or less, these boats encounter great difficulties in navigating to distant waters for the annual inspection, so that access to an inspection station near their home bases is a major factor in maintaining viable operations.

Based on the proximity of shrimp fishing grounds, shrimp trawlers are understandably the dominant vessel class among medium-sized fishing boats based at Quelimane, with 3 new shrimp vessels expected to enter the fishery in January, 1998, followed by another 3 ~ 6 within the next few years. Assuming that 5 vessels join the medium-size shrimp trawler fleet, this would mean employment for 120 new seamen, including reserve crews. This expanded fleet can be expected to spend 12 ~15 billion MT per year, thereby boosting total economic activity in the fishing sector by some 10%.

(3) Sustaining Resource Utilization

Large-size fishing vessels are allocated an annual TAC (Total Allowable Catch) of 6,500 tons for shallow-water shrimp, and their TAC has been completely consumed every year since the system was established. In terms of shrimp resource management policy, with the added catch effort from medium-size vessels, barring a major change in environmental factors at the reproductive stage, it can be anticipated that there will be a need to impose further controls on catch effort in this fishery. For some time now, the Fishery Research Laboratory has been considering the desirability of lengthening the closure period for shrimp fishing, starting on January 1 each year, from the present 60 to 90 days. While the industry is expected to resist such a proposal, based on the resulting fall in vessel operating rates, from the standpoint of sustaining resource utilization, the probability of realization is quite high. If the closure period is set at 90 days, based on Plan implementation, the number of fishing vessels able to enter for the dry docking inspections during this period can be expected to increase from 8 vessels in 1997 to at least 12 ~ 13 per year in the near future, which implies a corollary reduction in the number of vessels requiring dry dock services during the rest of the fishing year.

The only means of offsetting a decline in vessel operating rates as a consequence of the extended closure period would be to increase operating rates during the fishing season, which would certainly facilitate acceptance of the necessary closure period to maintain a stable resource.

4.2 Recommendations

Following are our recommendations to help solve two major problem areas related to the subject Plan.

(1) Monitoring Erosion Conditions

Based on Plan implementation, the target facility will develop substantial capabilities for coping with future riverbank erosion. However, owing to the limited amount of data on present river conditions as well as the special conditions existing on the river, where fair and back currents continually develop as a result of large tidal variations in the estuary area, it is still quite difficult to make accurate future projections on riverbank erosion at the Project site. For this reason, the Quelimane Dry Dock should inaugurate a program for measuring and monitoring changes on the riverbank and coordinating and analyzing the recorded data. The subject Plan includes equipment and a boat for measuring riverbed depth, along with guidance on the usage of this equipment, but it is the Quelimane Dry Dock itself, the implementing body for this Plan, which will have to take responsibility for organizing the data developed from observations and measurements into time series to facilitate judgments with respect to the progress of riverbank erosion, based on historical comparisons. But since specialized knowledge will be required to evaluate and analyze erosion conditions, it is vital that they be monitored on a regular basis, based on the monitoring manual that will be prepared at the stage of Plan implementation.

(2) Establishing a Suitable Operating and Maintenance System

Operations at the Quelimane Fishing Boat Repair Facility had been developing smoothly up to the suspension of dry dock operations in August, 1997, based on efforts by the implementing organization. In tandem with the future expansion of the fishing fleet in the Quelimane area, demand for repair facilities is projected to grow, and so the socio-economic need for restoring functions at this dock is considered to be of the highest order. In order to maintain the restored capabilities on a continuing basis after Plan implementation, a solid maintenance and administrative structure will be indispensable in terms of protecting slope surfaces on both sides of the dock from the encroachment of future erosion. Furthermore, in order to cope with the future expansion of fishing effort by medium-size vessels, investment in facilities to support optimum service to these vessels will, needless to say, be an imperative in facility administration. In order to take advantage of slack periods at the dock to achieve the capabilities required to meet this goal and spread the operational load as much as possible over the entire year, as opposed to relying just on a brisk demand for repair services, a sustained and positive effort is called for in establishing a sound and highly efficient management structure.