

CHAPTER 246

DESIGN AND CONSTRUCTION OF PLEASURE CRAFT HARBOUR **- CLUB MYKONOS LANGEBAAN**

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1. **ABSTRACT**

A description of the pre-design investigation, design and construction of a pleasure craft harbour is given. The influence of the breakwaters on the sedimentation pattern in the direct vicinity of the harbour during construction and after completion of the harbour is described. Adaption of the design of some of the harbour components due to unforeseen circumstances revealed during the construction period is also discussed.

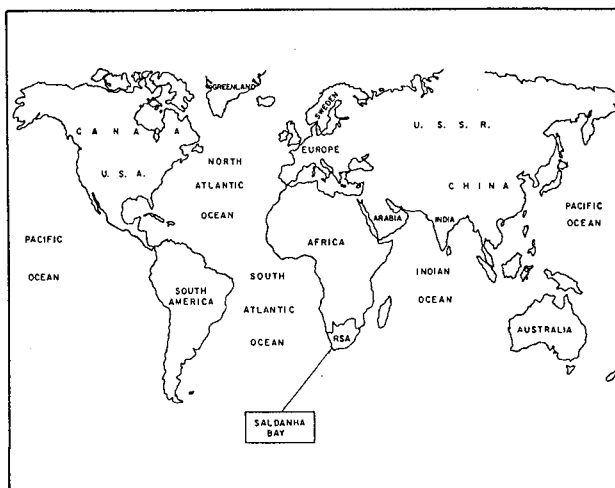


FIGURE 1 : LOCALITY MAP OF CLUB MYKONOS LANGEBAAN SMALL CRAFT HARBOUR AT SALDANHA BAY

2. **INTRODUCTION**

The Club Mykonos Langebaan Pleasure Craft Harbour Development is situated to the east of and in line with the entrance to Saldanha Bay on the West Coast of South Africa.

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In view of the similarity in climate and scenery of the South African West Coast and that of the Greek Island settlements in the Aegean Sea, the architectural and aesthetic model of the development was drawn from the latter - therefore the Greek name: Club Mykonos Langebaan.

It is the first privately owned pleasure craft harbour on the open coast of South Africa and forms the nucleus of an ultimately planned 100 million US Dollar private resort. The development includes a commercial centre (restaurants and shops), accommodation (single residential, timeshare, share block and sectional title), recreational, hotel and conference facilities. Figure 2 illustrates the proposed ultimate project.



FIGURE 2 : ILLUSTRATION OF ULTIMATE SCHEME

The harbour works were completed in 14 months during 1988/89 at a cost of about 4 million US Dollars. Figure 3 presents the layout and components of the harbour.

The harbour can accommodate yachts with draft up to 3 m and provides floating walk-on moorings for about 130 vessels. A boat ramp, launching jetty, fuelling and sewage pump-out facilities are also provided.

The basin size is approximately 4 ha of which a part had to be dredged to the required depth. Spoil from dredging has been used to create approximately 1,4 ha reclaimed land adjacent to the harbour basin.

The size of the harbour was determined mainly by the size of the land development from which the funds for it had to be generated. The layout of the harbour breakwaters was designed such that the harbour could be expanded towards the south if required in future.

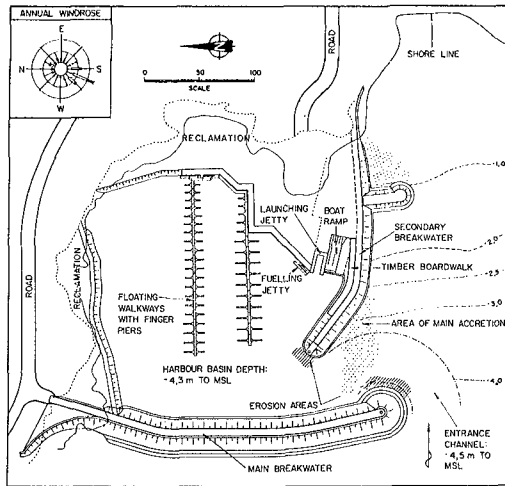


FIGURE 3 : HARBOUR LAYOUT AND COMPONENTS

For marketing reasons and since the harbour forms the nucleus of the development, creating a focal attraction point and therefore increases the market value of the adjacent land, it had to be completed before the land development.

To minimize time for which bridging capital had to be provided (i.e. the time during which expenditure exceeds income), selling of accommodation units on land had to commence soon after expenditure on construction of the harbour had started. This requirement from the developer's side necessitated a fast track programme for the design and construction of the harbour. The predesign investigations and design were done in about 4 months followed by a two months tender and contract adjudication period. The contract was completed in 14 months.

3. ENVIRONMENTAL CONDITIONS

Figure 4 indicates the location of the development in Saldanha Bay as well as the macro scale bathymetry, wind and wave conditions. The wind direction is predominantly from the South-Southwest sector.

The spring tidal range at the site is about 1,5 m.

Waves that directly penetrate Saldanha Bay in the direction of the pleasure craft harbour are reduced in height due to refraction to about 60% of their deepsea height. The 1:10 year deepsea significant wave height in the area is about 7 m which will therefore reduce to a significant wave height of 4,2 m at the harbour site. Peak wave periods vary between 11s and 13s.

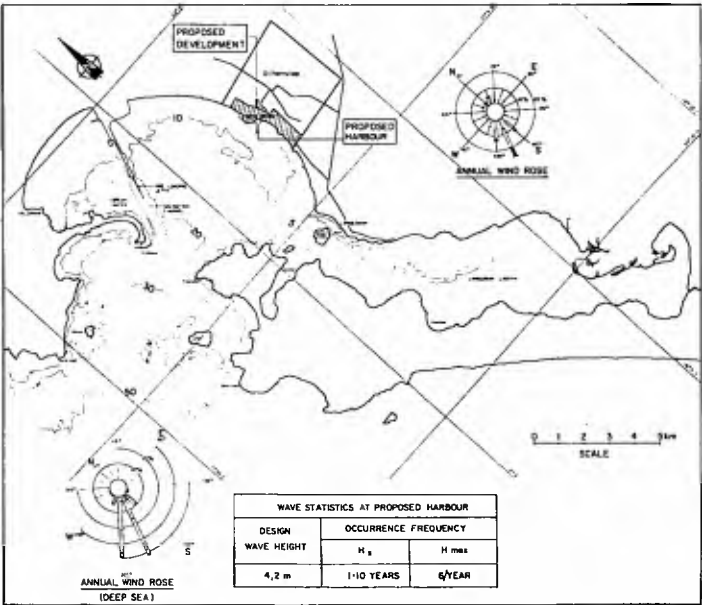


FIGURE 4 : ENVIRONMENTAL CONDITIONS

Figure 5 indicates the wave crest patterns in Saldanha Bay which is practically unidirection at the harbour site. Because of the latter, negligible longshore sediment transport due to wave action was expected. Tidal currents are reasonably high in the entrance to Langebaan lagoon (maximum of about 0.5 m/s) however the tidal current velocities at the Mykonos harbour site are very small.

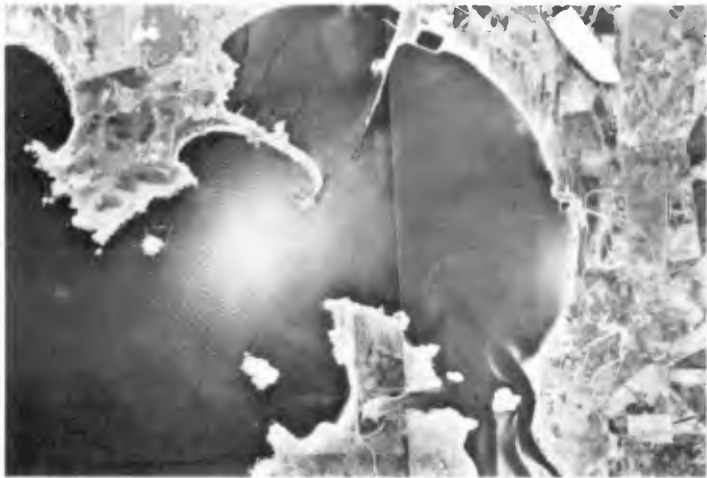


FIGURE 5 : WAVE REFRACTION PATTERNS IN SALDANHA BAY

The sediment on the seabed in the vicinity of Mykonos harbour is fine ($D_{50} = 140$ micron) and sediment movement due to wind-driven currents is possible.

Figure 6 indicates the hydrographic survey of the harbour site. Water jet probings were also done to establish rock depths. From the latter the landward limit of the harbour mooring area (after sand dredging) was derived.

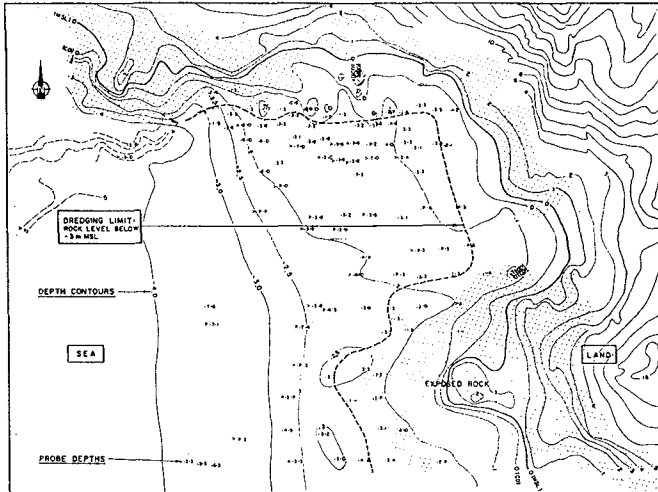


FIGURE 6 : HYDROGRAPHIC SURVEY OF HARBOUR SITE

Good quality granite rock was available from an existing quarry about 2 km from the harbour site.

4. DESIGN AND CONSTRUCTION OF HARBOUR

4.1 Breakwaters

4.1.1 Layout of Breakwaters:

A survey of existing yacht population on the South African coastline indicated the distribution of sizes as indicated in Table 4.1 below. The required mooring water depth for these sizes are also indicated in Table 4.1.

**TABLE 4.1 : LENGTH DISTRIBUTION OF YACHTS AND
REQUIRED MOORING WATER DEPTH**

Length	Percentage Distribution %	Required Mooring Depth below Mean Sea Level m
Up to 8 m	17	2,9
8 m - 11 m	40	3,9
11 m - 15 m	37	4,2
Longer than 15 m	6	

Based on the above boat size distribution, provision was made for 180 moorings of the walk-on floating jetty type anchored to concrete piles.

The following facilities were also needed:

Boat launching jetty

Boat ramp

Fuelling and sewage pump-out jetty

To provide for the above needs, a water area (including access channel and fairways) of about 4 ha was required.

The required water area together with the following factors determined the basic breakwater layout:

- o Landward limit of mooring area that could be dredged to the required depths as determined from the rock depth survey.
- o Required minimum depth at harbour entrance = 4,5 m below Mean Sea Level and entrance width of 45 m.
- o Provision for expansion of harbour towards the south.

The Phase I breakwater layout as optimised by means of three-dimensional hydraulic model tests together with a possible layout of an extended harbour (Phase II) is illustrated in Figure 7.

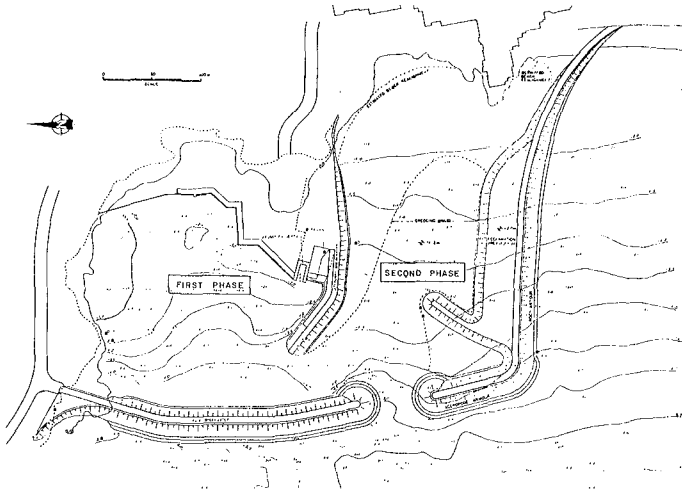


FIGURE 7 : BREAKWATER LAYOUT, PHASE I, WITH POSSIBLE EXTENSIONS, PHASE II

4.1.2 Main Breakwater:

The design conditions for the main breakwater were:

- Design water depth = 5,4 m
- Design wave height = 4,2 m (breaking wave)

The occurrence frequency of a significant wave of 4,2 m is 1:10 years, however considering the maximum wave in the wave spectra, breaking waves can occur approximately 6 times per year at the location of the main breakwater.

- Wave period = 11s to 13s.

Two basic alternative breakwater designs for the main breakwater (as indicated in Figure 8) were included in the tender to establish the cheapest solution. Stability tests on these were done in both two- and three-dimensional model tests.

The two alternatives were (refer Figure 8):

Alternative 1 : Berm type breakwater

Alternative 2a: Conventional rubble mound protected with Dolos armour units

Alternative 2b: Conventional rubble mound protected with Accropode units

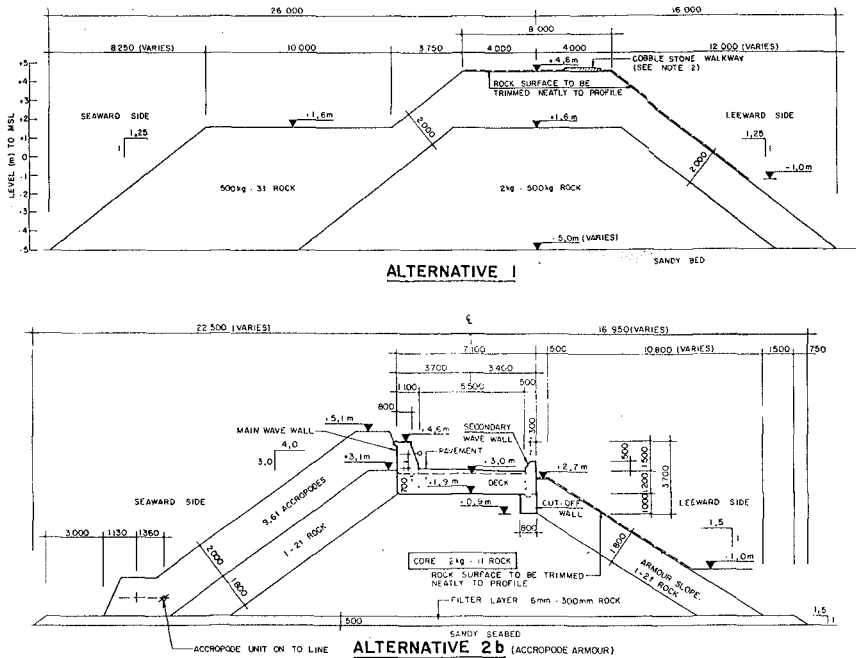


FIGURE 8 : ALTERNATIVE BREAKWATER DESIGNS

Alternative 2b protected with 9.6 t Accropodes was finally selected based on both cost and stability considerations. This design was also checked by Sogreah who holds the patent rights on the Accropode units. Although Alternative 1 proved to be the cheapest, it was not chosen because of the remote possibility of crest damage under extreme storm events. The latter problem could not be investigated in more depth due to the limited time available for the design period.

Some of the characteristics of the finally selected main breakwater design (Alternative 2b) are:

- o The crest level at +5.1 m above Mean Sea Level allowed some overtopping and the leeward armour slope is thus protected by means of a secondary wave wall.
- o The main breakwater comprises of 45 000 cub metres of rock, 1 400 Accropodes and 3 000 cub. metres of concrete capping and wave walls.

The main works on the main breakwater were completed in 5 months. It was subjected to breaking waves (i.e. the design wave height) for a number of times during the construction period. The wave action during one of these occasions are shown in Figure 9.



FIGURE 9 : BREAKING WAVES ON MAIN BREAKWATER DURING CONSTRUCTION

A precast and placed cut-off wall designed to be finally part of the concrete capping, rendered significant protection to the structure against breaking wave attack during the vulnerable construction stages. Figure 10 shows the effectiveness of the cut-off wall protecting the vulnerable crest during construction against breaching. The completed main breakwater with concrete capping, wave walls and Accropode protection is shown in Figure 11.



FIGURE 10 : ILLUSTRATION OF PROTECTION PROVIDED BY CUT-OFF WALL DURING CONSTRUCTION



FIGURE 11 : COMPLETED HARBOUR WITH MAIN BREAKWATER ON LEFT SIDE

4.1.3 Secondary Breakwater:

The secondary breakwater is also of the conventional rubble mound type with 1 to 3 t primary armour on the seaward side (see Figure 12).

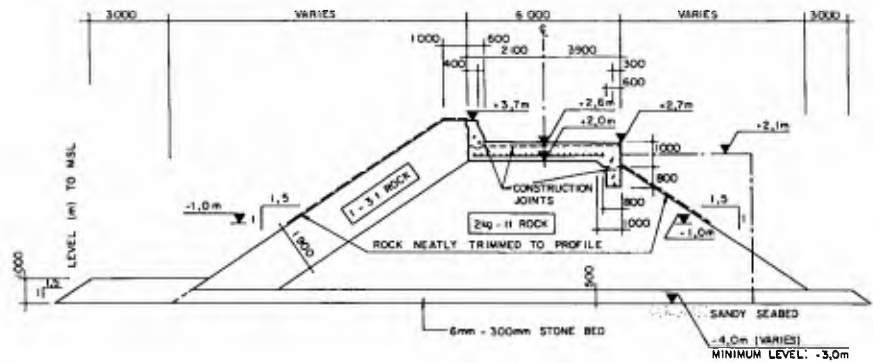
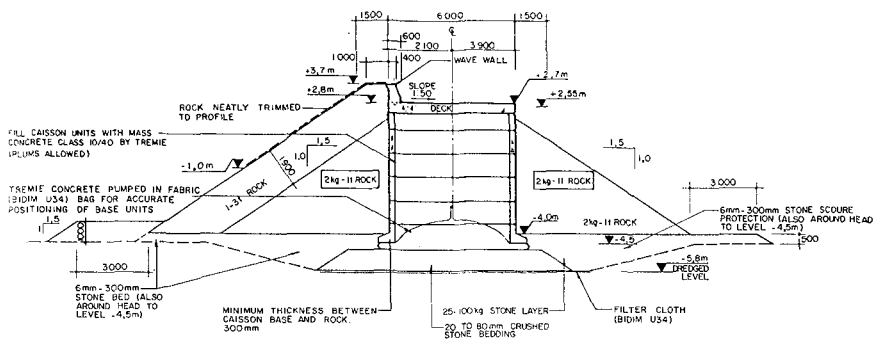


FIGURE 12 : SECTION THROUGH SECONDARY BREAKWATER

At commencement of construction of this breakwater it was found that the construction level (i.e. level at crest of rubble core at which the crane and trucks operated) had to be raised from the designed +1,4 m to +2,0 m above Mean Sea Level due to accessive damage experienced at the construction face.

Due to sediment accretion at the position of the secondary breakwater from the original -2,5 m contour and deeper (refer Section 4.1.4), the breakwater had to be founded on a higher level than the original seabed level. To prevent future undermining of the breakwater due to possible scour, a special toe protection on the seaward side was provided (see Figure 12).

The end of the breakwater comprises a 12 m caisson section built up of concrete box units filled with concrete after placement. The units were founded on a 1,3 m thick stone bed based at a dredged level of -5,8 m. The concrete box units were placed on aligned and levelled concrete pads with the void between the rubble foundation and concrete boxes being filled by tremie concrete contained in a filter cloth bag as shown in Figure 13.



**FIGURE 13 : CROSS SECTION THROUGH SECONDARY
BREAKWATER CAISSON END**

4.1.4 Local Sediment Accretion and Erosion Patterns during and after Construction of Breakwaters:

The main breakwater was constructed first to provide protection to construction operations of other components in its lee. The construction progress (in months) of the breakwaters is indicated in Figure 14; the main breakwater was completed in 4 months after which construction of the secondary breakwater commenced.

Construction of the secondary breakwater was delayed when about half of its length was constructed (at month 7 of breakwater construction - Figure 14) due to a significant amount of sediment accretion encountered at the construction face. An attempt to remove the sediment by grab crane so that the breakwater could be founded on the original seabed level was not successful since accretion took place at the same rate as removal.

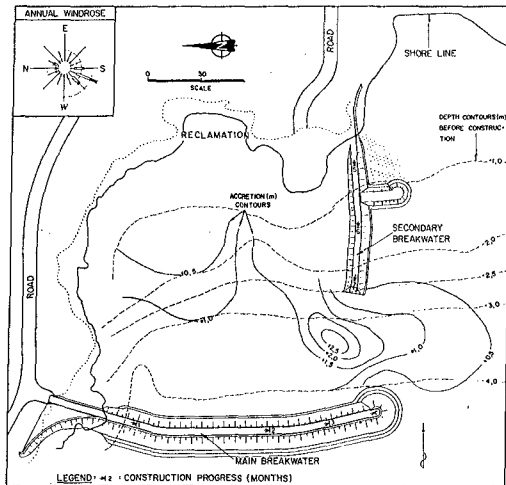


FIGURE 14 : CONSTRUCTION PROGRESS AND SEDIMENT ACCRETION DURING CONSTRUCTION

A hydrographic survey of the harbour area during this stage of construction indicated sand accretion that took place during construction as shown in Figure 14. Current measurements in the prototype during this stage indicated a rip current along the seaward side of the secondary breakwater causing sediment accretion in the area of the construction face.

Based on current measurements in the 3-D model of the harbour, it was decided to build a spur breakwater as indicated in Figure 14 to deviate the rip current so that the sediment accretion problem in the construction area as well as in the harbour mouth area could be limited.

The sediment accretion/erosion pattern 12 months after completion of the breakwaters is indicated in Figure 15. Erosion areas occurred at the end of both main and secondary breakwaters. Accretion occurred in the lee of the spur breakwater as well as in the harbour mouth area. The latter will necessitate periodic maintenance dredging estimated at about 15 000 m³/year.

4.2 Reclamation and Revetment

The reclaimed area (1,3 ha) as shown in Figure 3, was formed from 43 000 cub metres sand dredged from the harbour basin. The reclaimed area is edged with stone revetment at a 1:1,5 slope to reduce undesirable wave reflection. Where required, timber boardwalks mounted on concrete columns were provided for public access to the water's edges. Vertical guides fitted to these columns restrain the floating jetties installed next to the boardwalk. Figure 16 illustrates the revetment, boardwalk and floating walkways.

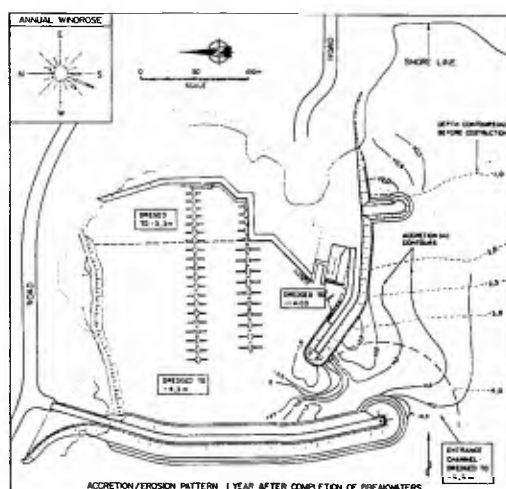


FIGURE 15 : SEDIMENT ACCRETION/EROSION AFTER CONSTRUCTION



FIGURE 16 : REVETMENT, BOARDWALK AND FLOATING WALKWAY

To prevent settlement of buildings founded on the reclaimed area, steep foundations of the buildings were founded on gravel piles (520 mm diameter; 19 mm stone; hammer compacted and spaced at 2 m intervals).

The timber boardwalk deck was constructed of untreated hardwood (Keruing) fixed with hot-dipped galvanized bolts and brass screws.

4.3 Floating Jetties

A floating walk-on jetty system comprised steelframed timber decking on polyethylene floats held by concrete gravity piles, was provided for boat moorings. The floats under the walkways were filled with polyurethane foam. The steel is protected after sandblasting with two zinc-rich base coats followed by two coats of polyachrothane paint. The floating jetty system is illustrated in Figure 17.



FIGURE 17 : FLOATING JETTY SYSTEM

Due to shallow rock substrata, concrete gravity anchor piles for the jetties had to be used. Three precast concrete rings, placed concentrically and the spaces between filled in situ, by tremie concrete, formed the gravity footing for the anchor piles (see Figure 18).

4.4 Launching Jetty, Boat Ramp

The yacht launching facility consists of two piers providing access for a mobile straddle carrier capable of launching yachts with a dry mass of up to 20 t. The two vertical walled parallel piers are constructed from precast concrete blocks on a rubble foundation.

A boat ramp is provided to enable the launching of motor boats on trailers with sufficient width to launch two craft simultaneously. The boat ramp consists of precast slabs on rubble fill below Mean Sea Level and in situ cast slabs above this level. The boat ramp slope is 1:8 and terminates at 2 m below Mean Sea Level.

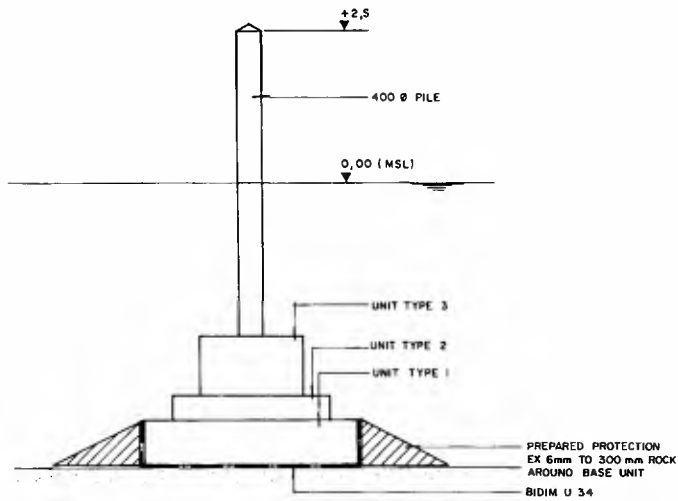


FIGURE 18 : CONCRETE ANCHOR PILES FOR FLOATING JETTIES

Figure 19 shows the launching facility, boat ramp and fuelling/sewage pump-out jetty.



FIGURE 19 : LAUNCHING FACILITY, BOAT RAMP AND FUELLING JETTY

5. **ACKNOWLEDGEMENTS**

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