2-3 Siltation

1) Objectives and Results of the Analysis

68. The objective of the study on siltation is to estimate the annual volume/rate of sediment deposited in Apia Harbour. The study flow of the siltation analysis is shown in Fig. 2.3.1. The main flow is represented by a bold line and the area enclosed by dotted lines is discussed in this chapter. The results are used for the port planning discussed in the Main Report.

69. The siltation volume in the turning basin, 126,000 m^2, is estimated as 9,500m^3/year, i.e. 7.5 cm/year, and the maximum siltation rate is estimated as 12 cm/year at the center of the turning basin.

2) Siltation in Apia Harbour

70. It is reported that there is a siltation problem in Apia Harbour. Apia Harbour has two sources of sediment, the Vaisigano River and the Mulivai Stream. The main source is the former, and the latter transports a smaller volume of sediment. Siltation in Apia Harbour is not a serious problem now because the maximum draft of ships calling at Apia Harbour is less than 9m and the depth of most parts of the turning basin is more than 10m. However, siltation will continue and the ship size of calling ships will become larger, so a siltation problem is expected to occur in the future.

71. Three reports describe the siltation problem in Apia Harbour. They are summarized as follows:

72. "Cruise Report No. 55, Apia Harbour Survey 1984, ESCAP"
The study team obtained only the drawings of this report. Judging from the comparison of two sounding charts, 1975 and 1981, contours are shifting towards the outer port of the harbour in the center of the harbour and the maximum shift of the 35 feet line is approximately 100m in 6 years. At the western inner part and near the northwestern end of the wharf, there is indication of minor scouring.

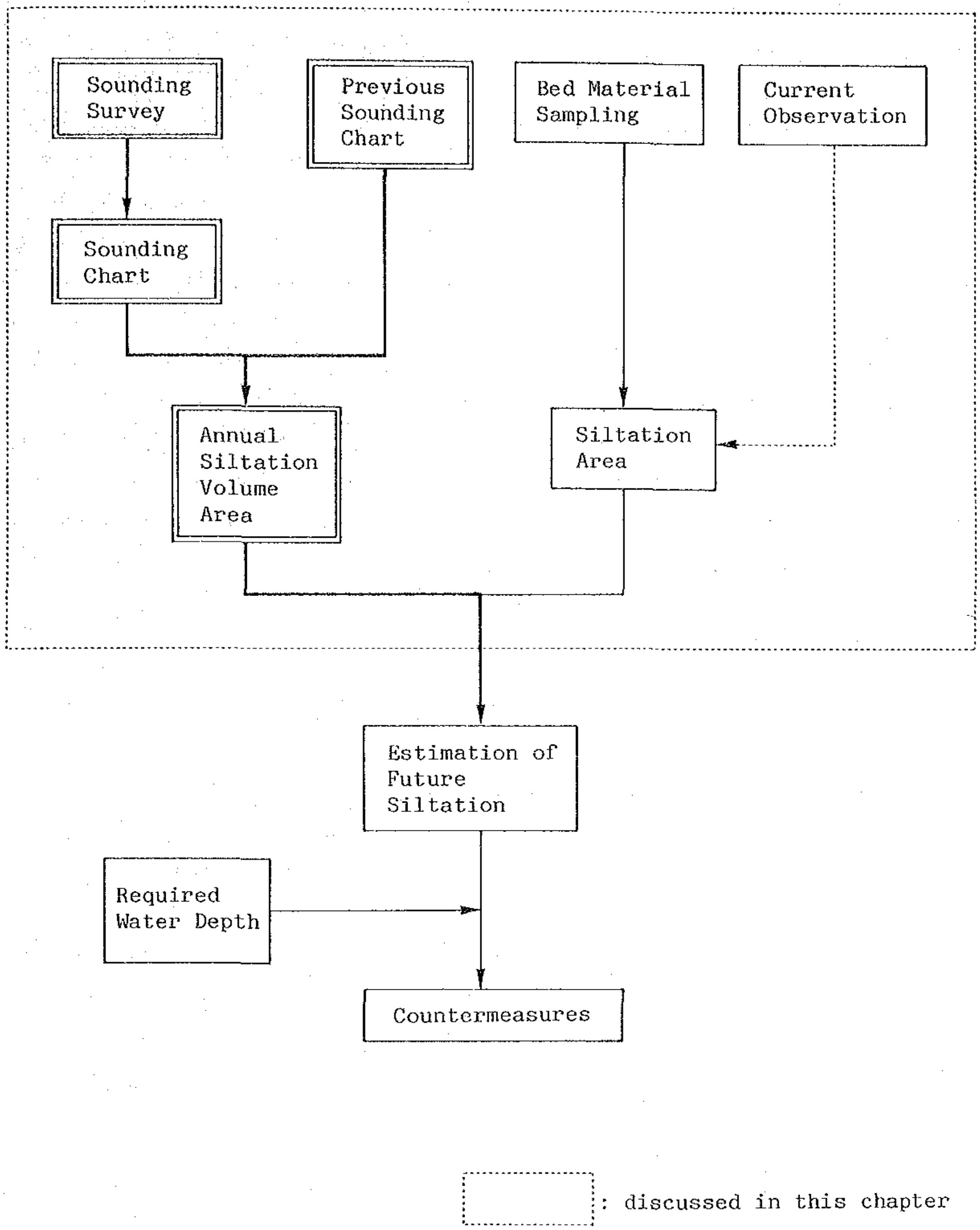


Fig. 2.3.1 Study Flow of Siltation Analysis

73. "Report on Siltation Problem and Desirability of Reclamation of Apia Harbour, 1983, ESCAP"
The author compared five Sounding Charts from 1975, '78, '81, '82 and '83 in five sections. Three lines are parallel and two lines are perpendicular to the Main Wharf. The estimated siltation rate is 1.8m in 15 years, i.e. 12cm per annum on the average.

74. "Land Fill Material and Harbour Surveys, 1984, ESCAP"
This report estimates the sediment volume discharged from the Vaisigano River. Judging from the soil profiles in the harbour, the author estimates the accumulated volume of sediment is 6,000,000m^3 in a 300,000m^2 area over 4,000 years, i.e. 1,500m^3 per year, and the shoaling rate is 0.5cm/year.

75. In order to estimate the siltation volume in Apia Harbour in the future, the team conducted a Sounding Survey, Current Observation and Bed Material Sampling.

3) Sounding Survey

(1) Comparison of Sounding Charts

76. A sounding survey was carried out from February 5, 1987 to February 11, 1987 using an echo sounder, PS-20 Kaijou Denki. The total area was approximately 400,000 m^2. Fig. 2.3.2 shows the sounding chart.

77. Fig. 2.3.3 shows the movement of contours compared with the sounding chart of 1981. The contours are compared in 1 meter intervals in the depth of 8m to 13m below Chart Datum. The siltation phenonmenon is clear at southern part of the harbour.

78. On the other hand, there is an indication of erosion in front of the wharf. This may have occurred mainly due to the frequent and strong agitation of sediment by ship propellers. And a tendency of erosion is also present at the west inner part of the port.

(2) Estimation of Siltation Volume in the Turning Basin

79. Generally speaking, siltation in a port is a serious problem. However, this is only a problem in areas of port activity such as the approach channel, turning basin and berthing areas. In Apia Harbour, the approach channel is deep enough and siltation will not be a problem in the near future. However with the large volume of sediment deposits in the turning basin and alongside the faceline of the wharf, a siltation problem is anticipated.

80. The team assumes the turning basin and berthing area is as shown in Fig. 2.3.4. Fig. 2.3.5 shows a sectional comparison between the sounding chart of 1987 and that of 1981. The calculated volume of sediment in this area is approximately 9,500m^3/year, i.e. the annual shoaling rate is 7.5cm/year, and the maximum shoaling rate can be read as 12cm/year at the center of the turning basin.

4) Current Observation

81. No reliable data exist on currents inside Apia Harbour, but according to the local mariners, these are very weak. Current observation was carried out using floats, 300mm in diameter, and they were tracked by two theodolites set on the control points on the Vaisigano Bridge and on the west reclaimed land. The observation was carried out on February 20 and 23, 1987, and the age of the moon was 7 and 10 days. The fastest currents during flood and ebb tide on each days were observed.

82. Fig. 2.3.6 shows the results of the observation. The currents during ebb tide in Apia Harbour are observed as follows:

East Side: 5 to 6 m/min, toward the outer harbour along with the main wharf

Center : 10 m/min, fastest towards the northwest

West Side: slow northward current

83. Even during the flood tide, currents to the north and norhtwest toward the outer harbour dominate and whirlpools and stagnations were observed at the west side of the harbour.

84. In order to obtain the data for hydraulic analysis of the siltation mechanism, further observations on such items as bottom current, suspended sediment concentration and flood volume are required.

5) Bed Material Sampling

85. Bed material sampling in Apia Harbour was carried out including points at the estuary and upstream of the Vaisigano River. The samples were collected by divers. Fig. 2.3.7 shows the locations of the sampling. Samples No.2 to 12 and WN, WC and WS are samples of the sea bed surface. Samples No. 21 to 24 were collected from 0.7 to 2.5m below the sea bed surface by divers using steel tubes. The sampling depths are as follows:

Sample No.	21	22	23	24
Depth below sea bed (m)	0.7	2.3	1.5	2.5

86. Sample No. 31 is dredged sand at the estuary of the Vaisigano River and No. 32 is a river bed sample collected 500m upstream from the Vaisigano Bridge.

87. The collected samples were tested for grain size, specific gravity and organic material content. Table 2.3.2 shows the results of the tests.

88. There are two clear distinctions on soil properties. One is the difference between No. 31 and No. 2, i.e. between the estuary of the Vaisigano River and the inner part of the harbour. The other is the difference between No. 4 and No. 5, i.e. between the approach channel and the turning basin. The materials are classified as river material, basin material and channel material. River material consists of gravel and sand, basin material is mainly silt, and channel material is silty sand.

89. There are clear difference of organic material content among them, i.e. 3 to 6 percent for river material, 13 to 17 percent for basin material and more than 17 to 29 percent for channel material.

90. There is also a clear difference of soil sampled along the wharf between the north end of the wharf and the center of the wharf. The

organic material content of W.N. is 5 percent higher than that of W.C. and W.S., and W.N. consists of mainly sand and gravel whereas the material of W.C. and W.S. is silt.

91. At the eastern part of the harbour, a clear difference of grain size is not seen. However, a difference of organic material content is evident between No. 10 and No. 11; No. 11 contains a high content of 28 percent.

92. There is an evident difference of sea bed material between the approach channel and the turning basin. Channel material contains a lot of organic material and basin material consists of sediment which originated in the basin of the Vaisigano River and contains less organic material.

Table 2.3.1 Sea Bed Material Tests

Semple No.	Organic Material Content (%)	Specific Gravity	Max	Grain Size (mm) 60% pass	30% pass	
2	17.2	3.004	0.420	0.03	0.0074	
3	16.6	3.051	0.840	0.03	0.0074	
4	13.3	3.095	4.760	0.11	0.075	
5	23.3	3.006	4.760	0.069	0.02	
6	23.3	2.973	2.000	0.069	0.006	
8	28.5	2.896	2.000	0.031	0.0094	
9	16.8	3.051	2.000	0.048	0.017	
10	17.4	3.003	9.520	0.035	0.017	
11	28.2	2.944	2.000	0.05	0.012	
12	21.7	2.985	4.760	0.1	0.03	
21	14.5	3.038	2.000	0.027	0.0096	0.7m below sea bed
22	13.4	3.010	0.420	0.023	0.0085	2.3m
23	21.9	3.009	2.000	0.038	0.012	1.5
24	14.9	3.044	2.000	0.07	0.016	2.5m
31	3.0	3.119	19.100	3.4	1.2	
32	6.6	3.117	19.100	2.5	0.83	
WN	23.9	3.017	19.400	0.92	0.21	
WC	17.3	2.966	2.000	0.05	0.013	
WS	18.0	2.985	9.520	0.031	0.0085	

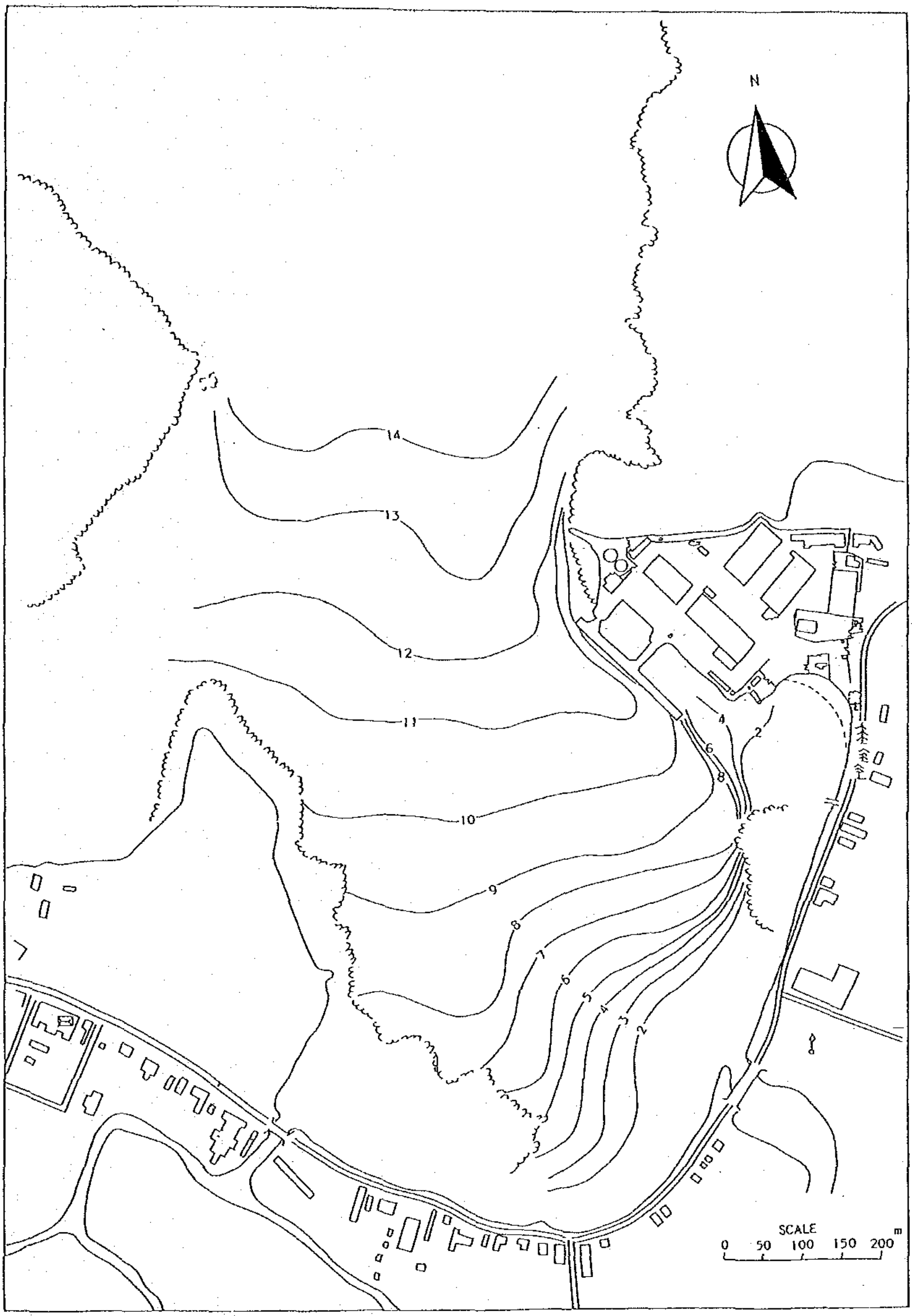


Fig. 2.3.2 Sounding Chart, Apia Harbour, 1987

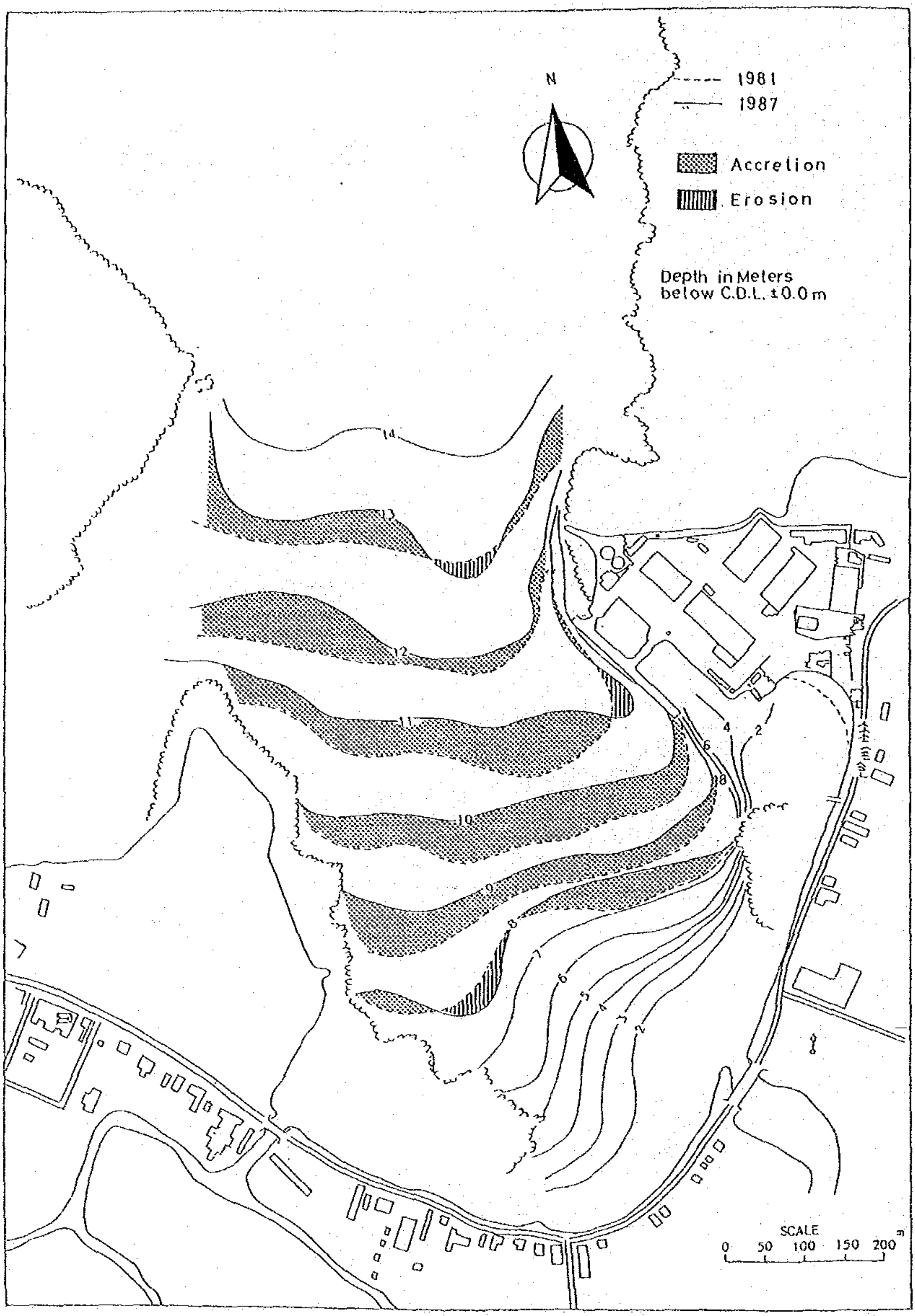


Fig. 2.3.3 Comparioson of Water Depths, 1981 and 1987

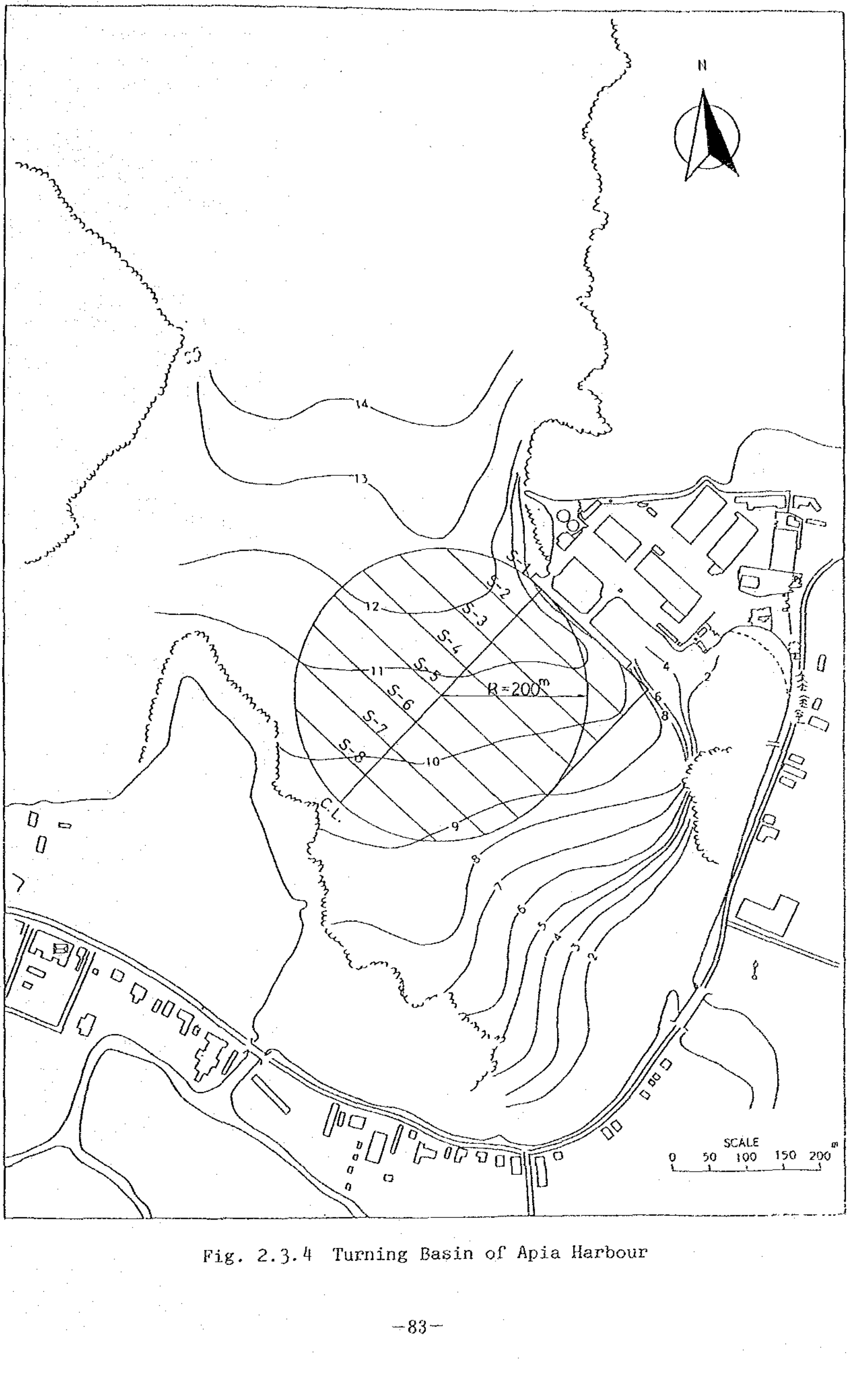


Fig. 2.3.4 Turning Basin of Apia Harbour

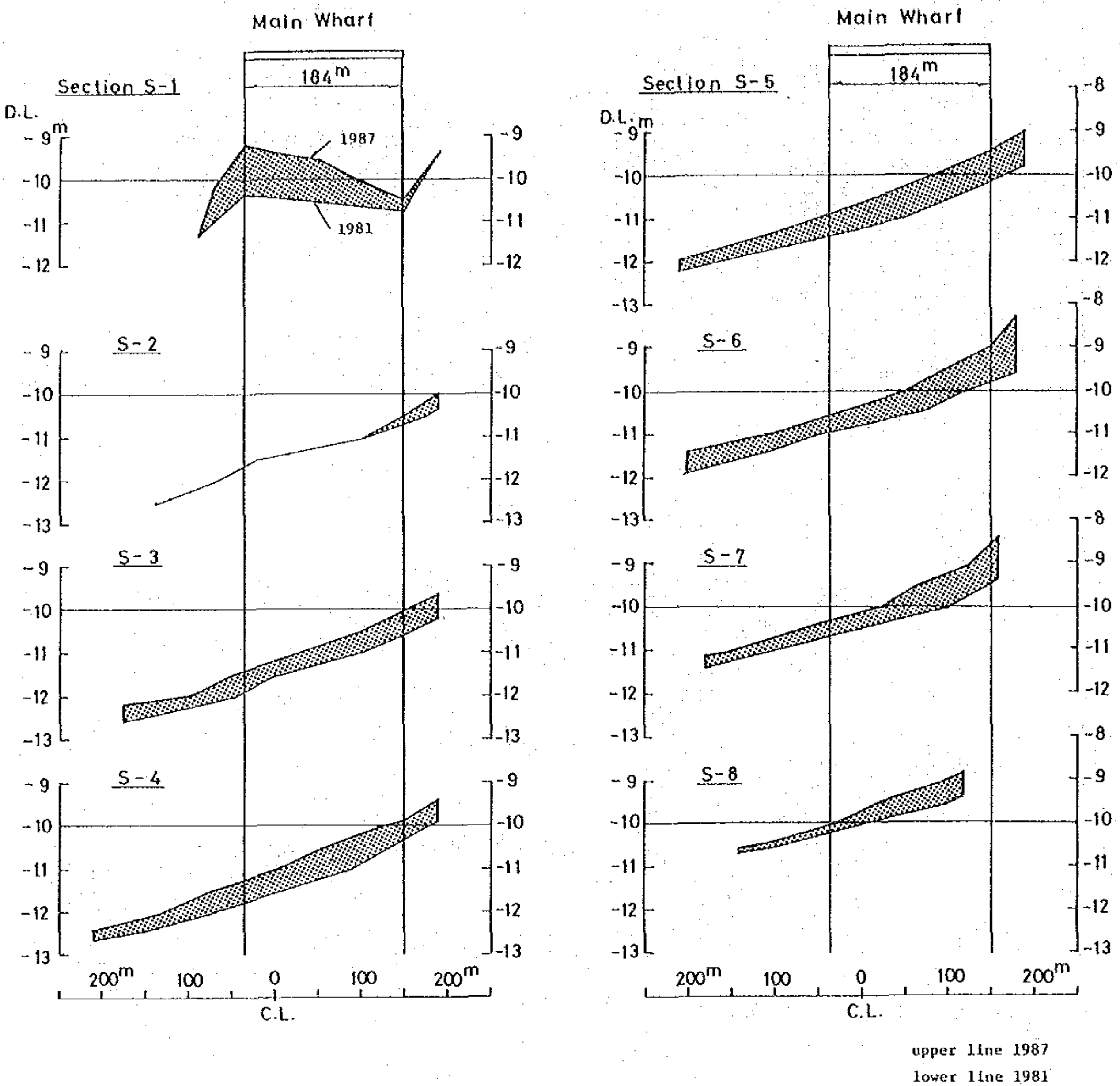


Fig. 2.3.5 Comparison of the Past Sounding Charts

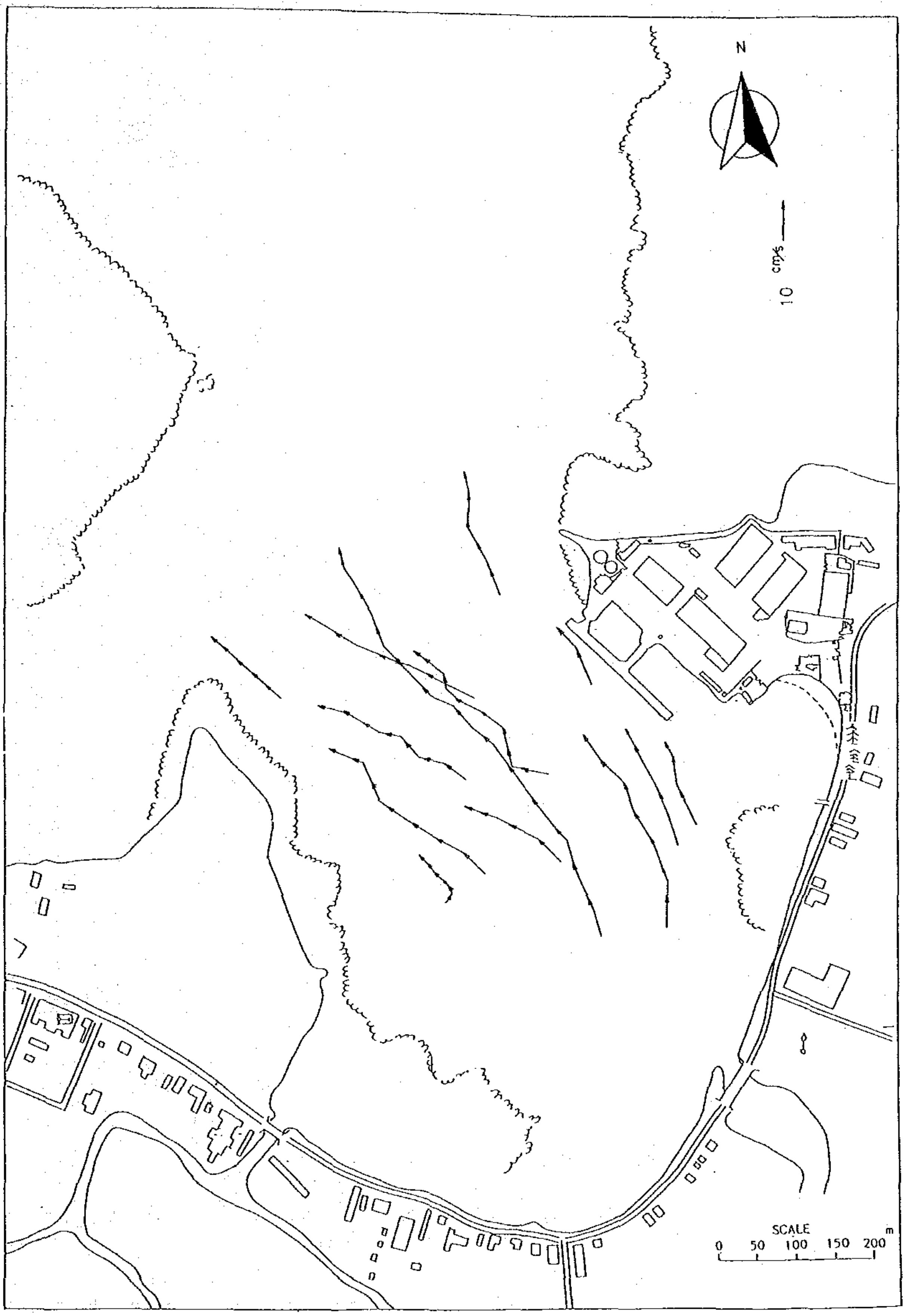


Fig. 2.3.6 (1) Results of Current Survey (Ebb Tide)

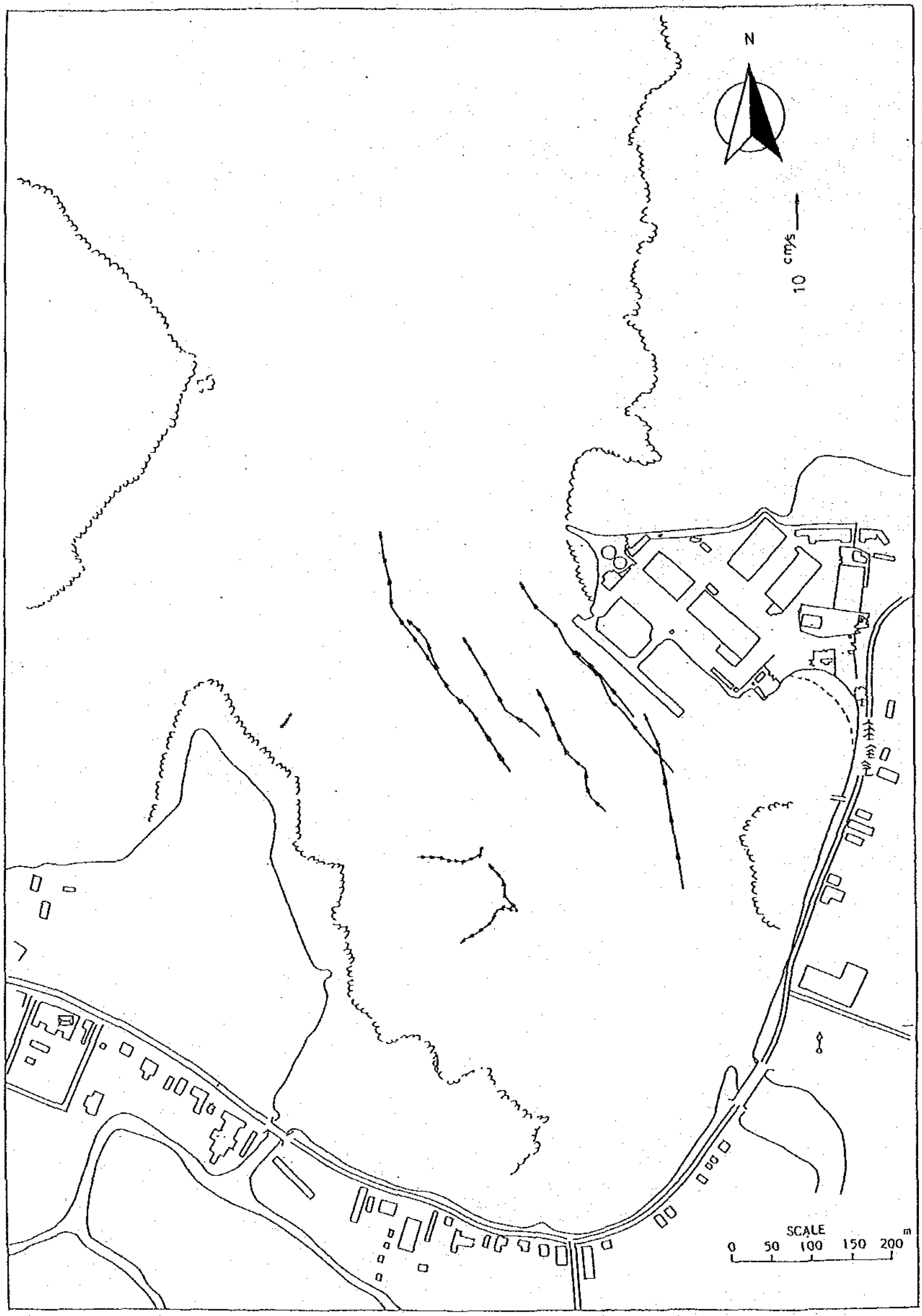


Fig. 2.3.6 (2) Results of Current Survey

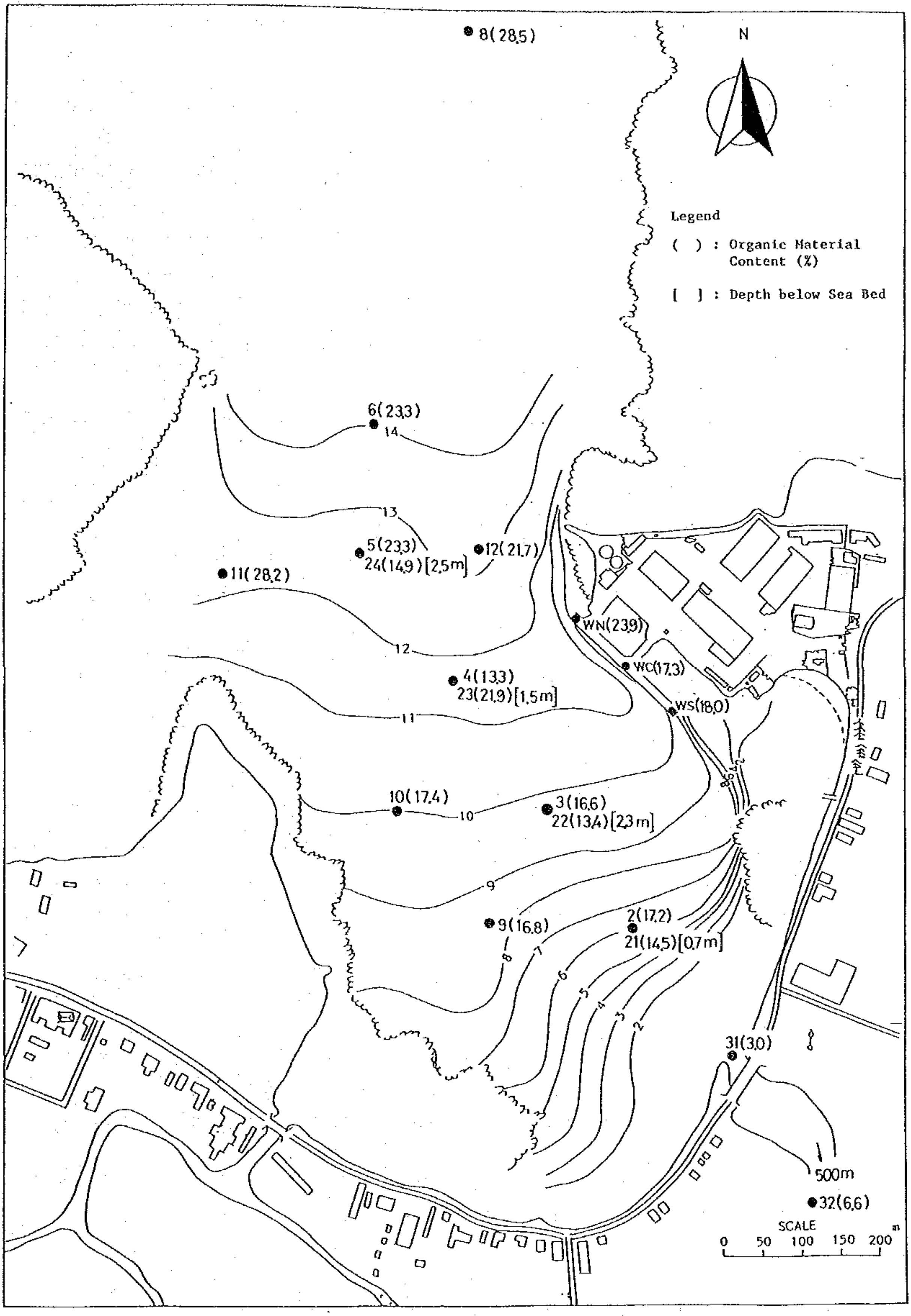


Fig. 2.3.7 Locations of Bed Material Sampling

2-4 Geotechnical Conditions

1) Outline of the Soil Survey

93. In 1963 and 1964, prior to the construction of the Main Wharf of Apia Port, a boring survey was carried out by the Department of Public Works of Western Samoa. The survey was conducted at 19 bore holes using a split spoon sampler 2" in diameter. In the drawing of the boring log, the N-value at some layers and the depth of the bearing layer are indicated. However, the data on mechanical and physical properties are not included.

94. The object of the new survey is to provide fundamental data on soil conditions to be used for basic design and improvement of the port structures. The soil investigation was carried out at two bore holes in order to obtain:

① information on the soil profile of strata and N-value,

② the physical and mechanical properties of the soil determined by in-situ and laboratory tests including data obtained by undisturbed soil samples, and

③ the bearing stratum up to the bed rock or hard stratum with an N-value of more than 50

95. Drilling, sampling and standard penetration tests (SPT) were carried out by a local consultant under the supervision of the study teams' geotechnical engineer. The soil samples including undisturbed samples were sent to Japan and laboratory tests were carried out.

2) Sounding and Sampling

96. The field work of soil investigation commenced at the beginning of February 1987 and was completed in the middle of March 1987 including an interruption of two weeks for the boring survey in Vaiusu Bay. Fig. 2.4.1 shows the location of the bore holes.

97. The items of the on-site soil investigation are as follows:

Bore Hole No.	Water Depth below MLWS (m)	Depth below Seabed (m)	SPT	Undisturbed Samples
A2	- 9.0	26	11	0
A3	- 9.0	25.5	8	3
Total		51.5	19	3

98. A rotary boring machine, model KR-50b Kano Boring Co., Ltd., was used for drilling. It was set on the main wharf and casings and boring rods were driven through the open space for the water supply pipe on the deck. SPT was conducted and three undisturbed samples were collected at bore hole A3 using a thin-walled sampler. The soil profile is shown in Fig. 2.4.2.

3) Soil Stucture

99. The soil profile of Apia Harbour was surveyed by ESCAP in 1984, and it is reported that the thickness of sediment is approximately 20m in the harbour and it has been deposited over 4000 years rising from sea level to its present elevation. The condition at the main wharf is the same as that shown in the boring logs.

100. The sediment stratum consists of two different materials. The upper layer is sandy silt transported by the Visigano River and it is very soft with an N-value of 0. Judging from old boring data, the thickness of this layer is estimated at 10 to 12 meters, and it was dredged from 1 to 6m for the construction of the Main Wharf. It is still in the process of accumulation.

101. The second layer is silty sand with a thickness of 13 to 17 meters and with an N-value of 1 to 40. The lower part of this layer contains coarse sand.

102. A thin coral layer with a thickness of 5 to 10cm is caught between these two layers. There is also a hard coral layer with a thickness of 0.5 to 3.5 meters on the bearing layer. The bearing layer is very hard volcanic basalt rock with a compressive strength of 300kg/cm^2 and more.

4) Laboratory Tests

103. The soil samples sent to Japan were tested.

The test items are as follows:

Test Item No.	Bore hole	Undisturbed Samples	Disturbed Samples	Total
Grain Size Distribution	A2 A3	0 3	3 2	3 5
Specific Gravity	A2 A3	0 3	3 2	3 5
Moisture Content	A3	3	0	3
Unconfined Compression Test	A3	3	0	3
Consolidation Test	A3	2	0	2

104. The results of physical tests are as follows:

Bore Hole	Depth below L.W.L. (m)	Specific Gravity	Grain Size (mm) Max	60%	30%
A2	16.5	3.019	2.00	0.023	0.0051
	26.0	3.027	2.00	0.021	0.0034
	31.0	3.022	2.00	0.022	0.0052
A3	16.0	2.987	4.760	0.023	0.0085
	20.0	2.945	4.4760	0.030	0.0011
	22.0	2.992	4.760	0.026	0.0095
	25.0	2.981	4.760	0.020	0.0054
	32.7		19.100	0.110	0.0370

105. The results of unconfined compression tests are as follows:

Bore Hole	Depth Below L.W.L. (m)	Specimen	r (g/cm^2)	w (%)	e	Sr (%)	Qu (kg/cm^2)	Eq (%)
	16.50-17.35	1	1.757	54.35	1.625	99.9	0.422	5.4
		2	1.781	53.14	1.568	101.2	0.322	3.9
	20.00-20.85	1	1.768	53.07	1.550	100.8	0.562	3.6
A3		2	1.749	52.69	1.572	98.7	0.582	2.1
		3	1.736	53.06	1.597	97.8	0.686	3.6
	22.00-22.85	1	1.748	56.98	1.686	101.1	0.806	1.9
		2	1.748	56.32	1.676	100.5	1.021	1.6
		3	1.762	56.20	1.652	101.7	0.810	1.2

106. Fig. 2.4.3 shows the distribution of shear strength.
The confirmed compression strength (qu) and cohesion (Cu) are as follows:

$$Cu = \frac{qu}{2} = 0.405Z \ (kg/cm^2)$$

where Z is the depth below the sea bed.

107. The results of the consolidation tests are as follows.

Bore Hole	Depth below L.W.L. (m)	Water Content W (%)	Volume Ratio f	Porous Ratio e	Degree of Saturation Sr (%)	Compression Index C_G	Yield Stress of Consolidation Po(kg/cm2)
A3	16.50 - 17.35	52.40	2.633	1.633	95.85	0.47	1.23
	20.00 - 20.85	52.29	2.575	1.575	97.77	0.47	1.40

108. Fig. 2.4.4 shows the log p - log Cv curve and the log p - log mv curve. The coefficient of consolidation (Cv) and the coefficient of volume compressibility are as follows.

Bore Hole	Depth below L.W.L (m)	Mv (cm^2/kg)	Cv (cm^2/sec)
A3	16.50-17.35	4.8×10^{-2}	2×10^3
	20.00-20.85	3.0×10^{-2}	1.2×10^3

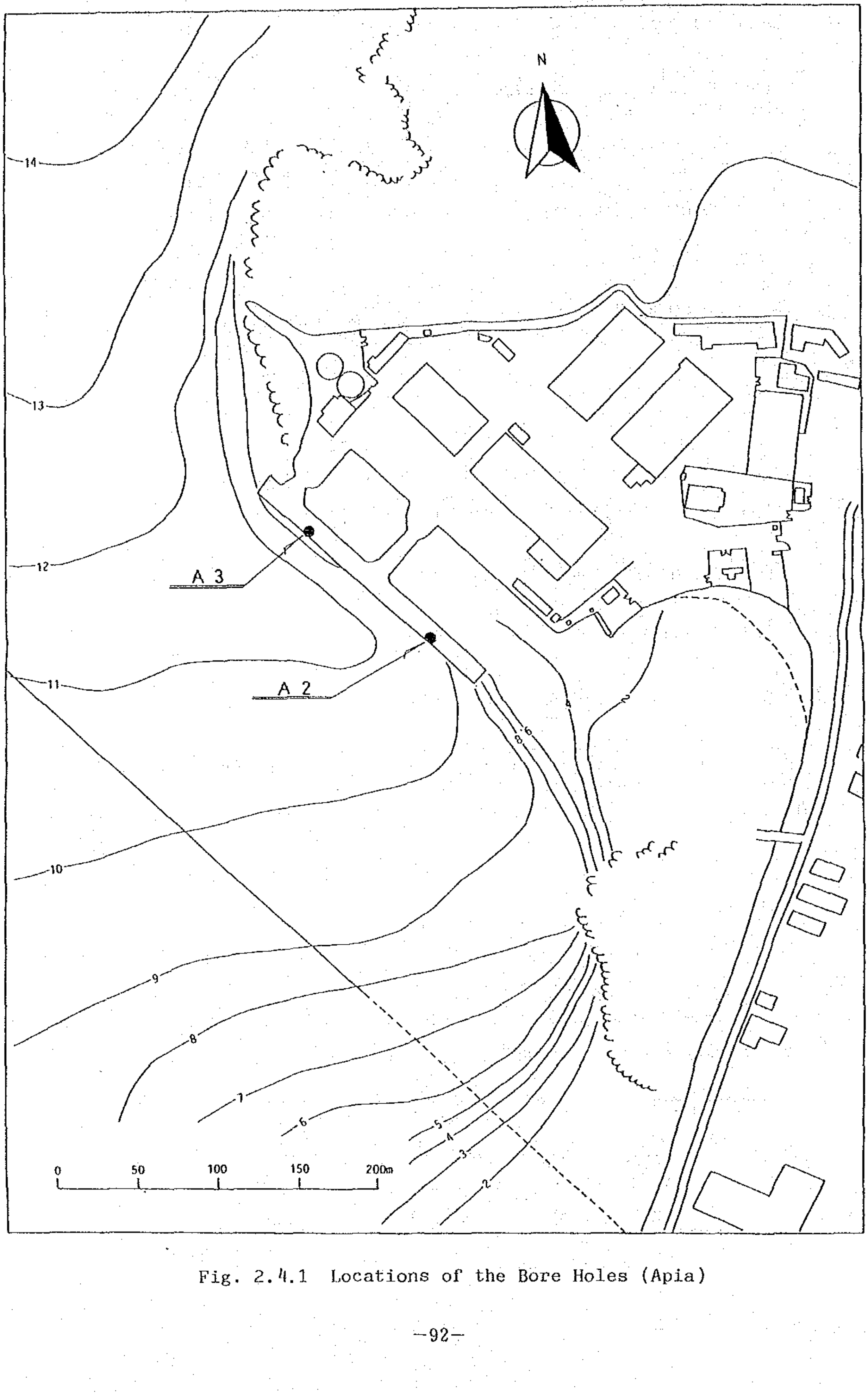


Fig. 2.4.1 Locations of the Bore Holes (Apia)

Bore Hole No. A2 (Apia)

Depth below L.W.L. (m)	Depth below Sea Bed (m)	Thickness (m)	Soil Symbol	Soil Classification	Colour	In-place Observation	Sample No.
-9.00	0	0		SEA BED			
-16.80	-7.80	7.80		SANDY SILT	DARK GRAY	fine sand with sedimentary mud	A-1
					DARK GRAY	silty fine sand	A-2
							B-1
							B-2
							B-3
-26.30	-17.30	9.50		SILTY SAND	GRAY	silty sand	B-4
							C-1
-29.90	-20.90	3.60		COARSE SAND	DARK GRAY	coarse sand with shell fragment	C-2
							D-1
-33.50	-24.50	3.60		CORAL	WHITE	coral	D-2
-35.00	-26.00	1.50		ROCK	BLACK	basalt	D-3

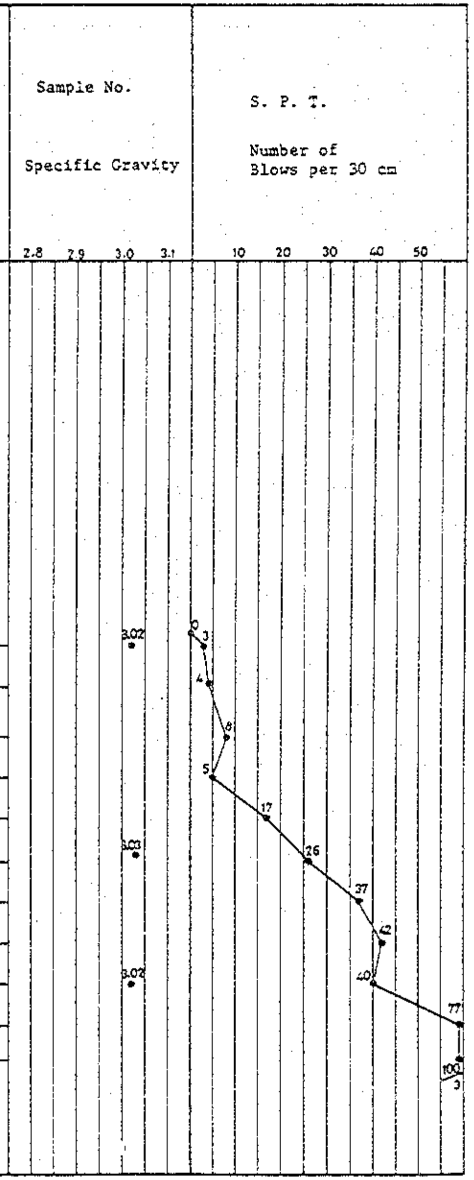


Fig. 2.4.2 (1) Soil Prafile (A2)

Bore Hole No. A 3 (Apia)

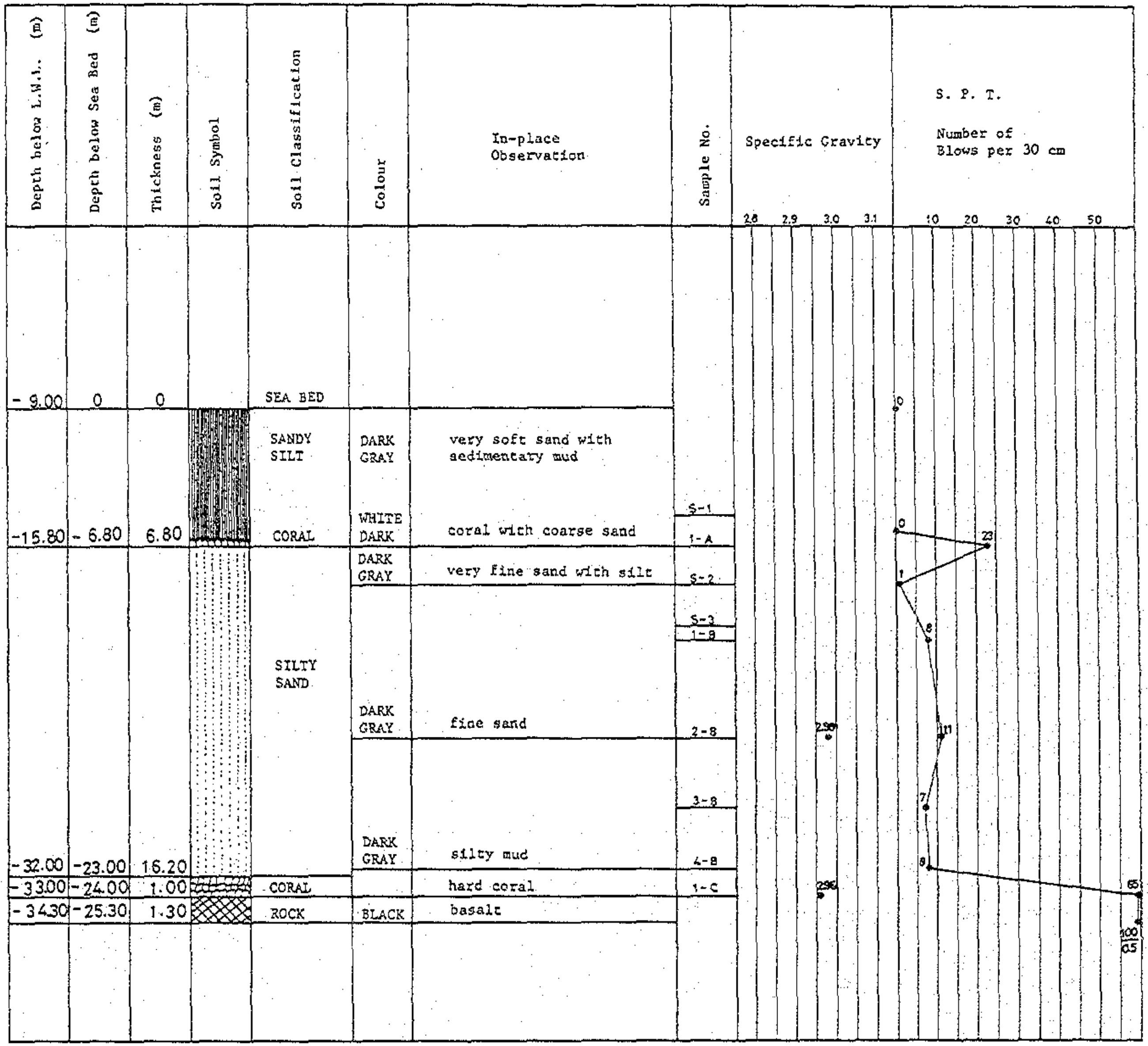


Fig. 2.4.2 (2) Soil Profile (A3)

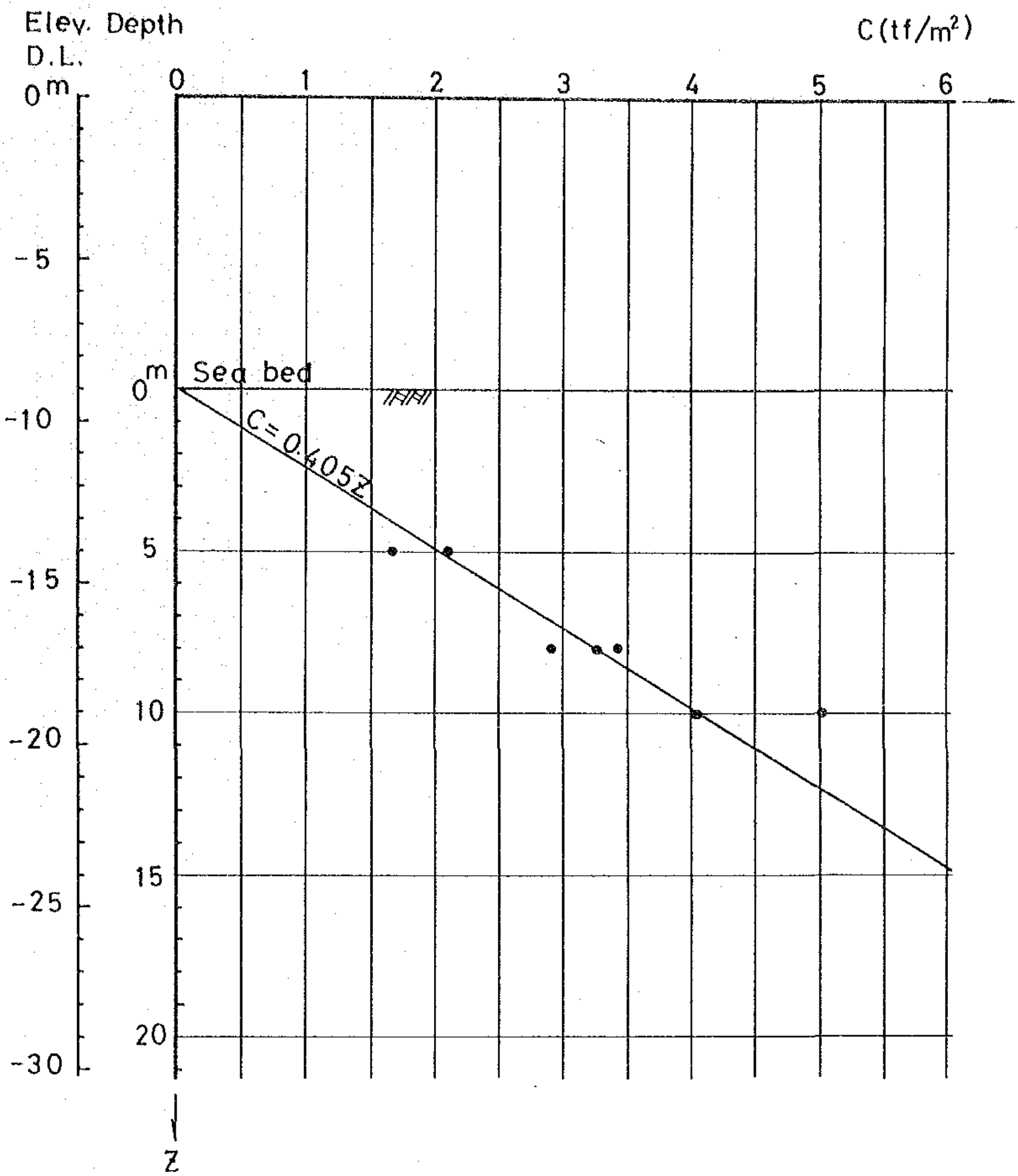


FIg. 2.4.3 Cohesive Strength

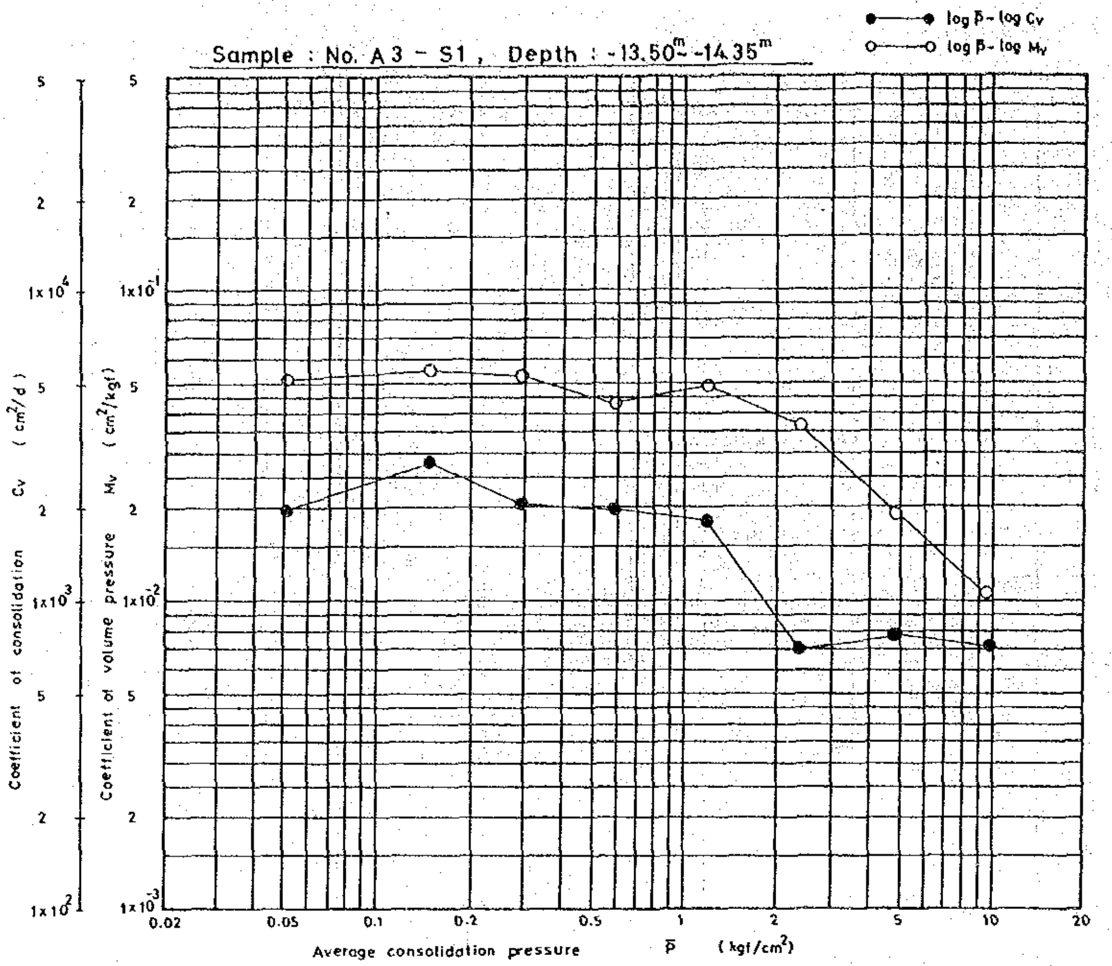


Fig. 2.4.4 (1) P - Cv, Mv Curve

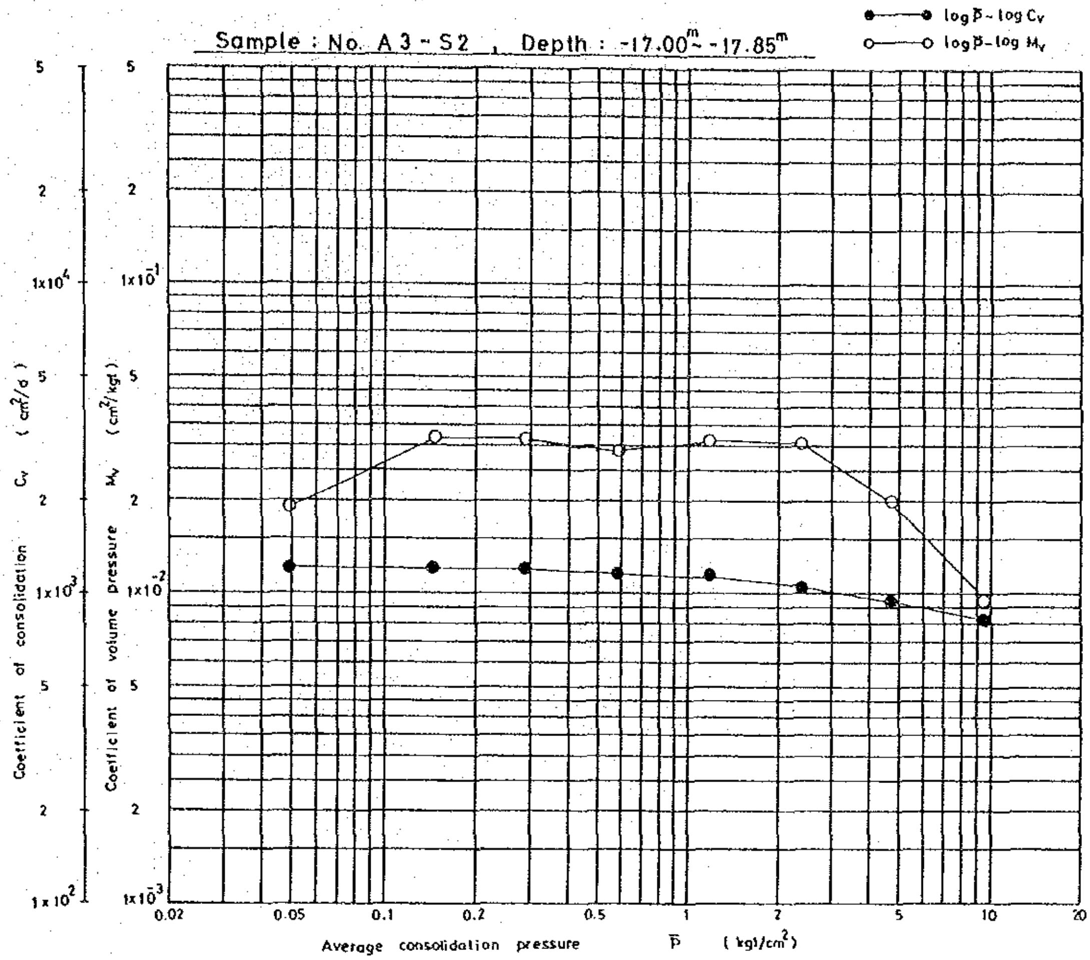


Fig. 2.4.4 (2) P - Cv, Mv Curve

CHAPTER 3
NATURAL CONDITIONS SURROUNDING ASAU HARBOUR AND VAIUSU BAY

Chapter 3 Natural Conditions at Asau Harbour and Vaiusu Bay

3-1 Asau Harbour

1) Sounding Survey

1. A sounding survey was carried out at Asau Harbour using an ultrasonic echo sounder from February 27, 1987 to March 6, 1987. The sounding area was about 5,000,000m^2. A tide gauge was set on a pile of the wharf. Three control points were set on the west corner of the wharf, the north corner of the saw mill camp's jetty and the east end of the airstrip.

2. The inside of Asau Harbour is well protected from waves by the coral reef. However, the waves at the northwest part of the entrance channel are rather rough in the wet season because of the dominant northeast wind. In the period of the survey the team tried sounding at the channel, but this was not possible because of rough waves. At the northward entrance of the channel, waves 1 to 2 meters in height were recorded on the recording chart of the echo sounder, and waves more than 3 meters in height were observed visually. An area of turbulent water was observed towards the northward end of the channel.

3. Fig. 3.1.1 shows the sounding chart inside of the harbour.
The minimum depth along the faceline of the wharf is 8.2m below chart datum at 30m eastward from the west end of the wharf. The water depth of the other port of the wharf is deeper than 10m.

4. Two shallow areas are shown near the wharf. One is located about 100 meters southward from the east end of the wharf and the minimum depth is 8m below chart datum. The other is located about 300 meters southwestward from the west end of the wharf, and the minimum depth is 5.3m.

5. The approach between the southern end of the entrance channel and the wharf has a depth of more than 12m. The center area of the harbour, about 400,000m^2, has a depth of more than 15m and a maximum depth of 17m.

2) Current Observation

6. Currents during ebb tides in the entrance channel were observed using floats on March 5, 1987. Two theodolites set on the eastern end of the embankment and on the sand bank located on the western side of the channel were used for tracking the floats.

7. Fig. 3.1.2 shows the results. Current velocity at the northwestern end, the center and the southeastern end of the channel are as follows:

	SE End	Center	NW End
Ebb Tide (cm/sec)	10 - 20	30 - 50	30 - 60

8. The currents concentrate to the center line of the channel and diffuse at the northwestern end of the channel. At the outside of the channel, westerly currents caused by northeasterly waves were observed.

3) Bed Material Sampling

9. Bed materials were collected at two points. The locations are indicated on Fig. 3.1.1. Sample S1, collected at the wharf 2m below sea bed, consists of white gray silt containing fragments of shell and coral. Sample S2, collected at the southern end of the sand bank, consists of gray coral sand.

10. The results of the physical tests are as follows:

Sample No.	Water Depth(m)	Depth below Sea Bed (m)	Specific Gravity	Grain Size (mm) Max	60%	30%	10%
S1	11.0	2.0	2.801	9.520	0.022	0.115	0.0032
S2	1.0	1.2	2.812	9.520	0.4	0.29	0.18

4) Tide

11. The mean high water spring tide at Asau Harbour is +1.2m above chart datum. In the period of the survey, a high tide of +1.76m was observed at 21:10 on February 28, 1987 under a new moon. This anomalous high tide was caused by the approach of a low atmospheric pressure.

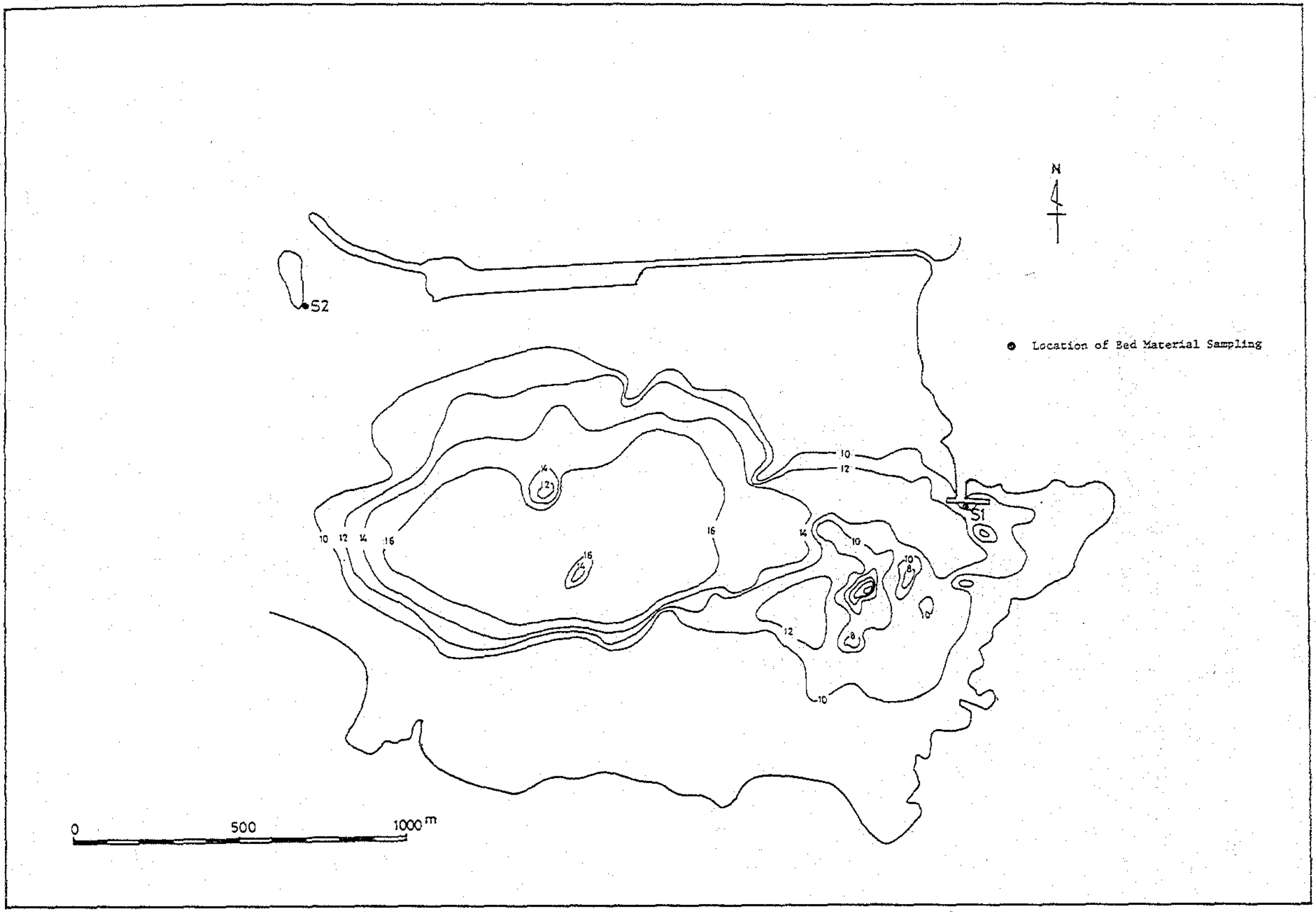


Fig. 3.1.1 Sounding Chart, Asau Harbour, 1987

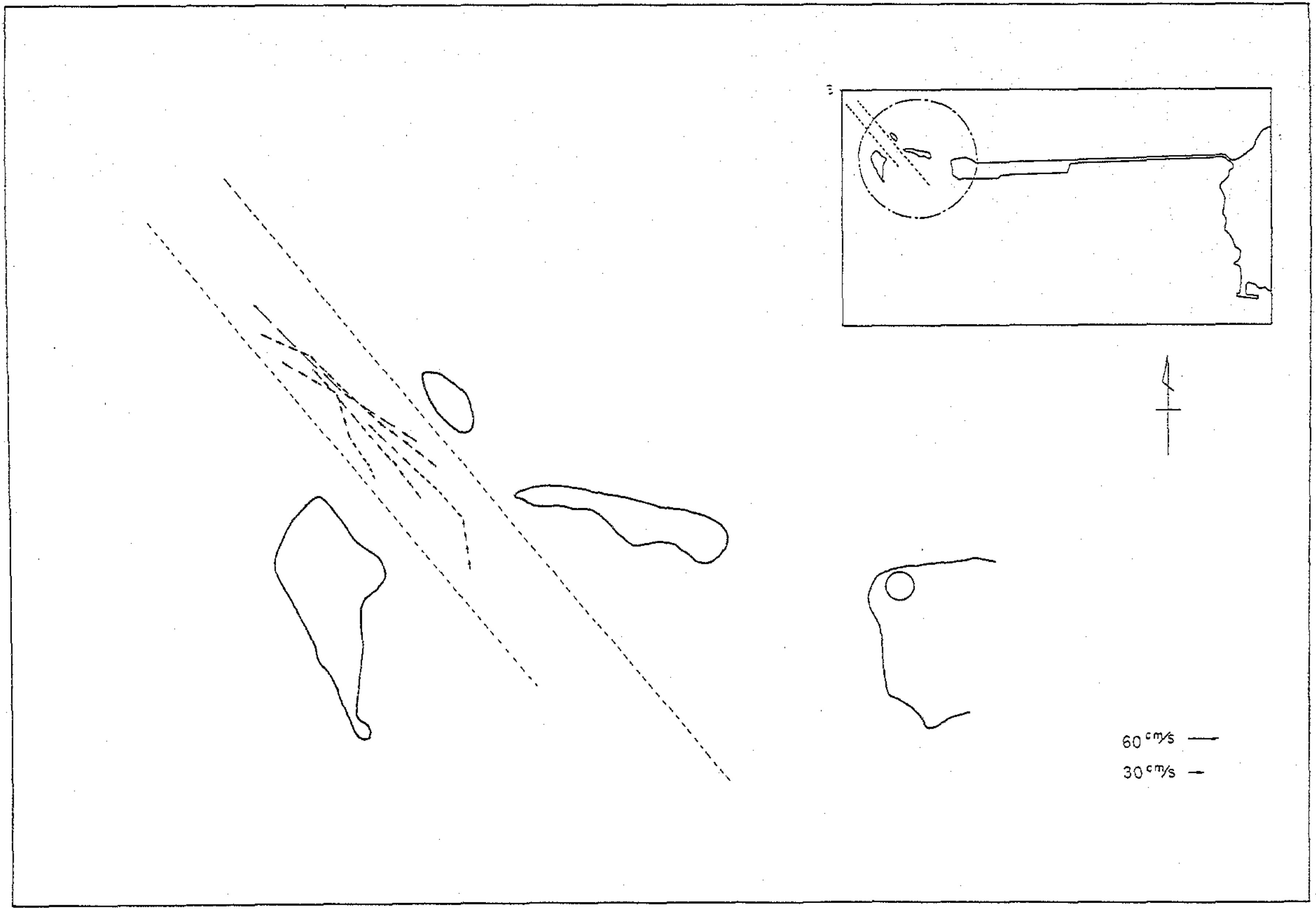


Fig. 3.1.2 Results of Current Survey (Ebb Tide)

3-2 Vaiusu Bay

1) Sounding Survey

12. A sounding survey was carried out from February 16, 1987 to February 18, 1987. The total area was approximately 3,500,000m^2. Two control points were set, one on the tip of the Mulinu Peninsula and the other on the causeway for dredging of coral sand.

13. Fig. 3.2.1 shows the sounding chart. The east and north sides of the area are surrounded by coral reefs. The inside of the coral edge is filled with coarse coral sand and is well protected from waves. The water depth falls in a very narrow range of 0 to -1m and is almost flat.

14. A deep inlet with a length of approximately 600m in the east to west direction and a width of approximately 200m in the north to south direction is shown in the southern part of the eastern reef. The edge of the reef is very steep and most of the area is deeper than 15m.

15. Currents with the speed of approximately 1m/sec. were observed during ebb and flood tides at the coral edge of inner part of the inlet.

2) Boring

16. A boring survey was carried out at the westward side of the inner part of the inlet. The location of the bore hole is shown in Fig. 3.2.1. The surface of the lagoon is covered by coarse coral sand, and coral lumps of 30 to 100cm in diameter lie throughout the area.

17. Fig.3.2.2 shows the boring log. The subsoil consists of mainly silty coral sand. From the results of SPT and grain size test, the soil is divided into two strata at about 15m below sea bed.

18. The upper layer is loose coral sand with an N-value of zero to 3; especially the layer up to 5m below sea bed has an N-value of zero. This layer contains coral fragments, shells and silt. The lower layer has an N-value of 9 at 17.3m below sea bed and consists of coarse coral sand and

gravel.

19. The results of the physical tests are as follows:

Depth Below Sea Bed	Specific Gravity	Grain Size (mm) Max	60%	30%
7.3	2.844	19.1	1.8	0.05
11.3	2.923	19.1	2.3	0.02
17.3	2.923	25.4	15.0	4.1

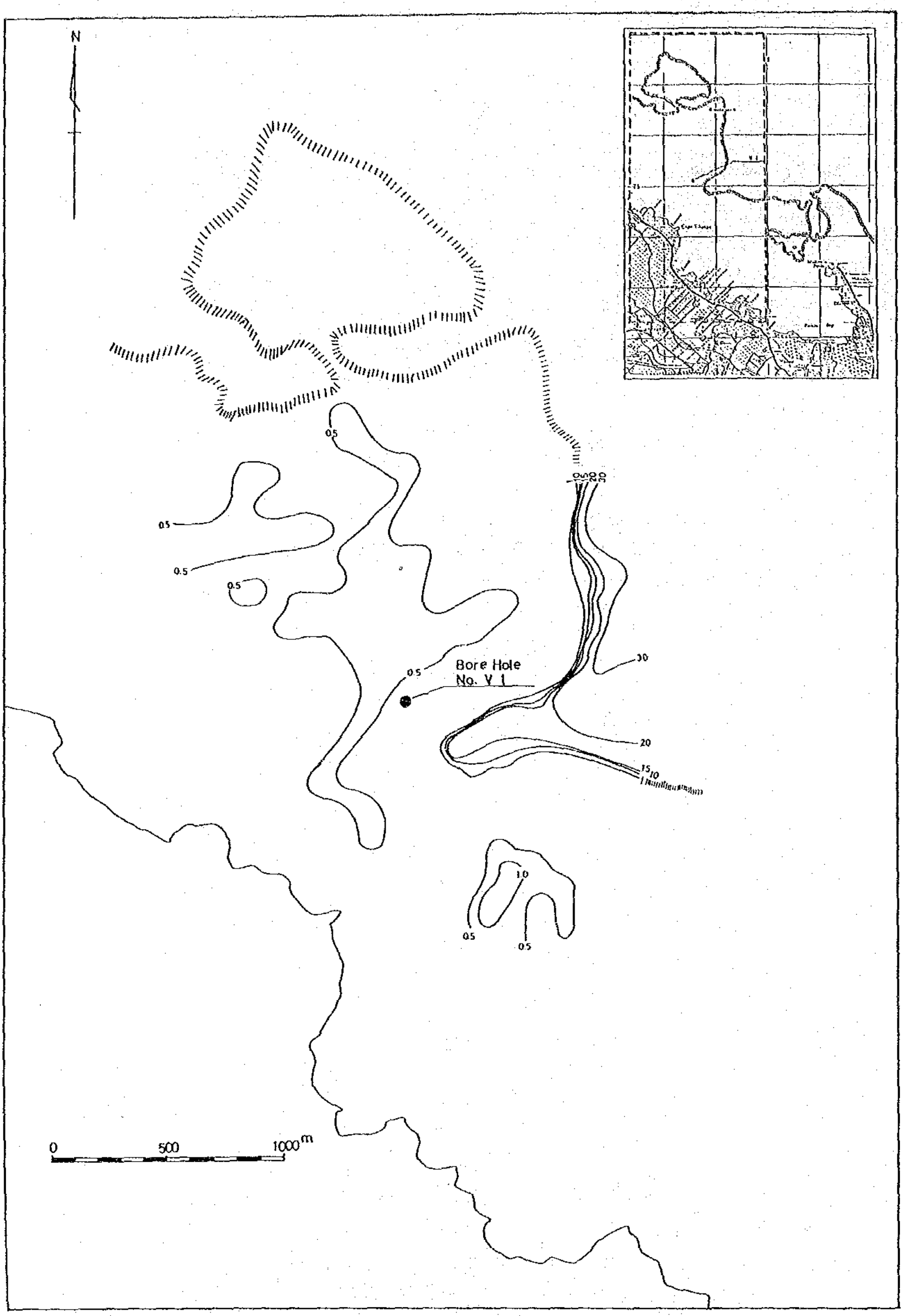


Fig. 3.2.1 Sounding Chart, Vaiusu Bay, 1987

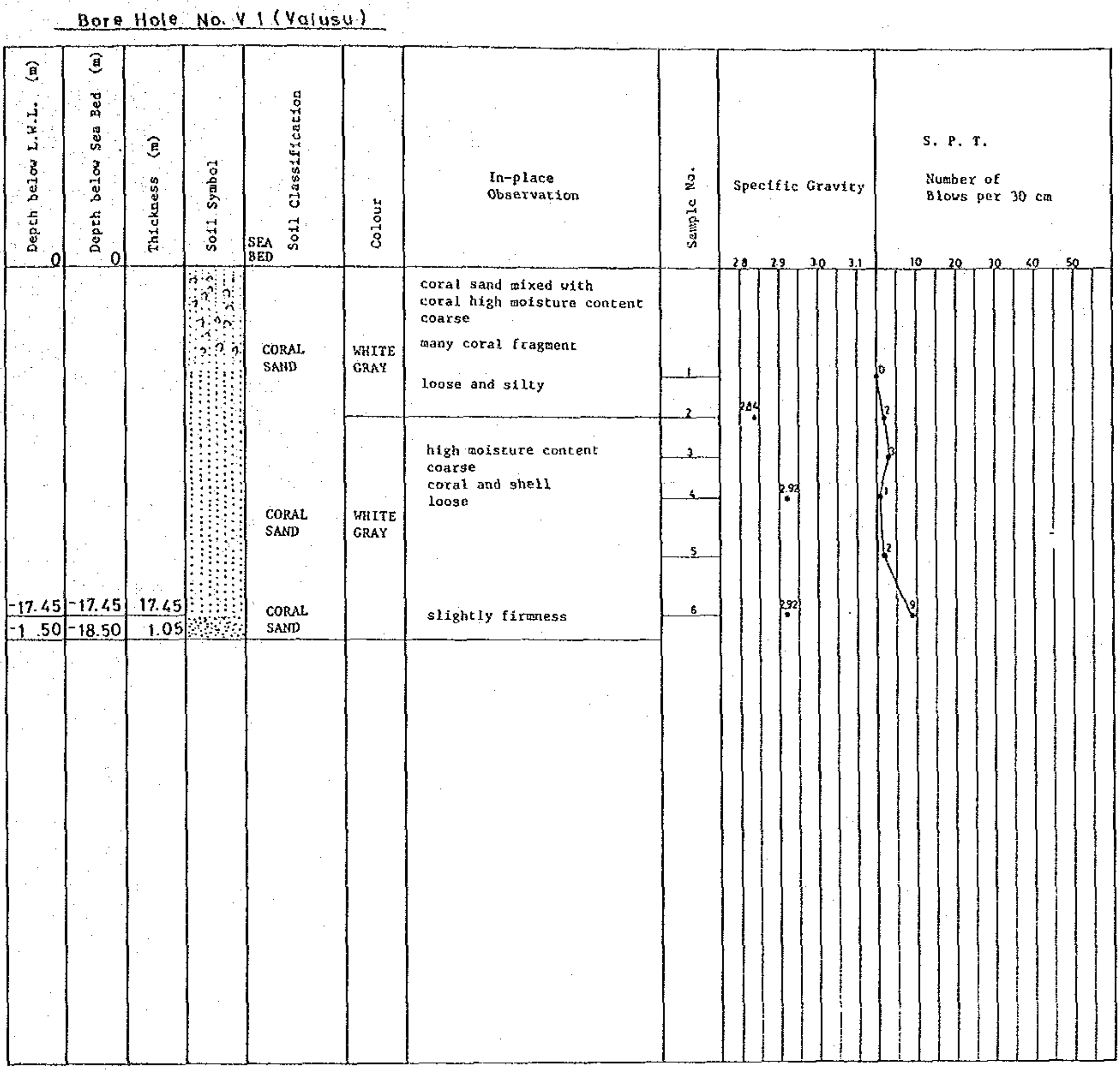


Fig. 3.2.2 Soil Profile (Vaiusu Bay, V1)

CHAPTER 4
INVESTIGATION OF THE EXISTING MAIN WHARF OF APIA PORT

Chapter 4 Investigation of the Existing Main Wharf of Apia Port

1. The existing main wharf shown in Fig. 4.1 - Fig. 4.3 was constructed in 1966. Structually, it is a vertical/batter H-shaped steel pipe pier with concrete beam and slab. The steel piles are encased in precast concrete sleeves from the underside of the deck to sea bed. The concrete sleeves are filled with tremied concrete in order to protect the steel piles from corrosion as well as to increase the rigidity against buckling. The piles are spaced in a grid of 15′x 9′1.5". The wharf is provided with two access bridges. The wharf is deteriorated, especially its supporting steel piles, and the live load on its concrete deck is limited.

2. To assess the serviceability of the wharf, an investigation on its structural strength has been carried out and the results are discussed in this chapter.

4-1 Method of the Investigation

3. The preliminary visual survey and the literature study led to the identification of the following survey items and investigation methods.

① Damage of the deck

Overall visual survey on the superstructure:
concrete slabs, curbing, bollards, fenders

② Strength of the deck concrete

Measurement of compressive strength by Schmidt hammer and laboratory test of concrete core sample:
top surface of the slab

③ Subsidence of the deck

Measurement of elevation of the slab by level:
entire area of the slab

④ Damage of the beams

Visual survey by boat:

all the beams supporting the concrete slab.

⑤ Damage of the piles

Steel thickness measurement by ultrasonic thickness gauge and visual survey by divers:

entire length above sea bed for selected piles

4. Survey item ⑤ above is the most important factor to assess the structural strength of the wharf and was thus carried out with the utmost care to obtain an overall picture of the present structural condition of the main wharf. The results of the surveys carried out this time are compared with the results of previous surveys.

4-2 Results of the Investigation

The results of the surveys are presented below.

1) Damage of the Deck

5. When a wharf experiences nonuniform subsidence, structural cracks are usually visible on the slab surface. A careful visual observation was carried out and only a limited number of hairline cracks were observed. These small cracks are judged to have been caused through shrinkage of the concrete at the time of construction and do not have any effect on the structural strength of the wharf.

6. The curbing of the wharf is damaged at 16 locations and the fender system is also considerably damaged from berthing impact at about 5 places. The results of the investigations are shown in Fig. 4.2.1 and Fig. 4.2.2 respectively. No significant damage to the bollards was observed.

2) Strength of the Deck Concrete

7. The investigation of the strength of the deck concrete was carried out by using a Schmidt hammer and at the same time by taking a core sample for a compressive strength test. Since a core sample was taken at only one location, the overall strength of the wharf is judged based on the data obtained by the Schmidt hammer. The values measured by the Schmidt hammer are calibrated by the value obtained by the laboratory compressive strength test of the core sample. The results of survey are shown in Table 4.2.1. A considerable dispersion of the values measured by the Schmidt hammer is observed ranging from 150 to 240 kg/cm^2 in compressive strength. The compressive strength of the existing main wharf is judged to be on the order of 200 kg/cm^2.

3) Subsidence of the Deck

8. The elevation of the deck was surveyed for the entire area of the wharf using a level. The results are shown in Fig. 4.2.3. Though a subsidence of about 10 cm has taken place at the extreme northern end of the deck, it is judged that no significant nonuniform subsidence is shown since no significant cracks are observed on the slab surface as described above. All the piles supporting the deck are driven to the hard bearing stratum of rock, and this is considered as the main reason for the high stability against sinkage of this wharf.

4) Damage of the Beams

9. The beams were checked by visual survey by boat, and no significant damage was observed.

5) Damage of the Piles

10. Based on the results of the preliminary visual survey on the overall condition of the piles, 23 heavily damaged piles were selected for detailed underwater investigation in such a way as to enable a comparison of the present condition with the condition during the previous surveys carried out by the Royal New Zealand Navy in 1966 only five months after the

the completion of the wharf and by the Australian Development Assistance bureau in 1977. Selected piles are shown in Fig. 4.2.4. Results are tabulated in Table 4.2.2.

11. The results of the investigation on the damage of piles are summarized below.

① Northward part; No.1 piles (row/A - F) are most damaged and concrete sleeves are missing below -5.0m.

② Center part; At No.19D and No.24E/N piles, big holes in the concrete sleeves are observed at -8.0m.

③ Southward part; H-shaped steel of No.37D pile is exposed at -4.5m.

12. As the center part of the wharf is actually used for handling the containers, it can be considered that the piles of the center parts of the wharf are the critical ones for the entire structural strength of the wharf. Therefore, the condition that the concrete sleeves of piles are missing below -8.0m is applied in the case of calculating the pile strength described in Chapter 5 of this report.

13. As for the thickness of the H-shaped steel pile, 16 points were measured using a ultrasonic thickness gauge. The results are tabulated in Table 4.2.3 and presented in Fig. 4.2.5.

The tendency of the results shown in Fig. 4.2.5 is as follows.

① In the case of 12BP53 piles, the measured thickness is smaller than the original dimension of them (+0.1 - -1.6m/m).

② In the case of 14BP73 piles, the measured thickness is bigger than the original dimension of them (+1.8 mm - -0.4 mm).

14. As for the results of 14BP73 piles, it is considered that the reason for the greater thickness is a variation in production or an alternation of

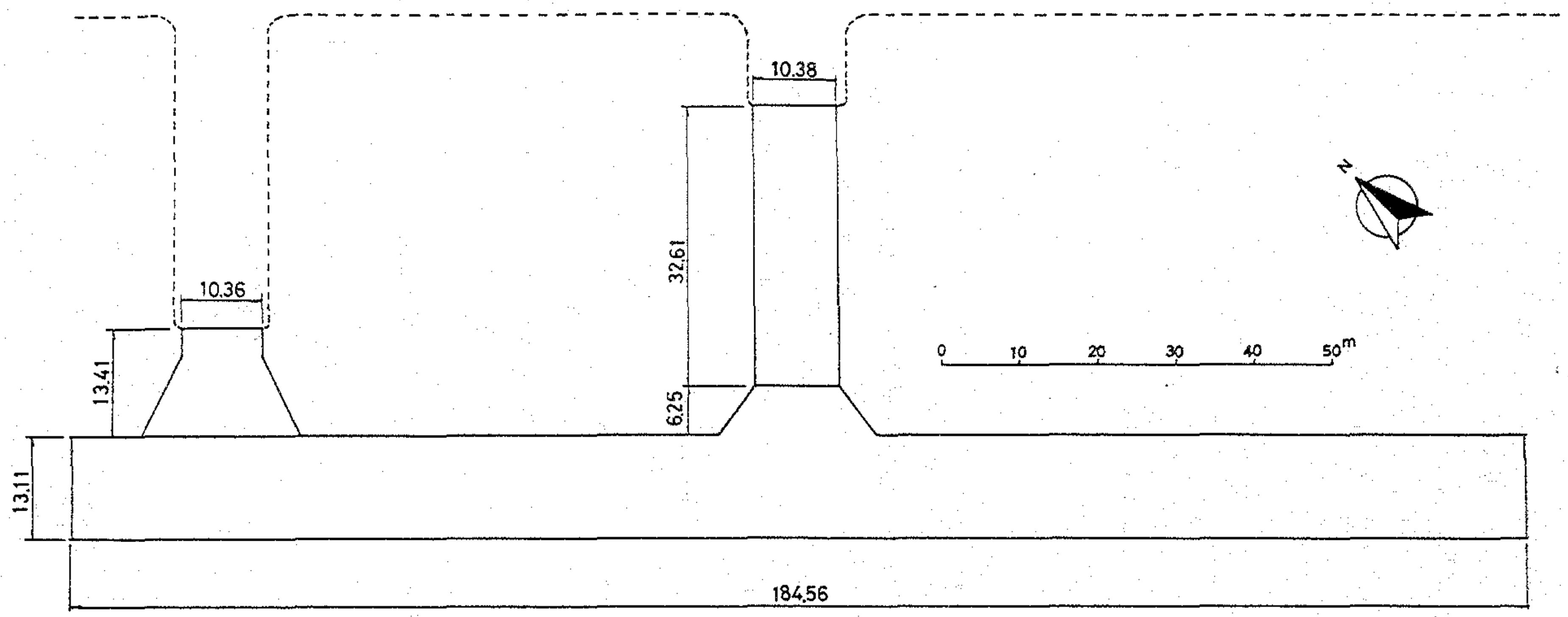
the piles when the pile driving was conducted. Therefore, it is reasonable to discuss the corrosion rate using the results of the thickness measurement on 12BP53 piles. Though the original dimensions with allowance of the H-shaped steel piles can not be determined because of the production variation, it is assumed that the difference between the original dimension and the measured value is the reduced thickness caused by corrosion.

15. Using the thinnest datum (No.1E pile), the corrosion rate would be estimated as 0.08 mm/year, (1E, -1.6mm/20year). As 0.1 mm/year is the standard corrosion rate of steel parts in water, it is assumed that 0.08 mm/year is a reasonable corrosion rate.

--- Comparison with the previous surveys

16. The survey in August 1966 carried out by the Royal New Zealand Navy reports a considerable number of cracks and spalling of concrete sleeves shortly after construction. This could be explained by the difficulty of the tremy work of filling concrete into the concrete sleeves. Another survey in July/August 1977 was carried out by the Australian Development Assistance Bureau. In this report, a detailed description of the condition of the wharf with emphasis on the underwater pile defects is reported. A comparison of the two previous surveys and the present survey is summarized in Table 4.2.4.

17. As shown in the table, the damage of the concrete sleeves concentrates at the lower part of the piles. Two piles, 32D and 33C, were surveyed in all three surveys. The damage of piles 33C observed this time was reported in both of the previous surveys. Therefore it should be noted that the damage which has occurred since the completion of the construction is relatively small.



(Unit:m)

Fig. 4.1 Plan of Apia Wharf

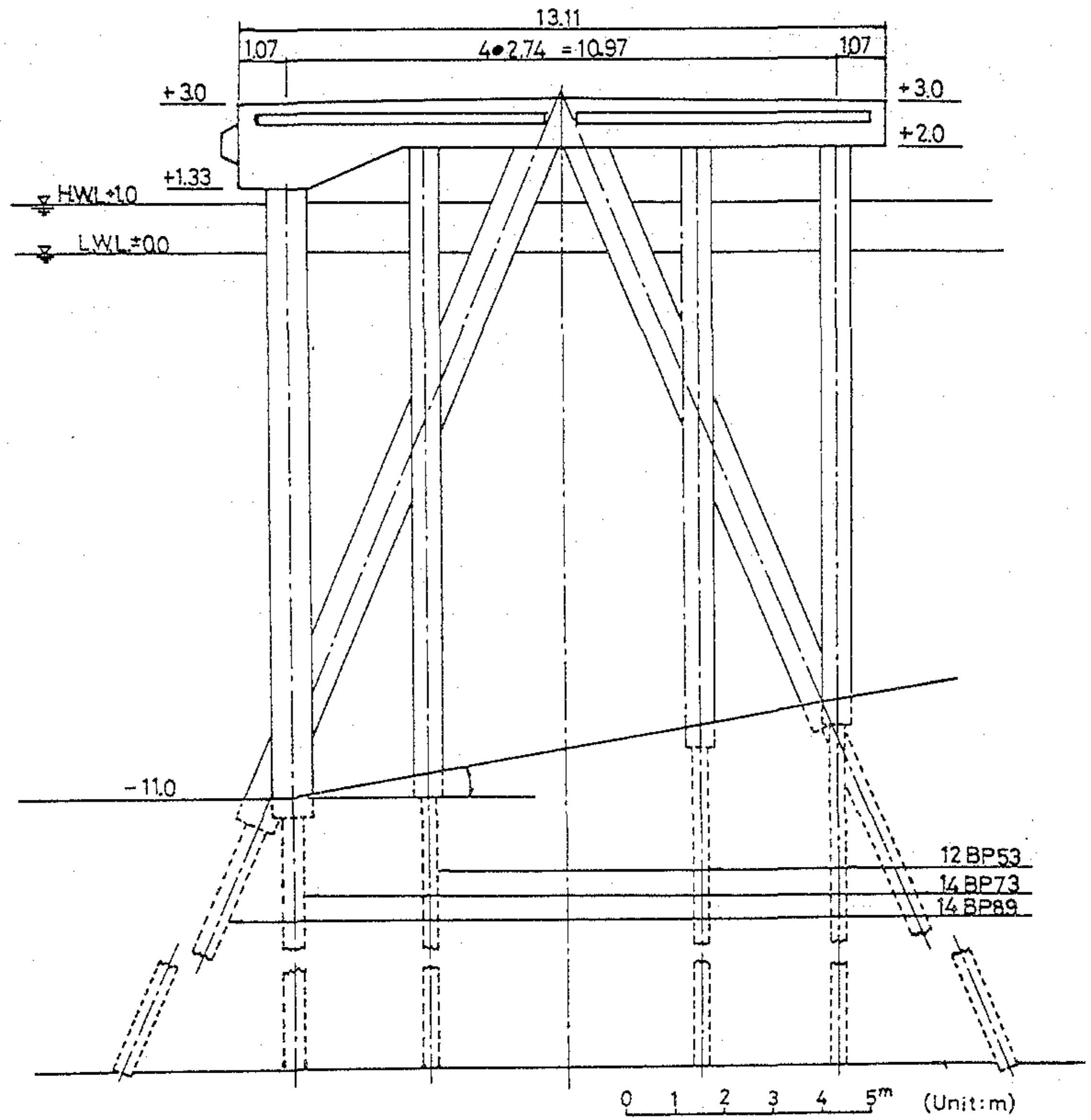


Fig. 4.2 Cross Section of Apia Wharf

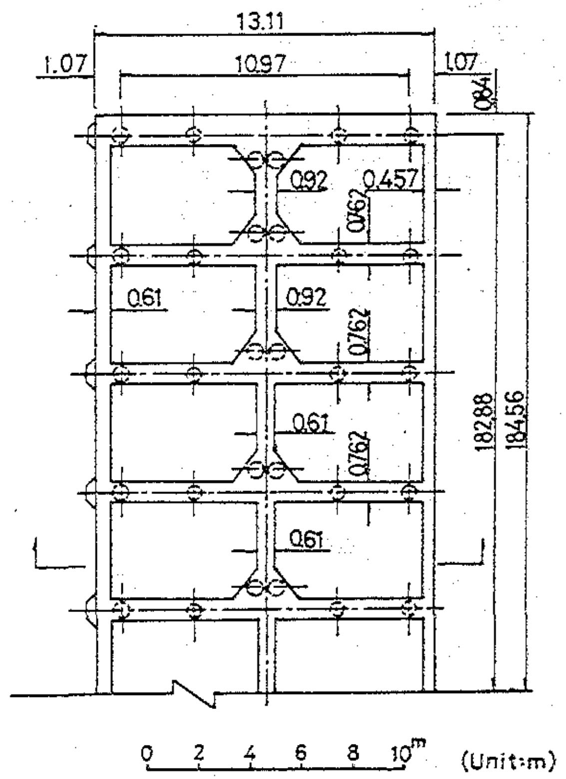


Fig. 4.3 Plan of Beams

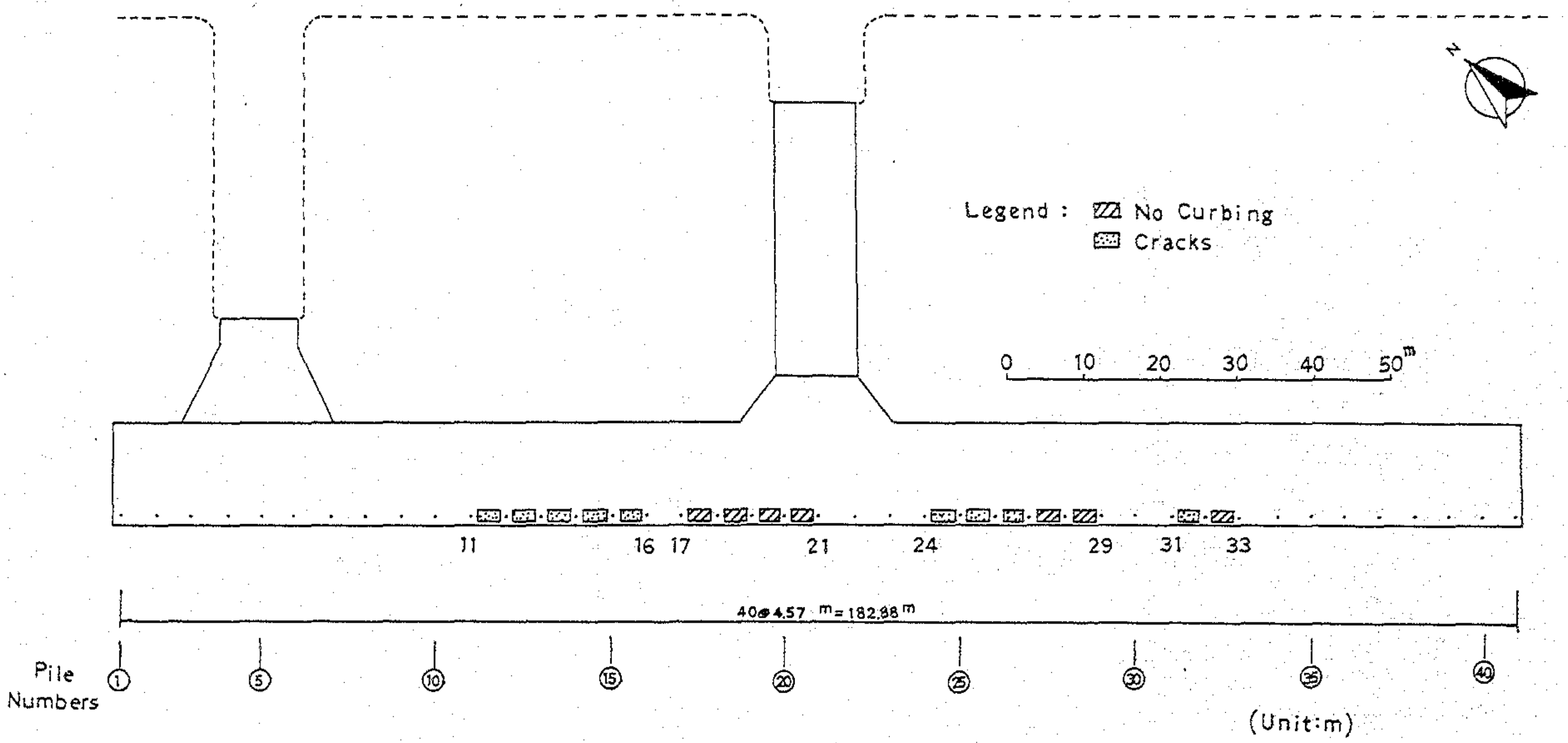


Fig. 4.2.1 Location of Damaged Curbing

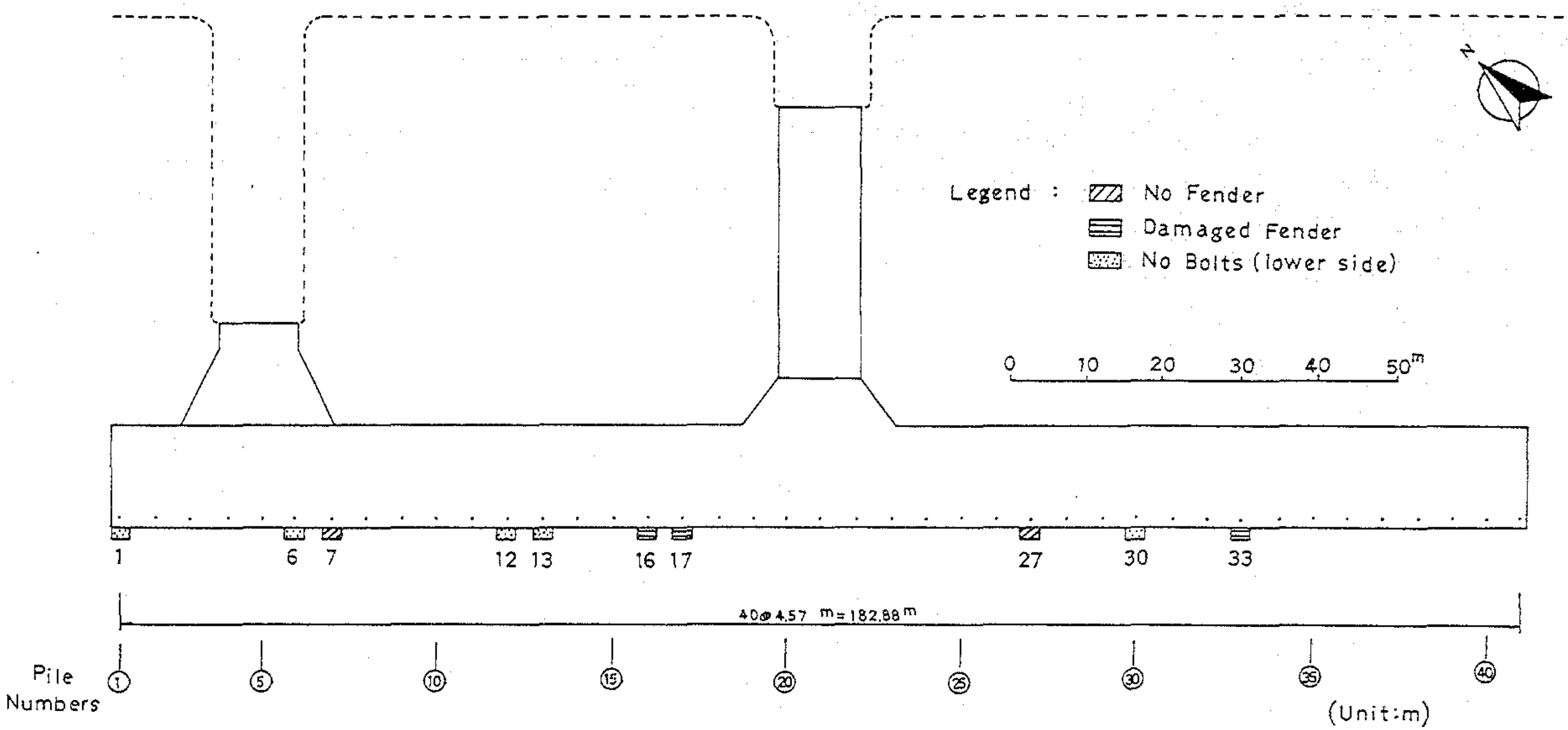
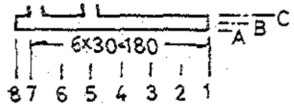


Fig. 4.2.2 Location of Damaged Fenders

Table 4.2.1 Result of Schmidt Hammer Test



	1			2			3			4		
	A	B	C	A	B	C	A	B	C	A	B	C
Compressive Strength of Concrete (kg/cm^2)	193	207	175	208	165	201	191	236	212	186	219	225

	5			6			7			8		
	A	B	C	A	B	C	A	B	C	A	B	C
Compressive Strength of Concrete (kg/cm^2)	212	150	187	168	213	193	175	220	192	209	197	205

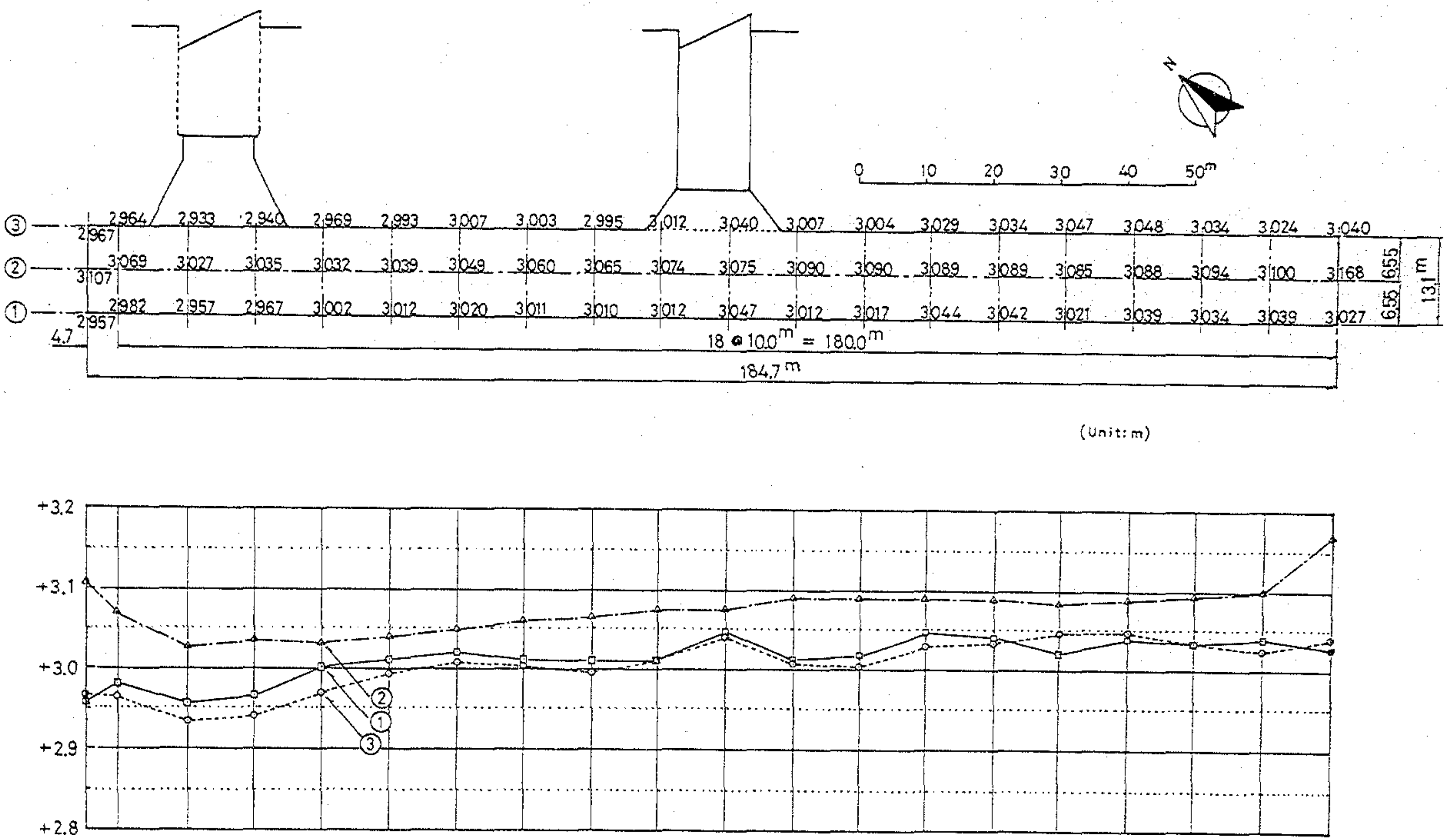


Fig. 4.2.3 Results of Leveling Survey

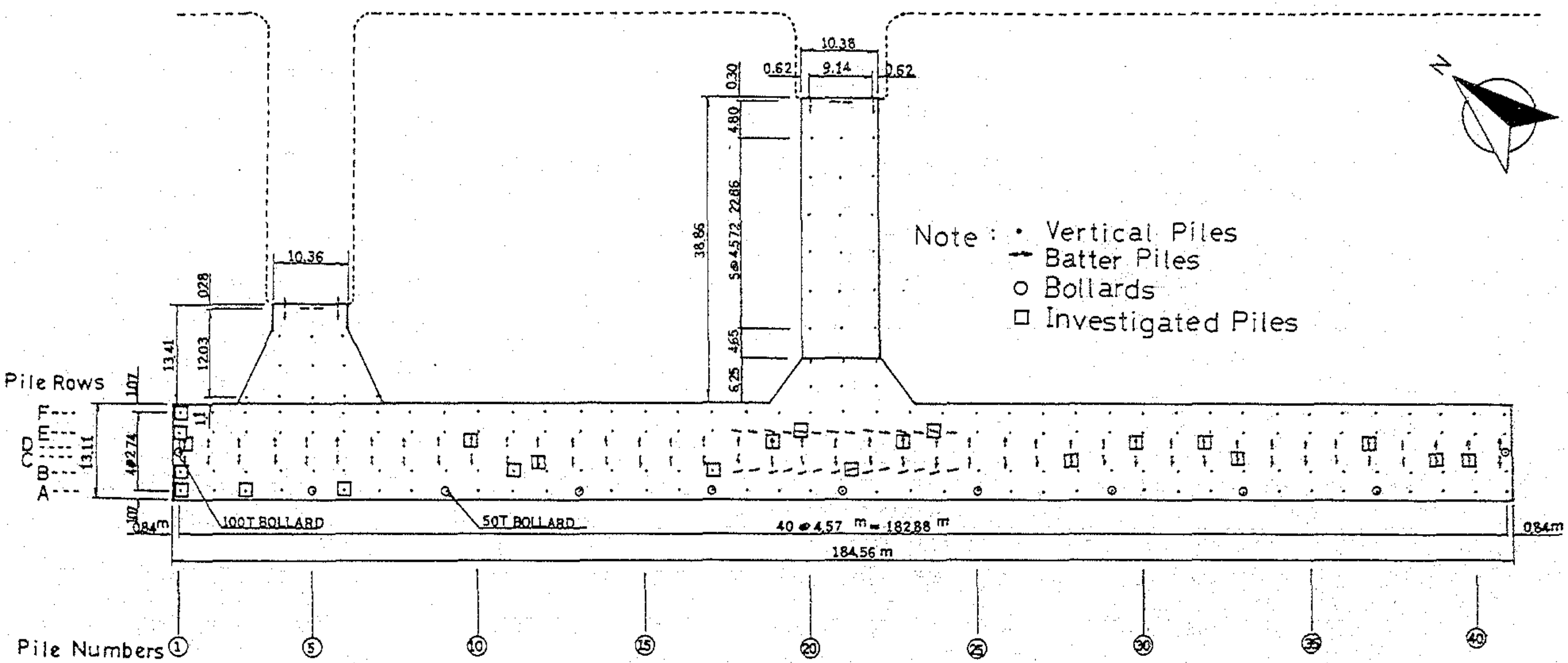


Fig. 4.2.4 Location of Selected Piles

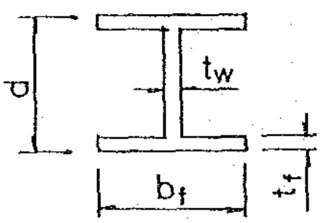
Table 4.2.2 Results of the Investigation of the Piles

Pile No.	Row	Location under L.W.L.	Remarks
1	A	-5.0 m	H.S.S. visible, concrete missing
	B	-4.0 m	Hole, reinforcing exposed
	D	-7.0 m	No tremied concrete is visible
	E	-4.0 m	Concrete missing
	F	-7.5 m	Concrete missing
3	A	-4.0 m	Concrete missing
6	A	-8.0 m	Concrete missing
10	D	-7.5 m	H.S.S. visible
11	B	-8.0 m	Hole, cracked all around
12	C	-7.5 m	Hole, cracked
17	B	-7.0 m	H.S.S. exposed, partly tremied concrete visible
19	D	-8.0 m	Large hole, H.S.S. exposed
20	E/N	-5.0 m	Cracked all around
21	B/S	-7.0 m	Small hole, cracked all around, H.S.S. exposed
23	D	-7.5 m	Large hole, H.S.S. exposed
24	E/N	-8.0 m	Large hole, cracked all around
28	C	-7.5 m	Cracked all around
30	D	-6.5 m	H.S.S. exposed
32	D	-10.0 m	Hole 0.4 m in diameter, H.S.S. visible
33	C	-9.5 m	Horizontal crack, 1cm breadth
37	D	-4.5 m	Hole, H.S.S. exposed
39	C	-8.0 m	Hole, H.S.S. exposed
40	C	-7.5 m	H.S.S. exposed, concrete missing

Note: H.S.S. means H-shaped steel.

Table 4.2.3 Results of the Thickness Measurement

Pile No.	H-steel Type	Original Dimension(mm)	Measured Value(mm)	Measured location(m) below L.W.L.	Balance (mm)
1A	14BP73	12.9	12.5	-8.0	0.4
1B	12BP73	1.1	11.1	-9.3	±0.0
1D	14BP73	12.9	12.8		-0.1
			13.5	-9.0	+0.6
			14.0		+1.1
1E	12BP53	11.1	10.2		-0.9
			9.5	-4.5	-1.6
			10.3		-0.8
			9.6	-7.5	-1.5
1F	12BP53	11.1	11.1		±0.0
			10.4	-9.5	-0.7
			11.2		+0.1
			11.0	-7.5	-0.1
17B	12BP53	11.1	11.1	-8.5	±0.0
19D	14BP73	12.9	14.7	-8.0	+1.8



Original Dimensions

(Unit: mm)

Type of H-shaped Steel	d	b_f	t_f	t_w
12BP53	299.2	306.0	11.1	11.1
14BP73	346.5	370.5	12.9	12.9
14BP89	352.0	373.3	15.6	15.6

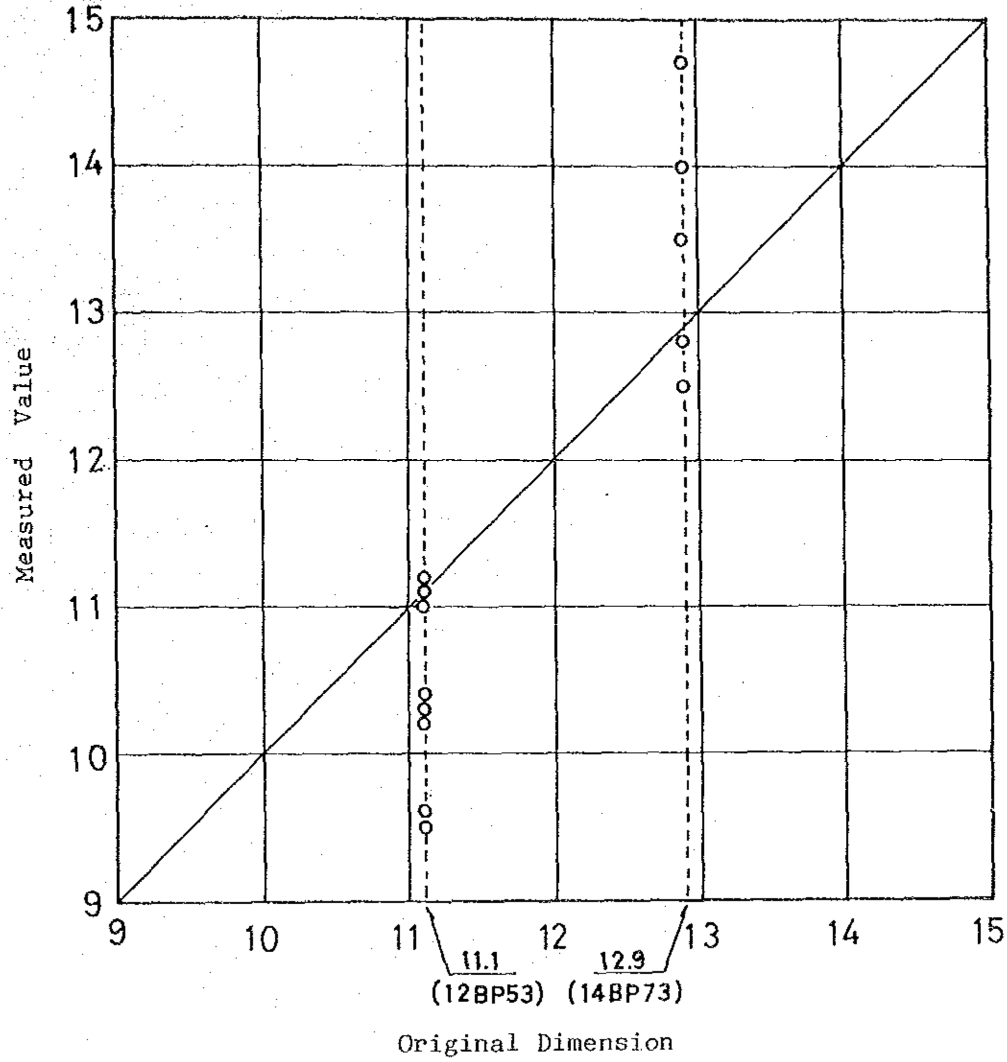


Fig. 4.2.5 Results of Thickness Measurement

Table 4.2.4 Comparison of Pile Condition

Year of Survey	Pile No. 32D	Pile No. 33D
1966	1/4" crack all round 3 feet from seabed. Pile 1 ft square full of concrete fallen out, but tremy still visible.	Cracked all round 3 feet from seabed. Basically 2 cracks but interlaced at 8 feet from present seabed.
1977	above seabed (m) 0.5 Vertical crack extends to 9 meters where the pile is cracked all around. 0.9-1.2 Hole 3 x 2 meters R.S.J. and reinforcing exposed	above seabed (m) .5 Cracked all round .5-1 Vertical crack 1 Cracked all around 1.5 " " " 1.8 " " "
1987	above seabed (m) 0 crack all around 1.0 Hole 0.4m in diameter concrete fallen out H-shaped Pile visible	above seabed (m) 0-1.5 Horizontal 1cm breadth crack

CHAPTER 5

STRUCTURAL ANALYSIS OF THE MAIN WHARF OF APIA PORT

Chapter 5 Structural Analysis of the Main Wharf of Apia Port

5-1 Objectives

1. The objectives of the structural analysis described in this chapter are to evaluate the present condition of the main wharf of Apia Port, to estimate the remaining life and to investigate the extended remaining life with appropriate countermeasure.

5-2 Method of the Structural Analysis and Preliminary Assumptions

2. The schematic flow of the structural analysis of the main wharf is shown in Fig. 5.2.1.

1) Assumptions for the Actual Acting Force Analysis -

(1) Superstructure is a continuous beam.
(2) Vertical force is distributed to the vertical piles and the coupled batter piles.
(3) Horizontal force is distributed only to the coupled batter piles.

2) Assumptions for the Allowable Force Analysis -

(1) Piles are long columns with two sections: an H-shaped steel section and a concrete reinforced section.
(2) Piles are supported under a fixed condition.
(3) Allowable force of the piles is equivalent to the buckling force of the column.
(4) Allowable force of the piles is considered to be decreasing with the lapse of time due to the corrosion of the H-shaped steel section.

3. The present structural strength of the wharf is evaluated by comparing the actual acting force with the allowable force.

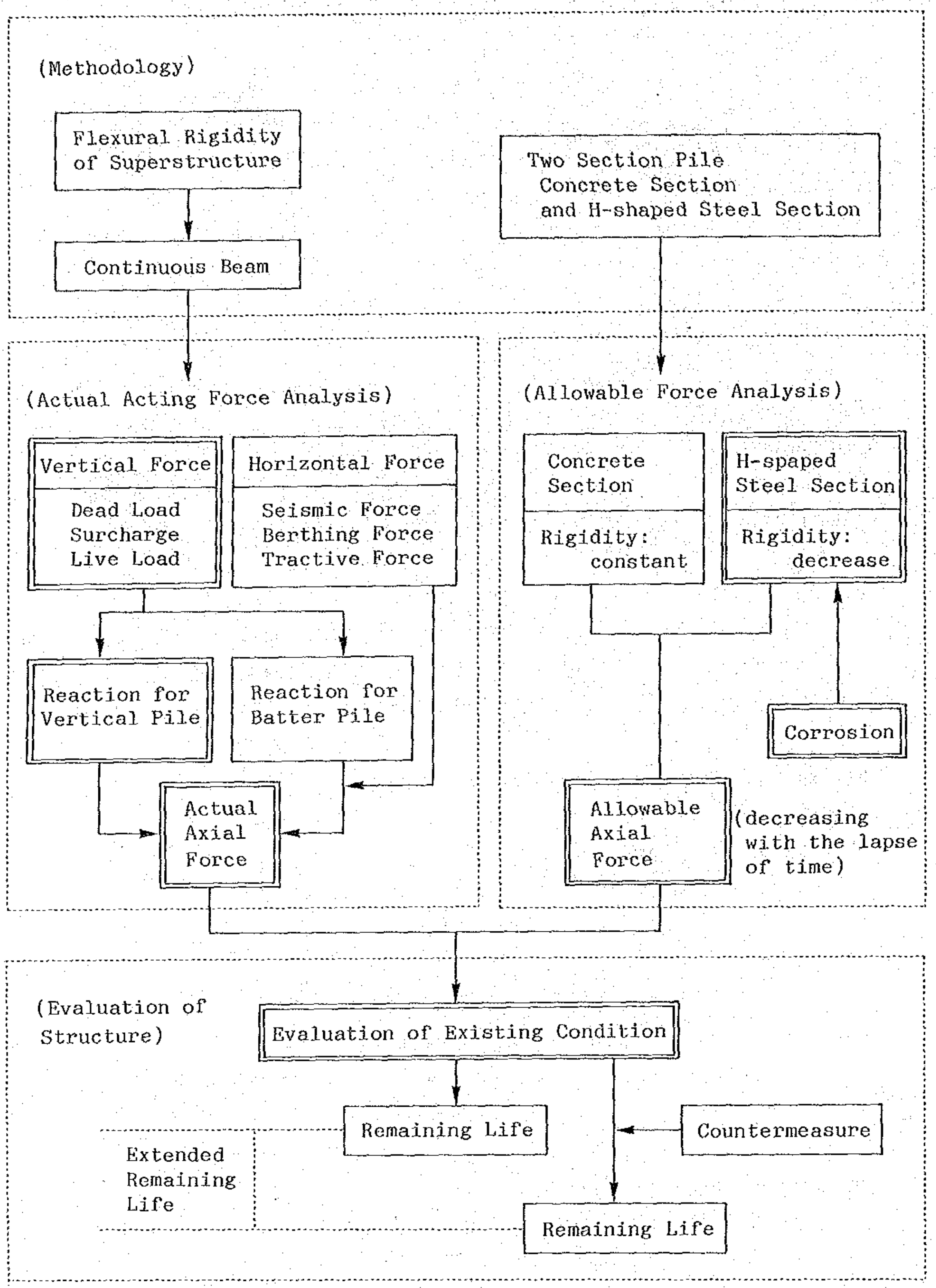


Fig. 5.2.1 Schematic Flow of the Structural Analysis of the Main Wharf

4. The remaining life of the piles is estimated based on the presumption that the future corrosion rate of the H-shaped steel sections is equal to the present value. The possibility of extending the remaining life of the piles is investigated for the case of cathodic protection. Modeled cross section is shown in Fig. 5.2.2. Typical cross sections of the piles are shown in Fig. 5.2.3 - 5.2.4.

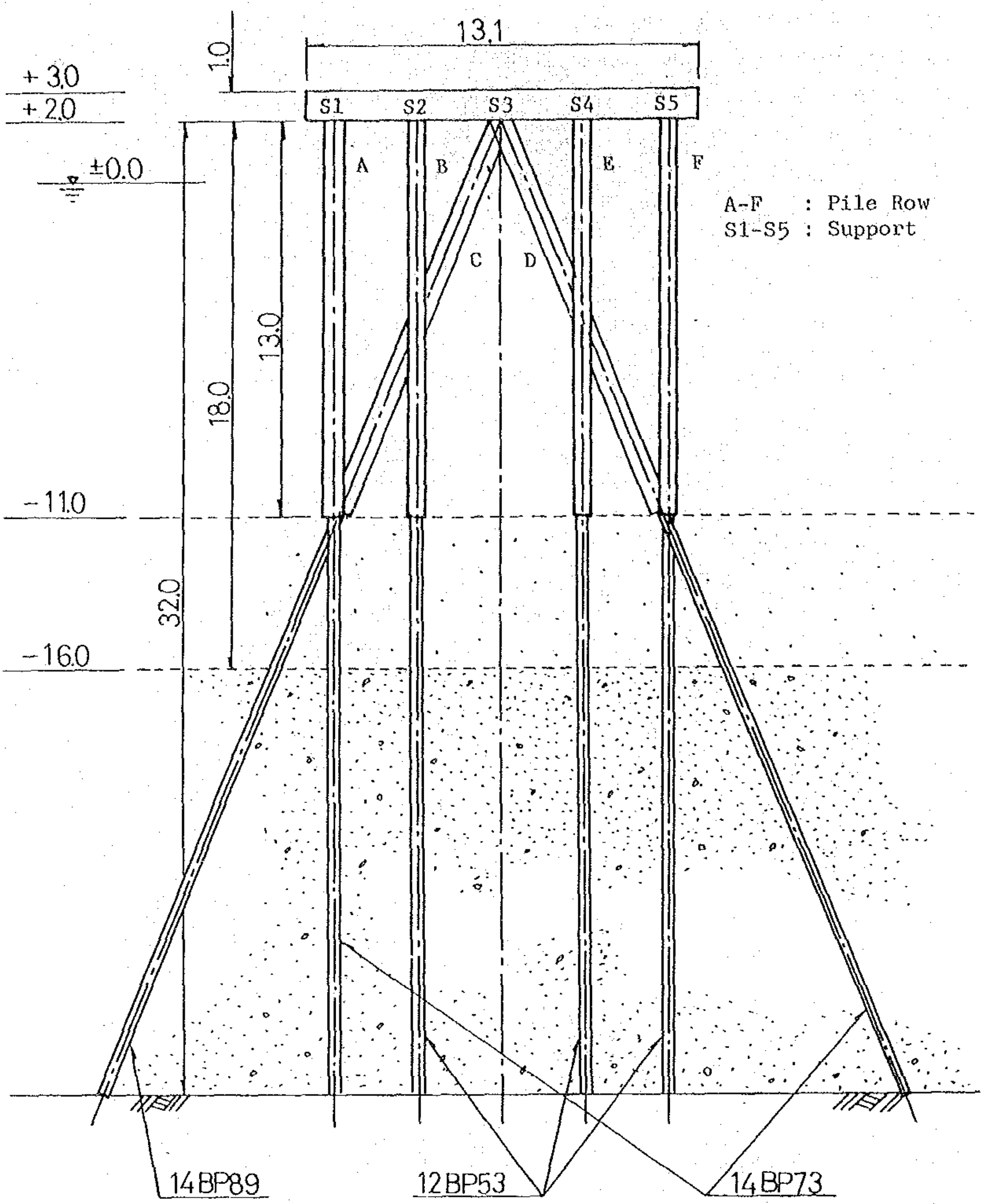


Fig. 5.2.2 Modeled Cross Section of the Wharf

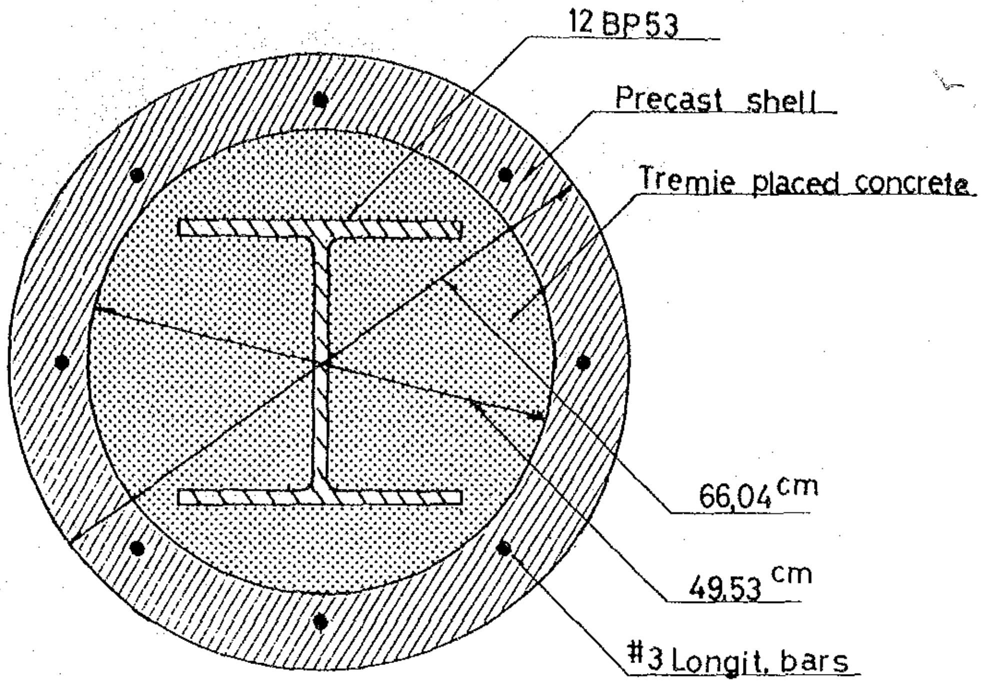


Fig. 5.2.3 Typical Cross Section of the 12BP53 Piles

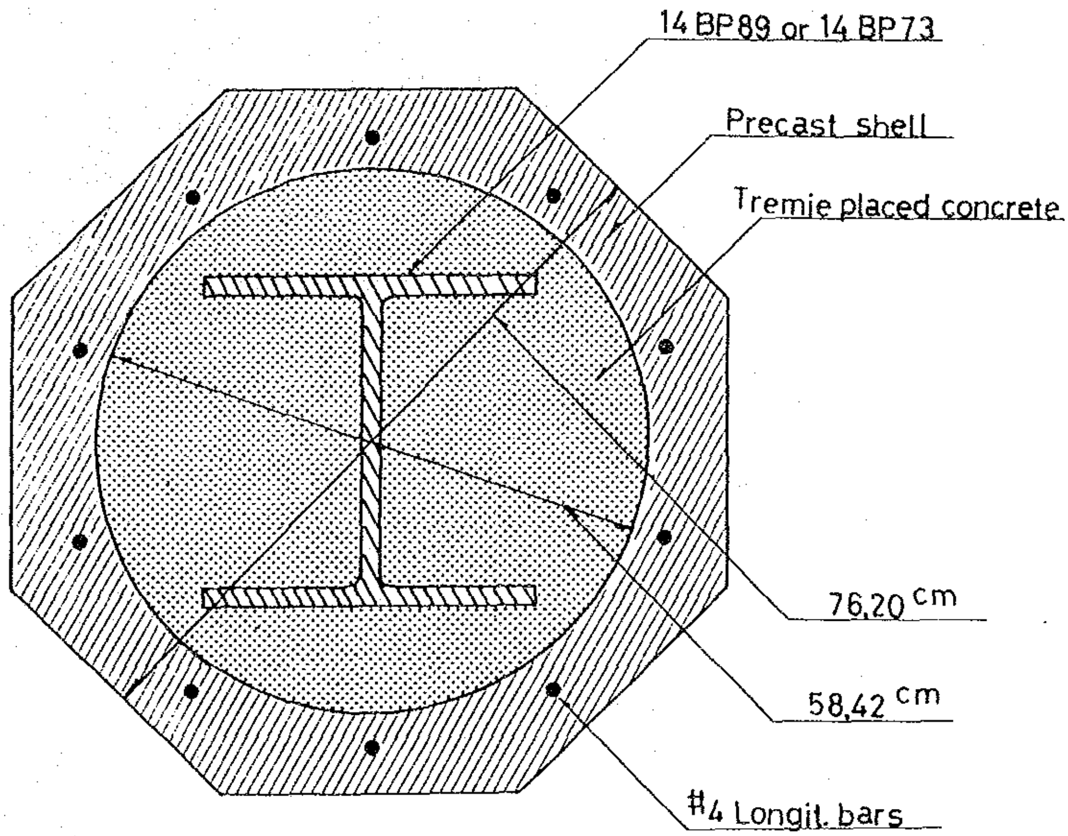


Fig. 5.2.4 Typical Cross Section of the 14BP73 (or 14BP89) Piles

5-3 Calculation of the Actual Acting Force

1) Vertical Force -

5. Vertical force comprises the dead load, the surcharge of the container cargo and the live load of the forklift.

(1) Dead Load: slab and beam

$Wd = 1.59\ t/m^2$

(2) Surcharge: for both ordinary and earthquake conditions; assumed arrangement of containers is shown in Fig. 5.3.1.

$Ws = 0.7\ t/m^2$

$(= \frac{20}{6.5 \times 4.4})$

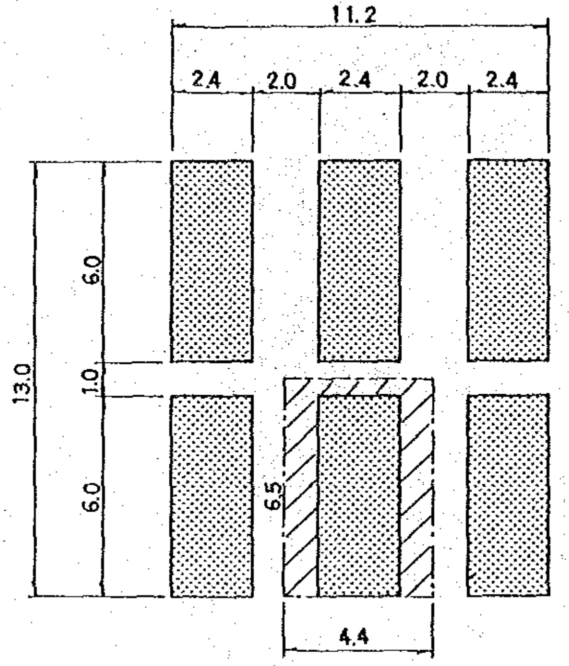


Fig. 5.3.1 Assumed Arrangement of Containers

(3) Live load: forklift weight 28.1 tons
container cargo 20.0 tons

$P_\ell = 48.1$ tons

(4) Converted Uniform Load for Each Continuous Beam

The dead load and surcharge for each beam is converted to the uniformly distributed load by applying the following expression.

$$\left.\begin{aligned} W_1 &= \frac{W\ell_\ell}{2}\left(1-\frac{1}{3}\cdot\frac{\ell_\ell^2}{\ell_t^2}\right) \\ W_2 &= \frac{1}{3}W\ell_\ell \end{aligned}\right\} \qquad (1)$$

where.

W_1 : converted uniformly distributed load action on the transversal beam

W_2 : converted uniformly distributed load acting on the longitudinal beam

W : dead load and surcharge (W_d or W_d+W_s)

ℓ_t : span length of the transversal beam

ℓ_ℓ : span length of the longitudinal beam

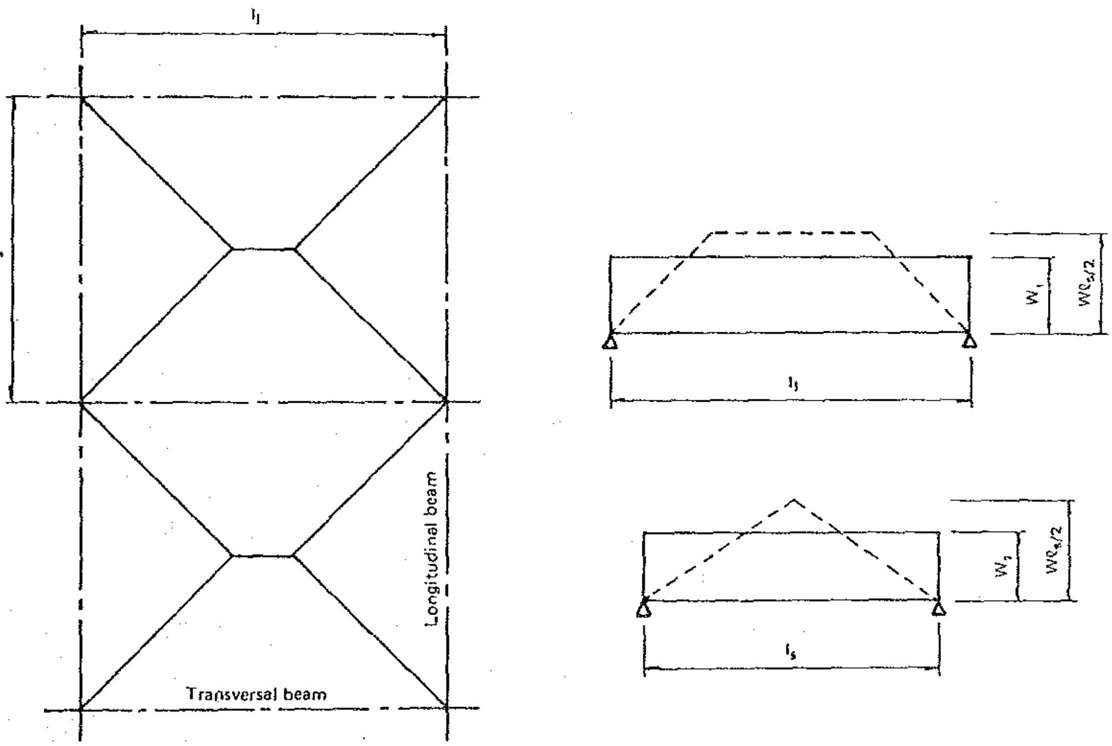


Fig. 5.3.2 Converted Load for Each Beam

The results of the calculation are shown in Table 5.3.1

Table 5.3.1 Converted Load for Each Beam

(tons)

	Longitudinal Beam	Transversal Beam
Dead Load	4.49	5.26
Dead Load and Surcharge	6.61	7.73

2) Calculation Method of the Reaction on the Continuous Beam

6. To calculate the reaction on the continuous beam, the theorem of three moments is applied.

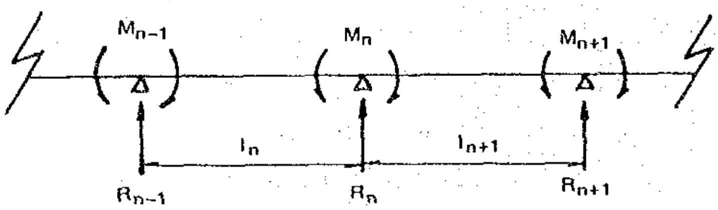


Fig. 5.3.3 Continuous Beam

$$\ell_n M_{n-1} + 2(\ell_n + \ell_{n+1})M_n + \ell_{n+1} M_{n+1} = 6EI(\theta'_{n1} - \theta'_{n2}) + 6EI(\theta_n - \theta_{n+1}) \quad (2)$$

where,

θ'_{n1}, θ'_{n2} : deflection angle by the loading

θ_n, θ_{n+1} : rotation angle

EI : flexural rigidity (assumed constant)

ℓ_{n-1}, ℓ_n, ℓ_{n+1} : span length of the continuous beam

M_{n-1}, M_n, M_{n+1} : moments acting on supporting points

The vertical force acting on the piles is determined by solving the equations for each supporting point simultaneously.

The reaction is calculated by the following expression under the condition of the determination of the unknown moments.

$$
\left.\begin{aligned}
R_{n+1},L &= \{(M_n - M_{n+1}) + W\ell_n^2/2\}/\ell_n \\
R_n,R &= W\ell_n - R_{n+1},L \\
R_n &= R_n,L + R_n,R \\
R_{n+1} &= R_{n+1},L + R_{n+1},R
\end{aligned}\right\} \qquad (3)
$$

where,

R_n , R_{n+1} :	reaction at the support positions
R_n,L , R_{n+1},L :	reaction of the simple beam on the left side of the support position
R_n,R , R_{n+1},R :	reaction of the simple beam on the right side of the support position
M_n , M_{n+1} :	moments acting at the support positions
W :	uniformly distributed load
ℓ_n :	span length

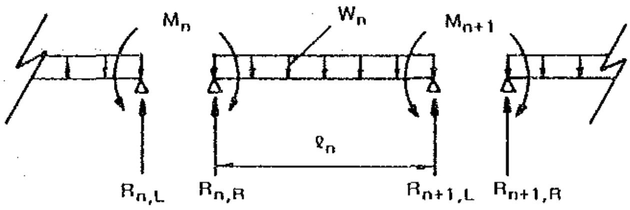


Fig. 5.3.4 Reaction on the Continuous Beam

The results of the calculation are shown in Table 5.3.2

Table 5.3.2 Reaction on the Continuous Beam

(tons)

Case \ Support	S1	S2	S3	S4
Dead load and Surcharge	32.1	24.2	24.2	32.1
Dead Load	23.0	16.5	16.5	23.0

3) Reaction by the Forklift Load

(1) Assumptions

7. It is assumed that the forklift runs along the longitudinal beam and does not run over the outer span of the wharf. The critical position of the forklift is assumed as shown in Fig. 5.3.5.

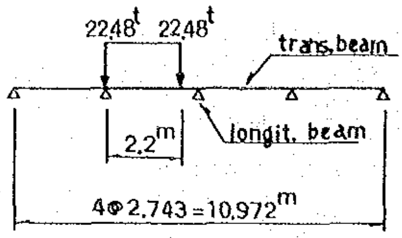


Fig. 5.3.5 Loading Condition of the Forklift

(2) Results of the Reaction Calculation

Table 5.3.3 Reaction by the Forklift

(tons)

S2 (12BP53)	S3 (14BP73, 14BP89)	S4 (12BP53)
27.0	33.7	27.0

(3) Cases and Results of Calculation of Actual Acting Force for the Vertical Piles

Table 5.3.4 Actual Acting Force for the Vertical Piles

(tons)

Support / Case	S1	S2	S4	S5
Dead Load and Surcharge	32.1	24.2	24.2	32.1
Dead Load and Forklift	23.0	43.5	43.5	23.0

5-4 Calculation of the Actual Acting Force on the Coupled Batter Piles

1) Horizontal Force

8. The horizontal force comprises the berthing force, tractive force and the seismic force.

(1) Berthing Force: ship size is 10,000 GRT
berthing speed is 0.15 m/s

Pb = 80 tonf

(2) Tractive Force: ship size is 10,000 GRT

Pt = 35 tonf

(3) Seismic Force

Ps = k W′

where,

k : seismic coefficient (assumed 0.15)
w' : dead weight acting on one coupled batter piles
Ps = 20.6 tons/coupled batter piles

(4) Horizontal Force on One Coupled Batter Piles

The horizontal force on one coupled batter piles is determined by the following expression taking into consideration the wharf rotation.

$$H_i = \frac{1}{n} H + \frac{\chi_i}{\sum_i \chi_i^2} eH \qquad (4)$$

where,

Hi : horizontal force acting on each pile

H : entire horizontal force

n : number of the coupled batter pile

x : distance between the center of the coupled batter piles and each pile

e : distance between the center of the coupled batter piles and the entire horizontal force

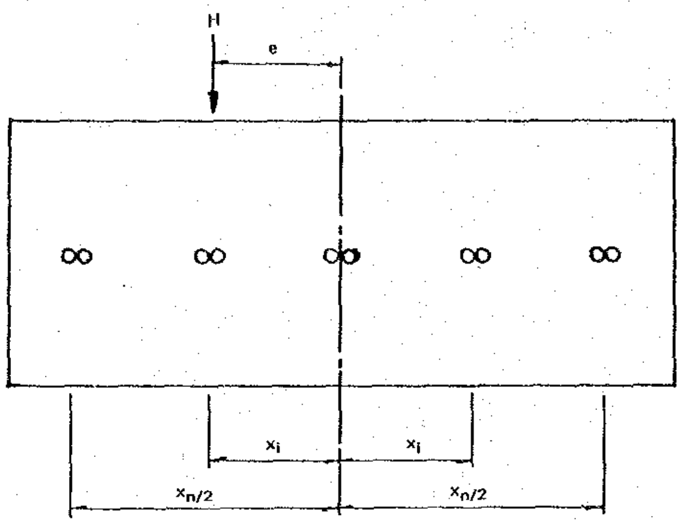


Fig. 5.4.1 Center of the Coupled Batter Piles and Individual Distance from it

The calculation results are shown in Table 5.4.1.

Table 5.4.1 Horizontal Force on Coupled Batter Piles

(tons)

Berthing Force	Tractive Force	Seismic Force
7.53	3.29	20.6

2) Calculation Method of Axial Force

9. The axial compressive force acting on the coupled batter piles is calculated by the following expression.

$$\left.\begin{aligned} P_1 &= \frac{V_i \sin\theta_1 + H_i \cos\theta_2}{\sin(\theta_1 + \theta_2)} \\ P_2 &= \frac{V_i \sin\theta_1 - H_i \cos\theta_2}{\sin(\theta_1 + \theta_2)} \end{aligned}\right\} \qquad (5)$$

where,

P_1, P_2: axial compressive force acting on the coupled batter piles

V_i: vertical force

H_i: horizontal force

θ_1, θ_2: angle of the coupled batter piles

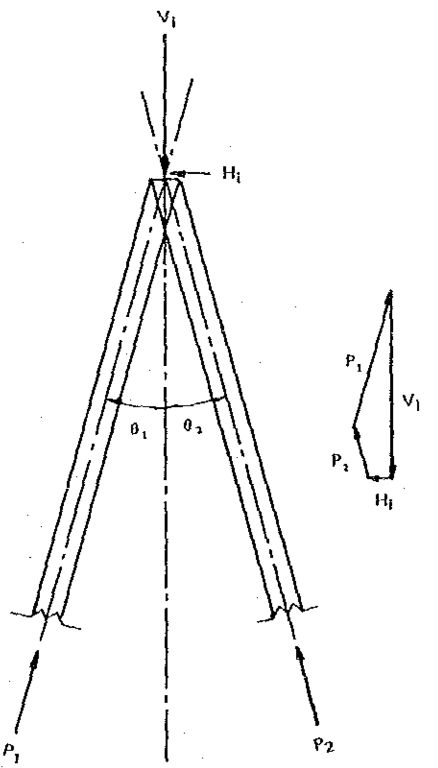


Fig. 5.4.2 Axial Force Acting on the Coupled Batter Piles

3) Case and Results of Calculation

10. Case and Results of Calculation are shown in Table 5.4.2.

Table 5.4.2 Actual Acting Force on the Coupled Batter Piles

(tons)

Case	Vi	Hi	Pi
Dead Load and Berthing Force	32.46	7.53	27.4
Dead Load, Surcharge and Seismic Force	47.8	20.6	52.8
Dead Load Surcharge and Tractive Force	47.8	3.29	35.7

5-5 Maximum Actual Axial Force on Each Pile

11. Maximum Actual Axial Force on Each Pile are shown in Table 5.5.1.

Table 5.5.1 Actual Axial Force on Each Pile

	Vertical Pile		Coupled Batter Piles	Vertical Pile	
Support	S1	S2	S3	S4	S5
Type of H-shaped Steel	14BP73	12BP53	14BP73 14BP89	12BP53	12BP53
Actual Axial Force (tons)	32.1	43.5	52.8	4.35	32.1

5-6 Calculation of the Allowable Force

1) Assumptions for the Calculation

(1) Vertical Surface

12. Based on the results of the boring investigation, it is assumed that the Vertual surface of the supporting layer is -16m.

(2) Pile length

In the case of calculation the buckling load, 18 meters is used for the pile length. That is the length above the virtual surface (-16m) to the under edge of the superstructure (+2.0).

Based on the results of the investigation of the piles, it is assumed that about 3 meters from the lower end of the concrete reinforced section is missing. Therefore, the length of the H-shaped steel section is equal to 10 meters and the length of the concrete reinforced section is equal to 8 meters.

(3) Flexural Rigidity of the Concrete Reinforced Section

The concrete reinforced section is regarded as the composite section of concrete and steel. Therefore, its flexural rigidity is determined by applying the following expression.

$$E'I' = E_s(I_s + I_c/15) \quad (6)$$

where,

E': modulous of elasticity of composite section

E_s: modulous of elasticity of steel

I': geometrical moment of inertia of composite section

I_s: " of H-shaped steel section

I_c: " of concrete section

(4) Supporting Condition

fixed ends

(5) Corroding Rate

$V_c = 0.08$ mm/y (based on the results of investigation described in Chapter 4)

(6) Allowable Stress of Steel

H-shaped steel is assumed to have a strength equivalent to the grade of SS41.
The followable stress is defined by the following expression.

$$
\left.\begin{array}{ll}
\sigma_{sa} = 1{,}400 & (\ell/r \leqq 20) \\
\sigma_{sa} = 1{,}400 - 8.4\,(\ell/r - 20) & (20 < \ell/r < 93) \\
\sigma_{sa} = \dfrac{12{,}000{,}000}{6{,}700 + (\ell/r)^2} & (\ell/r \geqq 93)
\end{array}\right\} \qquad (7)
$$

where,

ℓ/r : slenderness ratio

ℓ : effective buckling length

r : radius of gyration of area

2) Method of Calculation

13. The basic expression on the buckling load is as follows.

$$(P_2)cr = m^2 \frac{EI_2}{\ell_2^{\,2}} \tag{7}$$

where,

- E : modulous of elasticity
- I_2 : geometrical moment of inertia of the H-shaped steel section
- m : coefficient of buckling
- ℓ_2 : length of the H-shaped steel section

As the edge condition is assumed as fixed end, the coefficient of buckling is given by the minimum solution of the following equation.

$$\left.\begin{aligned} & m\left(\frac{P_0}{P} - \frac{\ell}{\ell_2}\right)\left(\tan m + \frac{\ell_1}{\ell_2} - \frac{I_2}{I_1}\tan \gamma m\right) + \left(\frac{\ell_2}{\ell_1}\gamma + \frac{1}{\gamma}\,\frac{\ell_1}{\ell_2}\right)\tan \gamma m \tan m \\ & = \frac{P_0^{\,2} + P_1^{\,2} + P_2^{\,2}}{2P_1P_2} - \frac{2}{\cos \gamma m \cdot \cos m} - 2\frac{I_1}{I_2}\cdot\frac{1}{\cos \gamma m} + 2\frac{P_1}{P_0}\,\frac{1}{\cos m} \end{aligned}\right\} \tag{8}$$

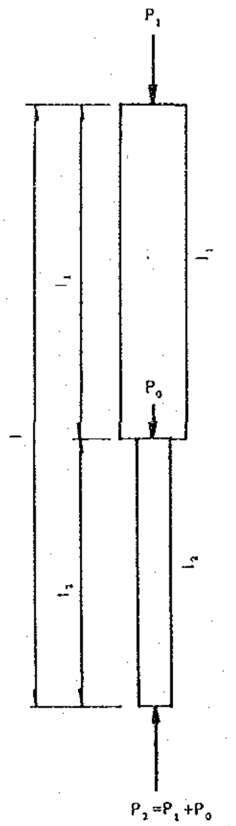


Fig. 5.6.1 Modeled Cross Section of the Piles

where, $\frac{\ell_1}{\ell_2}\sqrt{\frac{P_1}{P_0}\cdot\frac{I_2}{I_1}}$

$\ell = \ell_1 + \ell_2$

$I_1 = I_2 + I_c/15$

$I_c =$ geometrical moment of inertia on concrete reinforced section

3) Results

14. The results of the calculation are shown in Table 5.6.1.

Table 5.6.1 Allwable Force for Each Pile

(tons)

	Vertical Pile		Coupled Batter Piles	Vertical Pile	
Support	S1	S2	S3	S4	S5
H-shaped Steel Type	14BP73	12BP53	14BP73	12BP53	
Allowable Force (ton)	144.8	90.4	217.4	90.4	91.1

The relation between the actual acting force and the critical buckling load in 1966 is shown in Fig. 5.6.2.

As the corrosion rate of the H-shaped steel section is equal for all the piles, it is concluded that the piles of row B and E are exposed to the severest conditions.

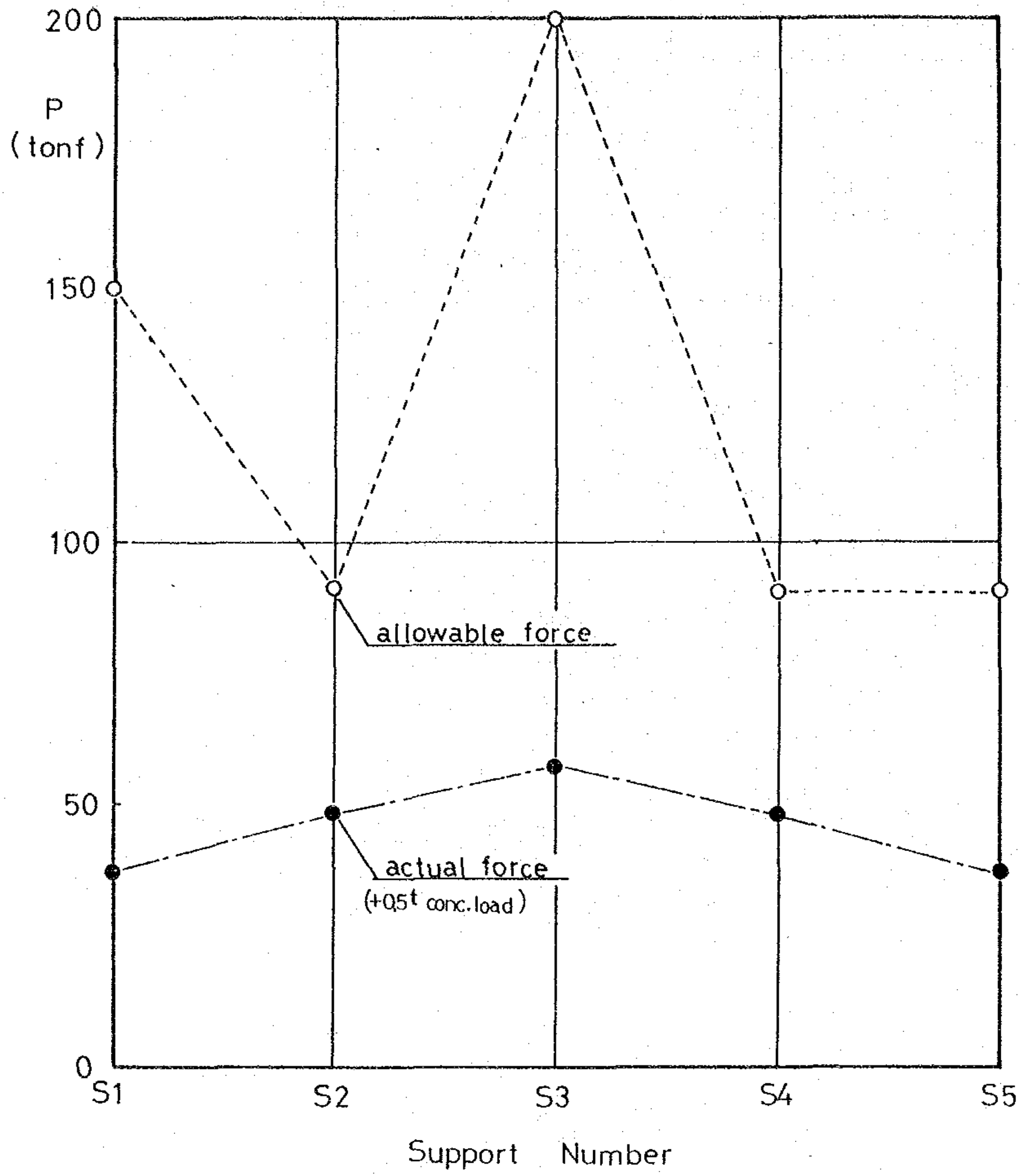


Fig. 5.6.2 Relation between the Actural Acting Force and the Allowable Force

5-7 Evaluation of the Structural Strength

1) Evaluation of the Existing Condition

15. As mentioned in the previous sub-section, the piles of row B and E are the critical ones for evaluating the strength of the entire structure.

The chronological allowable forces on the piles of row B and E are shown in Fig. 5.7.1.

The ratio of the critical allowable force to the actual acting force on the piles of row B at present is estimated at abuot 1.2.

2) Remaining Life

16. The remaining life of the piles of row B and E with a corrosion rate of 0.08mm/y is estimated at about 8 years from 1987 up to about 1995.

3) Extended Remaining Life

17. If the cathodic protection is carried out, the corrosion of the H-shaped steel would be prevented. It is assumed that the cathodic protection would reduce the corrosion rate to 25% of the present value of 0.08mm/year. Therefore, a corrosion rate of 0.02mm/year is considered reasonable. The extended remaining life is thus estimated at about 15 years from 1991 up to 2006 as shown in Fig. 5.7.1.

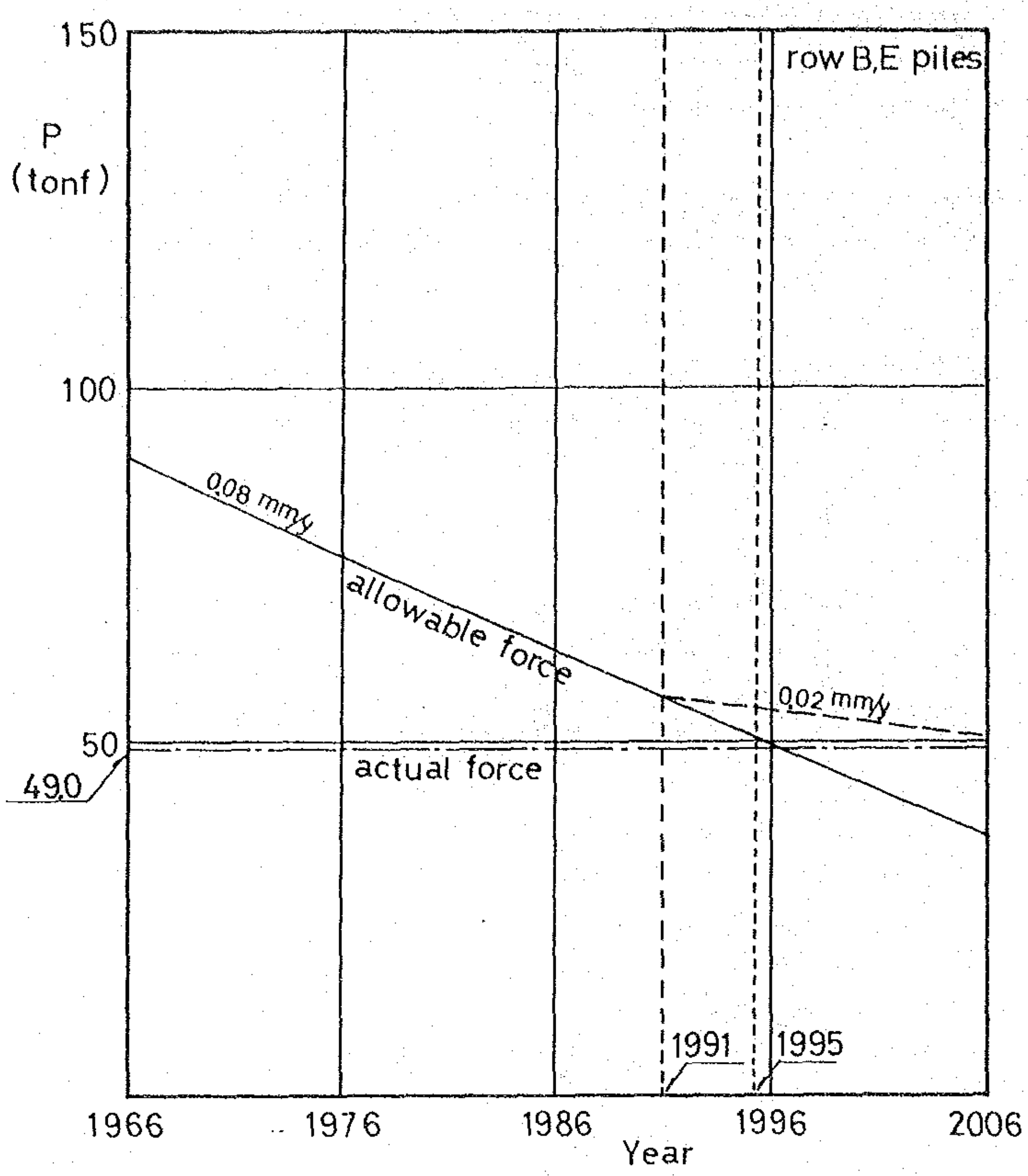


Fig. 5.7.1 Reduction of the Allowable Force

CHAPTER 6
PRELIMINARY DESIGN OF THE MAIN FACILITIES FOR THE FIRST STAGE PLAN

Chapter 6 Preliminary Design of the Main Facilities for the First Stage Plan

1. In this chapter, the outline of the preliminary design of the main facilities under the first stage plan is described. The main facilities to be discussed are the breakwater, the retaining wall of the expansion of the container yard, the quaywall and the dolphin for the ferry terminal. All the facilities are designed based on Japanese Standards for the design of port facilities.

6-1 Breakwater

1) Design Condition

2. ① Off-shore design wave : (Ho)1/3 = 7.0m, (To)1/3 = 10.0sec

② Design wave at breakwater : H1/3 = 4.2m, T1/3 = 10.0sec

③ Design depth : D.L. -13.5m

④ Crest height : D.L. + 2.8m
(= H.W.L. 1.0m + 0.6 x 3m)[1]

⑤ Tidal level : H.W.L. + 1.0m
: L.W.L. ± 0.0m

⑥ Unit weight

- sea water : $1.03 t/m^3$
- plain concrete : $2.3\ t/m^3$ (above water level)
- quarry stone : $2.6\ t/m^3$ (above water level)
- rubble : $1.0\ t/m^3$ (below water level)
- foundation soil : $1.0\ t/m^3$ (below water level)

⑦ Friction coefficient

- concrete against concrete: 0.5
- concrete against rubble : 0.6

---

*1 The crest height of a breakwater is generally designed to be higher than 0.6 times a particular wave height above H.W.L. In this case, the wave height of 3m is selected, which is the wave height at the breakwater equivalent to the critical wave height of 70cm at the wharf area based on the wave analysis described in Chapter 2 of this technical report.

⑧ Angle of internal friction

rubble mound : $\phi = 40^\circ$

foundation soil : $\phi = 30^\circ$

⑨ Allowable safety factor

against sliding : 1.2

against overturning : 1.2

2) Alternative Breakwater Types

3. (1) Rubble Mound Type Armoured with Wave Dissipating Concrete Blocks (Alternative A: Fig. 6.1.1)

i) Wave Dissipating Concrete Blocks

4. The stable weight of individual concrete blocks can be determined based on Hudson's formula below.

$$W=\frac{\gamma_r H^3}{K_D(\gamma_r/\gamma_w-1)^3\cot\alpha}$$

where

W : weight of individual armour unit (t)

γ_r : unit weight of armour unit (= 2.3t/m^3)

γ_w : unit weight of sea water (= 1.03t/m^3)

α : angle of slope measured from horizontal plane ($\cot\alpha$ = 1.5)

H : design wave height (= 4.2m)

K_0 : stability coefficient of armour unit (= 20)

Therefore,

$$W=\frac{2.3\times4.2^3}{3\times(2.3/1.03-1)^3\times1.5}=3.03(t)$$

5. Since armour units at the head of the structural axis line are exposed to the attack of waves from various directions, special attention should be paid to the decision of the armour unit's weight there. In general, it is recommended to use armour units with 1.5 times the weight calculated by the formula above. In this case, the recommended weight is;

Wp = 1.5 x 3.03 = 4.55 (t)

Therefore, 6 ton DOLOS is recommended as the primary cover layer.

6. In general, armour units of less weight than that calculated above can be used for sections deeper than 1.5H below L.W.L. for the seaward side and 1.0H below L.W.L. for the landward side.

ii) Secondary and Tertiary Layers

7. The weights of the secondary and tertiary layers are calculated as follows.

- Secondary layer

  Ws - Wp/10 - Wp/15 = 0.5 - 0.3 ton

- Tertiary layer

  Wt = Wp/200 - Wp/6000 = 2 - 1 kg

Accordingly, 2 ton stone and 50 to 100kg rubble are recommended for the secondary and tertiary layers, respectively.

(2) Concrete Block Composite Type Armoured with Wave Dissipating
8. Concrete Block (Alternative B: Fig. 6.1.2)

i) Concrete Blocks

The width of the concrete blocks is determined by various factors such as the safety against sliding and overturning, and the toe pressure of the rubble mound, etc. under the attack of particular design wave.

The various factors calculated for the cross section shown in Fig. 6.1.2 for the design wave (Hmax = 7.56m) are summarized in the table below.

Table 6.1.1 Results of the Calculation

	Item	Evaluation
Examination at + 0.3m	Safety against Sliding	1.24 > 1.20 GOOD
	Safety against Overturning	7.85 > 1.20 GOOD
Examination at - 1.2m	Safety against Sliding	1.57 > 1.20 GOOD
	Safety against Overturning	6.08 > 1.20 GOOD
	Toe Pressure (t/m^2)	8.77 > qta =50 GOOD

ii) Wave Dissipating Concrete Block

9. The same as Alternative A.

(3) Rubble Mound Type Armoured with Quarry Stone
(Alternative C: Fig. 6.1.3)

i) Quarry Stone

10. The stable weight of individual quarry stones can be determined based on the same formula mentioned in the previous sub-section. In this alternative, the parameters for the formula and the stable weight are as follows.

$r = 2.6\ t/m^3,\ w = 1.03\ r/m^3,\ \cot = 1.5\ ,\ H = 4.2\ m\ ,\ K_D = 3$

$$W = \frac{2.6 \times 4.2^3}{3 \times (2.6/1.03 - 1) \times 1.5} = 12.1(t)$$

11. Based on the same reasoning mentioned in the previous sub-section, concrete blocks of more than 1.5 times the weight above (i.e. 1.5 x 12.1 = 18.1 tons) are recommended. Therefore, 20 ton quarry stone is used in this alternative.

ii) Secondary and Tertiary Layers

Same as in Alernative A.

(4) Comparison of Alternatives

12. The 3 alternatives shown in Fig. 6.1.1 - Fig. 6.1.3 are examined from the viewpoints of the construction cost, supplying capacity of the materials, difficulty of the construction work, etc. The results are shown in the table below. According to the table, Alternaive A is preferable.

Table 6.1.2 Comparison of Alternatives

Comparison Items \ Alternative	A (Fig. 6.1.1)	B (Fig. 6.1.2)	C (Fig. 6.1.3)
Construction Cost Ratio to Alternative A	1.0	0.8	1.3
Supplying Capacity of the Materials *	○	○	△
Easiness of Construction Work	○	△	○
Structural Stability	○	○	○
Function as a Breakwater	○	○	○
Total Evaluation (Priority)	1	2	3

Key ○ - good

△ - some difficulty

* The quantity of 20 ton rocks is very limited.

6-2 Reclamation of the Container Yard

13. In this sub-section, the preliminary design of the retaining wall for the extension of the container yard and the stability of slope is outlined.

1) Design Conditions

14. ① Crown level : D.L. + 3.0m

② Design depth : D.L. -11.0m

③ Tidal level : H.W.L. + 1.0m

: L.W.L. ± 0.0m

④ Residual water level : D.L. + 0.33m

⑤ Seismic coefficient : Kh = 0.15

⑥ Surcharge : 1.4 t/m^2 (Ordinary)

: 0.7 t/m^2 (Earthquake)

⑦ Soil conditions

backfilling material: $\gamma = 1.8t/m^3 = 1.0ton/m^3$, $\phi = 40°$

rubbles : $\gamma = 1.8t/m^3 = 1.0ton/m^3$, $\phi = 40°$,

qta = $50t/m^2$

cohesion : $1.0t/m^2$ (-11.0m ~ -13.0m)

: $2.2t/m^2$ (-13.0m ~ -16.0m)

⑧ Friction coefficient

concrete against concrete: 0.5

concrete against rubble : 0.6

⑨ Allowable safety factor

against sliding : 1.2 (Ordinary), 1.0 (Earthquake)

against overturning : 1.2 (Ordinary), 1.0 (Earthquake)

against circular failure : 1.3 (Ordinary)

2) Stability of the Retaining Wall

15. The results of the stability calculation for the cross section shown in Fig. 6.2.1 are summarized in Table 6.2.1.

3) Stability of the Slope

16. The bearing capacity of the soft clay layer (-11m - -16m) is not strong enough for the toe pressure of the retaining wall. And the circular failure will be occurred through the soft clay layer as shown in Fig. 6.2.2. As the soft layer is relatively thin, displacement method is recommended for soil stabilization of the foundation. The cross section which the soft layer is displaced shown in Fig. 6.2.1 is stable enough for the circular failure.

Table 6.2.1 Calculation Results

	Item	Evaluation					
		Normal			Earthquake		
Examination at ± 0.0m	Safety against Sliding	2.2	1.2	GOOD	1.2	1.0	GOOD
	Safety against Overturning	4.0	1.2	GOOD	2.1	1.0	GOOD
Examination at - 2.0m	Safety against Sliding	2.4	1.2	GOOD	1.1	1.0	GOOD
	Safety against Overturning	5.2	1.2	GOOD	2.3	1.0	GOOD
Examination at - 4.0m	Safety against Sliding	3.1	1.2	GOOD	1.2	1.0	GOOD
	Safety against Overturning	6.0	1.2	GOOD	2.3	1.0	GOOD

6-3 Ferry Terminal

17. In this sub-section, the preliminary designs of the quaywall and the dolphin for the ferry terminal are outlined.

1) Quaywall

(1) Design Conditions

① Crown level : D.L. + 1.7m

② Design depth : D.L. - 3.5m

③ Tidal level : H.W.L. + 1.0m

: L.W.L. ± 0.0m

④ Residual water level : D.L. + 0.33m

⑤ Seismic coefficient : Kh = 0.15

⑥ Surcharge : 1.0 t/m^2 (Ordinary)

: 0.5 t/m^2 (Earthquake)

⑦ Objective ship tonnage: 700 GRT (Queen Salamasina Class)

⑧ Soil conditions

backfilling material: $\gamma = 1.8t/m^3$ $\gamma' = 1.0t/m^3$, $\phi = 40°$

rubble : $\gamma = 1.8t/m^3$ $\gamma' = 1.0t/m^3$, $\phi = 40°$

: $qta = 50t/m^2$

⑨ Friction coefficient

concrete against concrete: 0.5

concrete against rubble : 0.6

⑩ Allowable safety factor

against sliding : 1.2 (Ordinary), 1.0 (Earthquake)

against overturning : 1.2 (Ordinary), 1.0 (Earthquake)

(2) Stability Calculation

18. The results of the stability calculation for the cross section shown in Fig. 6.3.1 are summarized in the following table.

Table 6.3.1 Calculation Results

	Item	Evaluation	
		Normal	Earthquake
Examination at - 1.0m	Safety against Sliding	1.38 > 1.2 GOOD	1.10 > 1.0 GOOD
	Safety against Overturning	1.62 > 1.2 GOOD	2.26 > 1.0 GOOD
Examination at - 2.5m	Safety against Sliding	1.80 > 1.2 GOOD	1.01 > 1.0 GOOD
	Safety against Overturning	2.48 > 1.2 GOOD	1.98 > 1.0 GOOD
Examination at - 4.0m	Safety against Sliding	2.65 > 1.2 GOOD	1.21 > 1.0 GOOD
	Safety against Overturning	3.47 > 1.2 GOOD	2.00 > 1.0 GOOD
	Toe Pressure (t/m^2)	14.97 > 50 GOOD	21.44 > 50 GOOD

2) Dolphin

(1) Design Conditions

19. ① Crown level : D.L. + 2.0m

② Design depth : D.L. - 4.0m

③ Tidal level : H.W.L. + 1.0m

: L.W.L. ± 0.0m

④ Residual water level : D.L. + 0.33m

⑤ Objective ship tonnage: 700 GRT (Queen Salamasina Class)

⑥ Soil condition : N-value = 30

(2) Design of Piles

20. i) Assumptions

. dimension of steel pipe pile

D = 500 mm, t = 12 mm

. corrosion rate

0.1 mm/Year

ii) Load Conditions

(tons)

Vertical Force			Horizontal Force		
Dead Weight of Superstructure	Dead Weight of Steel Pipe Pile	Surcharge	Seismic Force	Tractive Force	*Berthing Force
50.7	1.0	0.0	8.2	20.0	18.3

* v = 0.15m/sec.

fender type : SA200H, l = 2.5m

iii) Moment and Axial Force for Each Pile

(tons)

	Moment	Axial Force	Axial Force by
	by Horizontal Force		Vertical Force
Northward Pile	-19.31	15.45	13.71
Southward Pile	-19.31	-15.45	13.71

iv) Stress of Piles

(a) Allowable Stress (assumed STK 41 grade)

axial tensile stress $\sigma_{ta} = 1400\ kg/cm^2$

axial compressive stress $\sigma_{ca} = 1362\ kg/cm^2$ ($\ell/r = 24.5$)

bending tensile stress $\sigma_{bta} = 1400\ kg/cm^2$

bending compressive stress $\sigma_{bca} = 1400\ kg/cm^2$

(b) Stress of Piles

a) Northward Piles

$$\frac{\sigma_c}{\sigma_{ca}} + \frac{\sigma_{bc}}{\sigma_{ba}} = 1.0$$

b) Southward Piles

$$\sigma_t + \sigma_{bt} = 1196\ kg/cm^2 < \sigma_{ta} = 1400\ kg/cm^2$$

$$-\sigma_t + \sigma_{bc} = 1170\ kg/cm^2 < \sigma_{ba} = 1400\ kg/cm^2$$

v) Embedded Length for Lateral Resistance

$3/\beta = 5.94$ m   Then, $\ell = 7.0$ m is recommended.

vi) Bearing Capacity of Pile Foundation

(tons)

Allowable Bearing Capacity	Allowable Pulling Resistance
Ra = 118 ton > P1 = 29 ton good	Ra = 21.7 ton > P2 = 1.7 ton good

vii) Pile Head Displacement

$$\Delta = \frac{H}{K_H} = 1.45 \text{ cm} < 10.0 \text{ cm}$$

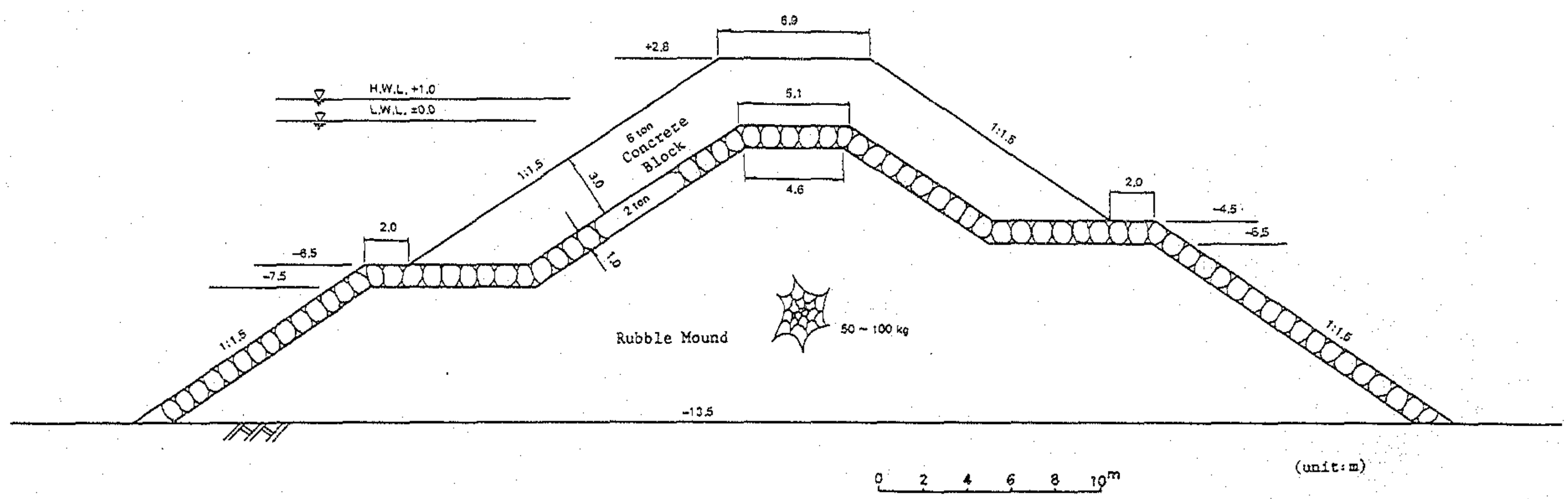


Fig. 6.1.1 Cross Section of Breakwater (Alternative A)

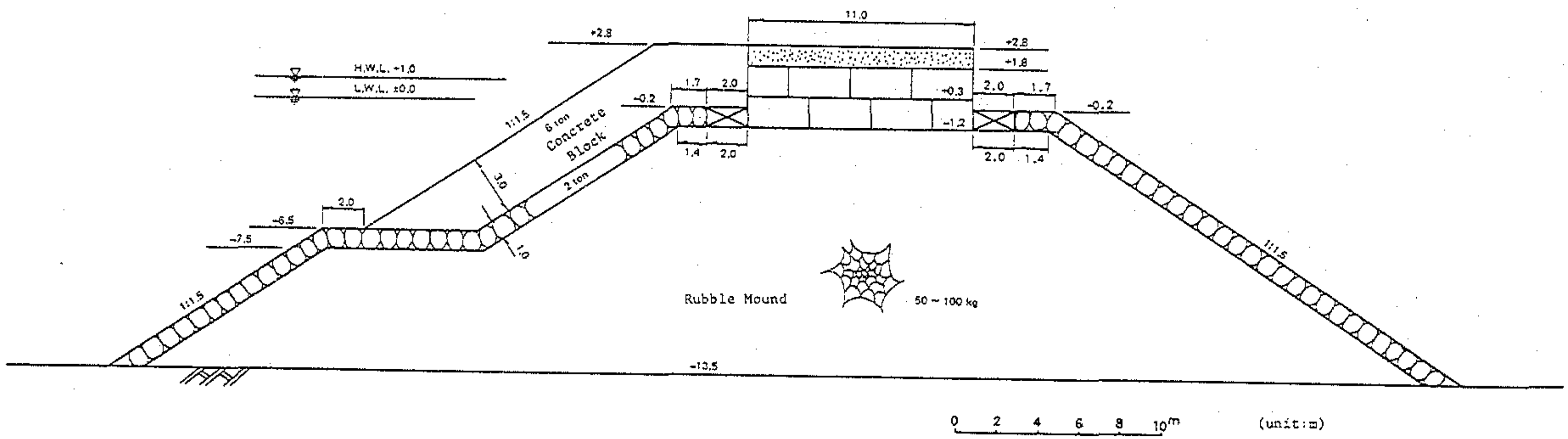


Fig. 6.1.2 Cross Section of Breakwater (Alternative B)

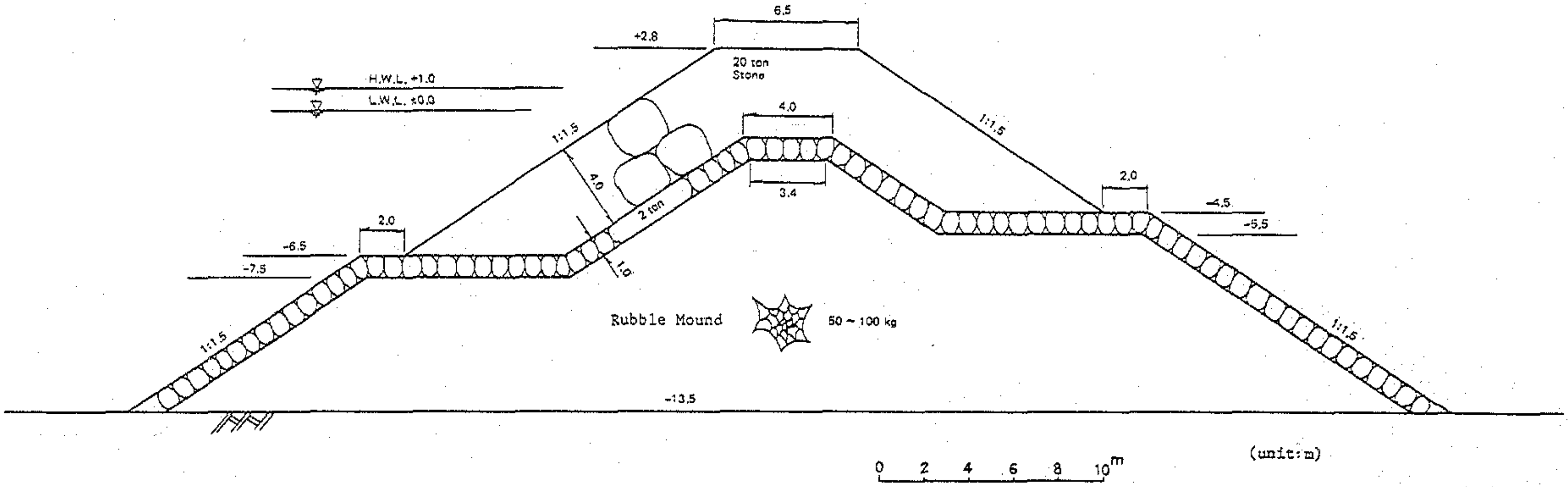


Fig. 6.1.3 Cross Section of Breakwater (Alternative C)

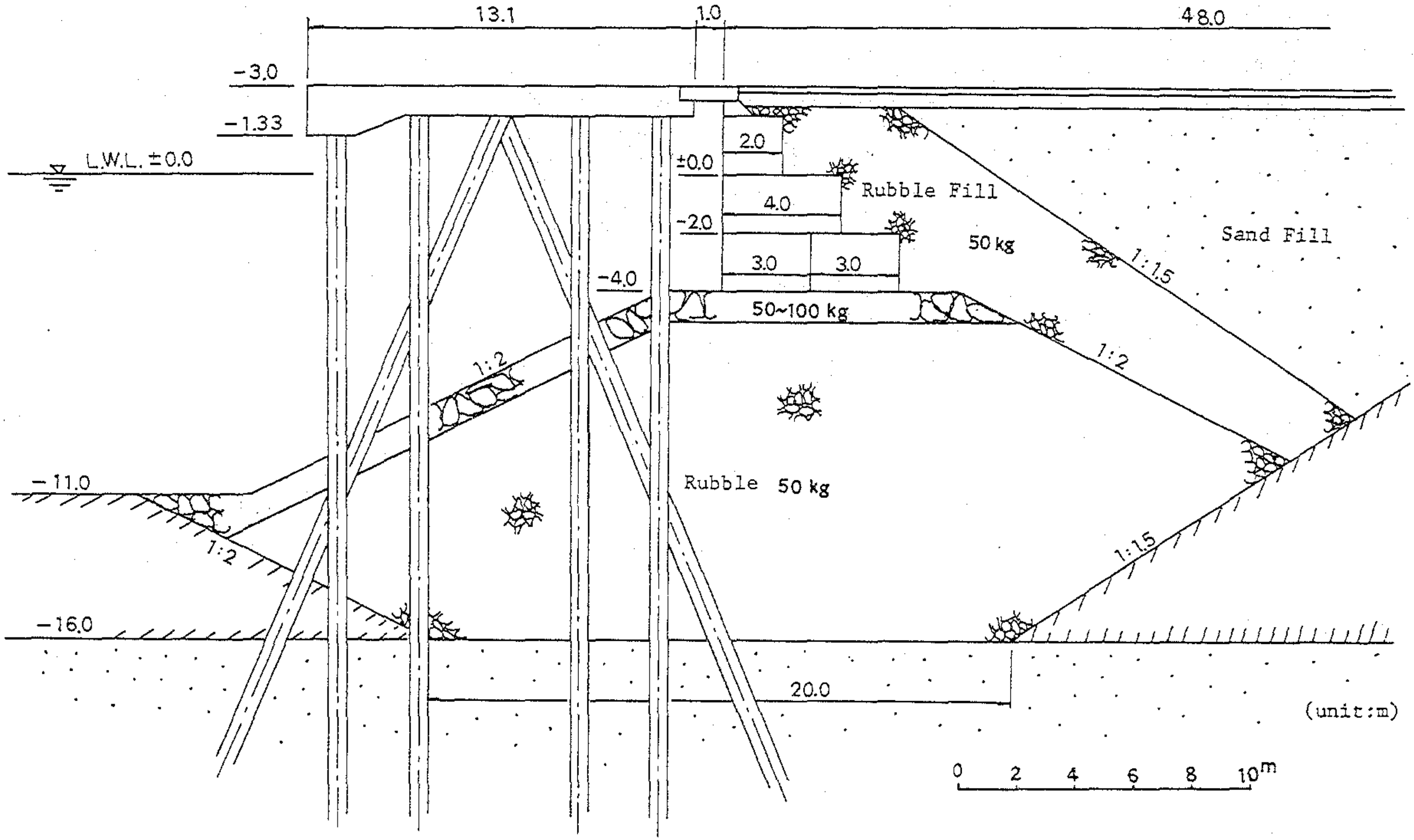


Fig. 6.2.1 Cross Section of Expanded Container Yard

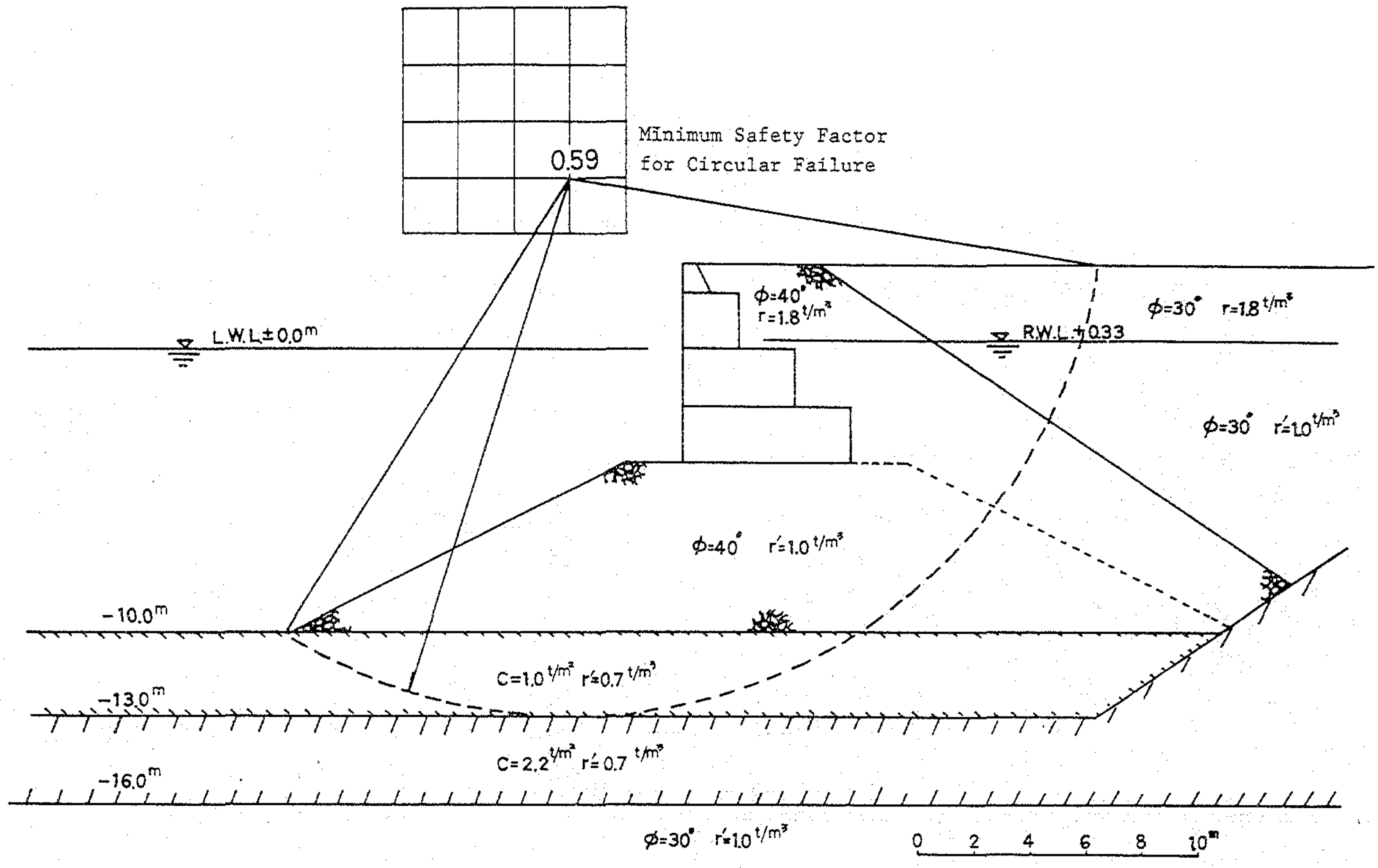


Fig. 6.2.2 Examination of the Stability of Slope

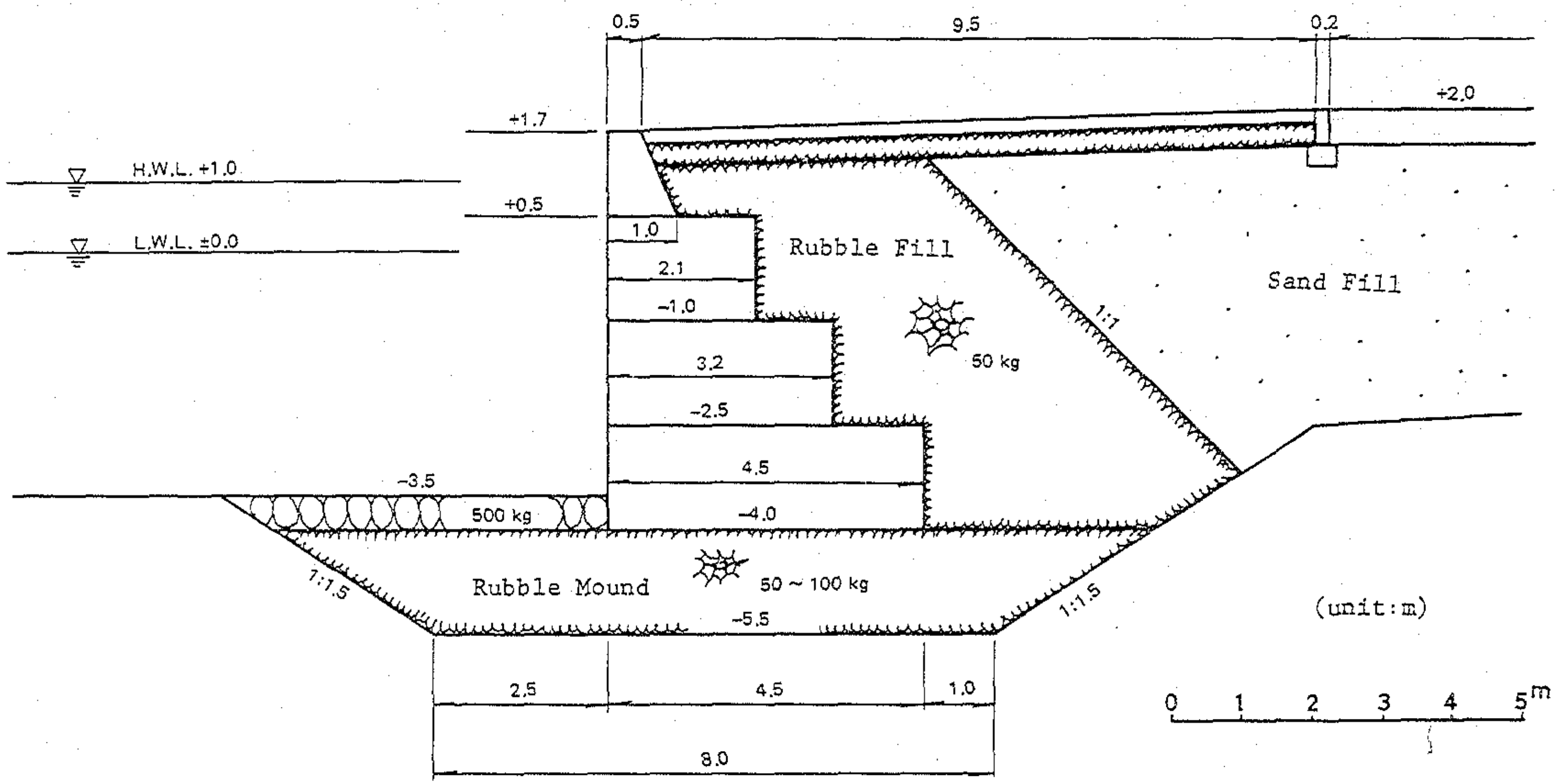


Fig. 6.3.1 Cross Section of Quaywall

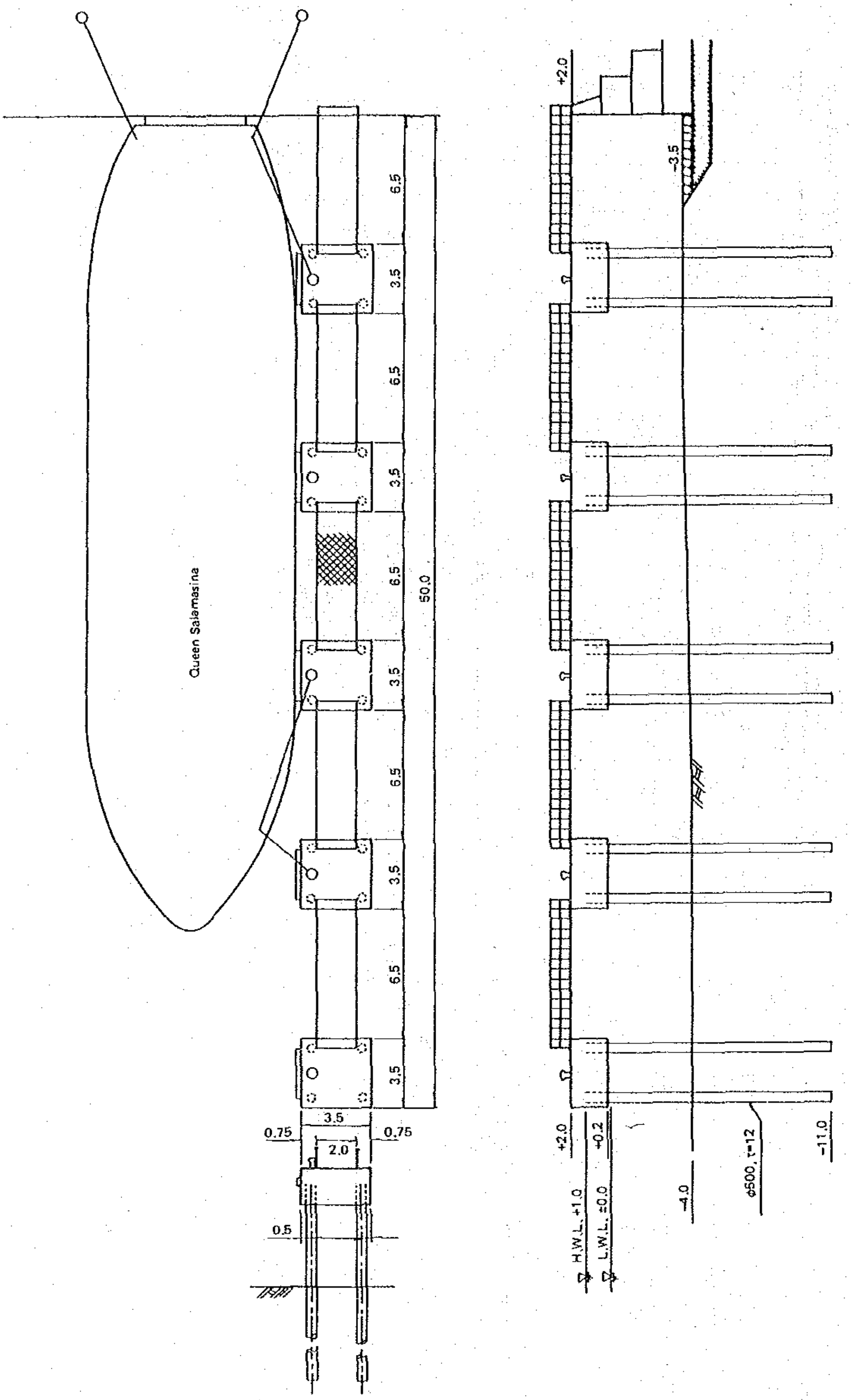


Fig. 6.3.2 Dolphin for Ferry Terminal

CHAPTER 7

CONSTRUCTION PLAN AND COST ESTIMATION

Chapter 7 Construction Plan and Cost Estimation

1. This chapter describes the construction plan of the First Stage Plan and the cost estimation of the Master Plan and the First Stage Plan.

7-1 Construction Plan for the First Stage Plan

2. The First Stage Plan of Apia Harbour includes the following major project items:

① Repair work of the existing main wharf
② Construction of the breakwater
③ Construction of the ferry terminal
④ Expansion of the container yard
⑤ Purchase of a tugboat
⑥ Lighting of the existing mooring buoys
⑦ Engineering services

3. In planning, special attention has been paid to minimize the utilization of foreign materials and equipment and to maximize the use of locally available resources. Further, the construction method has been selected through consideration of the local conditions.

1) Detailed Design

4. Before the actual construction work begins, a detailed engineering study will be conducted. The detailed engineering services are scheduled to commence in 1988 and will be completed within 7 months from the starting date. In this period, detailed field investigations will be carried out on soil conditions, corrosion of the H-shaped steel piles of the existing main wharf, construction material availability, etc.

5. Since the project will involve large-scale stone works for the construction of the breakwater, the ferry terminal area and the container yard, special attention should be paid to investigation of a quarry site, the physical property of the stone, production rate, transportation method/distance and material cost.

6. The soil survey carried out for this study is not detailed enough for the detailed design services, and a further boring investigation covering the planned project area should be conducted. Also, a further detailed corrosion survey on the steel piles of the main wharf should be conducted for planning the countermeasures against deterioration of the wharf.

7. A detailed structural design will be prepared for all the structures included in the First Stage Plan. Then the project cost will be estimated in detail based on the bill of quantity and the construction schedule. Special attention will be paid to ensure that the construction work does not interfere with regular operations.

8. A set of tender documents will be prepared. The tendering procedure is estimated to take about 5 months until the contract is awarded.

2) Mobilization

9. After the award of the contract, the construction material and equipment will be mobilized to the project site within a period of about 3 months including preparation, transportation and customs clearance of the construction material/equipment.

3) Repair of the Main Wharf

10. The repair works of the existing main wharf consist of anti-corrosion measures to prolong the remaining life of the wharf and repair/renewal of curbings and rubber fenders to improve the safety.

11. According to the underwater survey on the steel piles, the concrete sleeve has spalled off and the steel piles are exposed in some cases. To ensure the stability of the piles in the future, adequate anti-corrosion measures are imperative and an aluminum galvanic method is adopted considering the ease of installation and maintenance. The alminum section will be welded to the cleaned surface of the steel piles before the formation of the rubble slope.

12. According to the visual survey, ten rubber fenders are damaged and

nine curbings are missing as mentioned in the former chapter. They will be repaired/renewed in the early stage of the project.

13. The repair works of the main wharf will take three months.

4) Construction of the Breakwater

14. The construction of the breakwater is one of major items of this project. This work involves approximately 60,000 m^3 of stone material. The core of the breakwater consists of small size stone weighing 50 - 100 kg and this mound will be covered by an armour layer of 1 ton rubble. The crown of the breakwater will be protected by wave dissipating concrete blocks weighing 6 tons.

15. The stone materials can be supplied from coconut and cacao plantations. The quantity of the material available in the plantations is roughly estimated as 100 m^3/hectare, and therefore the required total area is 6 km^2.

16. The plantations' supply capacity of the stone material is sufficient. However, the stone is scattered over the surface of the ground. Therefore, special attention should be paid to methods and equipment for collection and transportation of the material.

17. The crown of the breakwater will be protected by wave dissipating concrete blocks weighing 6 tons. Special attention should be paid to the method of curing the concrete in hot weather.

18. The construction work is scheduled to be executed sequentially for the above three layers. The core mound will be covered by the 1 ton layer immediately after the formation of the slope and the concrete block layer will be set immediately after the armour layer is in place. The concrete blocks will be placed by a 50 ton class crane mounted on a pontoon.

19. A stock yard for the stone materials will be prepared near the port entrance in order not to disturb the cargo handling operations in the port yard. Then the materials will be loaded onto a barge from a temporary

loading jetty and carried to a dumping location as directed by a diving boat. The total volume of stone required for this work is on the order of 60,000 m^3 and the construction period is estimated at about 10 months assuming a monthly stone supply rate of about 6,000m^3.

5) Construction of the Ferry Terminal

20. The construction work of the ferry terminal includes five sets of berthing and mooring dolphins, a 45 m long gravity type quay wall, a two story terminal building with a floor area of 710 m^2 and pavement of 3600m^2.

21. The work will be completed within a 13 month period and the terminal will be in service 10 months after the commencement of the project. The construction schedule of the ferry terminal is planned so that it will not interrupt the present ferry service, i.e. the existing ferry ramp will be demolished at the final stage of the construction of the new terminal after the ramp and the berthing/mooring dolphins are ready for service.

6) Expansion of the Container Yard

22. Reclamation of the water area behind the main wharf is scheduled to commence at the initial stage of the construction period. This is so scheduled in order to secure the required settlement period of the reclaimed land area.

23. As mentioned in the former chapter, the top layer of the sea bed behind the main wharf is poor in strength and should be removed to avoid subsidence and to secure stability against circular slip at the front rubble mound slope. The work will be scheduled in such a way that the interference with regular cargo handling operations on the wharf and the access bridges will be minimized.

24. Demolition of the northern access bridge will be done at the first stage and dredging and reclamation works will be started at the northern area enclosed by the wharf, the two access bridges and the existing rubble slope. After the completion of the northern revetment for alternative access from the existing container yard to the wharf, the southern access

bridge will be demolished and dredging and reclamation works will be continued.

25. The reclamation work will executed by grab dredger, crawler crane mounted on a pontoon. This pontoon will also be used for the concrete block placing work of the retaining wall.

26. A retaining rubble slope mound will be formed under and adjacent to the existing main wharf, and in this work special attention should be paid not to damage the steel piles of the existing main wharf. The placing work of stone is planned to be executed by chutes and small flat barges.

27. The reclamation work does not involve any major construction equipment and can be started by stocking the fill material. Therefore the work is scheduled to commence two months after the contract begins, before the mobilization will be completed. The filling stone will be moved and graded by a bulldozer. The wharf and the reclaimed area will be connected by precast concrete slabs.

28. The pavement of the reclaimed land area will be carried out after settlement of the reclaimed area to avoid nonuniform subsidence of the pavement.

7) Tugboat

29. The specifications of the tugboat to be purchased are as follows.

Tonnage	180 GT
Engine Power	1,500 HP (750 HP x 2)
Propellers	Twin

30. The tugboat will be ordered immediately after the contract is signed, and the shipbuilding and delivery to the port will take about 6 months.

8) Lighting of the Existing Mooring Buoys

31. The existing mooring buoys are not lighted, and will be lighted to avoid possible collisions at nighttime.

9) Construction Equipment

32. For the execution of the project, many kind of construction equipment are required. The main construction equipment to be used is listed below.

	Machine	Capacity	Number
①	Crawler Crane	50 t	1
②	Crawler Crane	35 t	1
③	Truck Crane	15 t	1
④	Bulldozer	11 t	1
⑤	Tractor Shovel	1.8 m^3	1
⑥	Motor Crawler	3.1 m	1
⑦	Tire Roller	8 - 20 t	1
⑧	Macadam Roller	19 - 20 t	1
⑨	Asphalt Finisher	2.4 - 4.5 m	1
⑩	Dump Truck	8 - 11 t	6 - 10
⑪	Vibro Hammer	40 KW	1
⑫	Concrete Plant	0.5 m^3	1
⑬	Asphalt Plant	20 t	1
⑭	Pontoon	500 t	1
⑮	Pontoon	350 t	1
⑯	Soil Barge	300 m^3	2
⑰	Tug Boat	300 ps	1
⑱	Anchor Boat	100 ps	1
⑲	Diving Boat	30 ps	2

10) Construction Materials

33. The main construction materials to be used for the project are listed below.

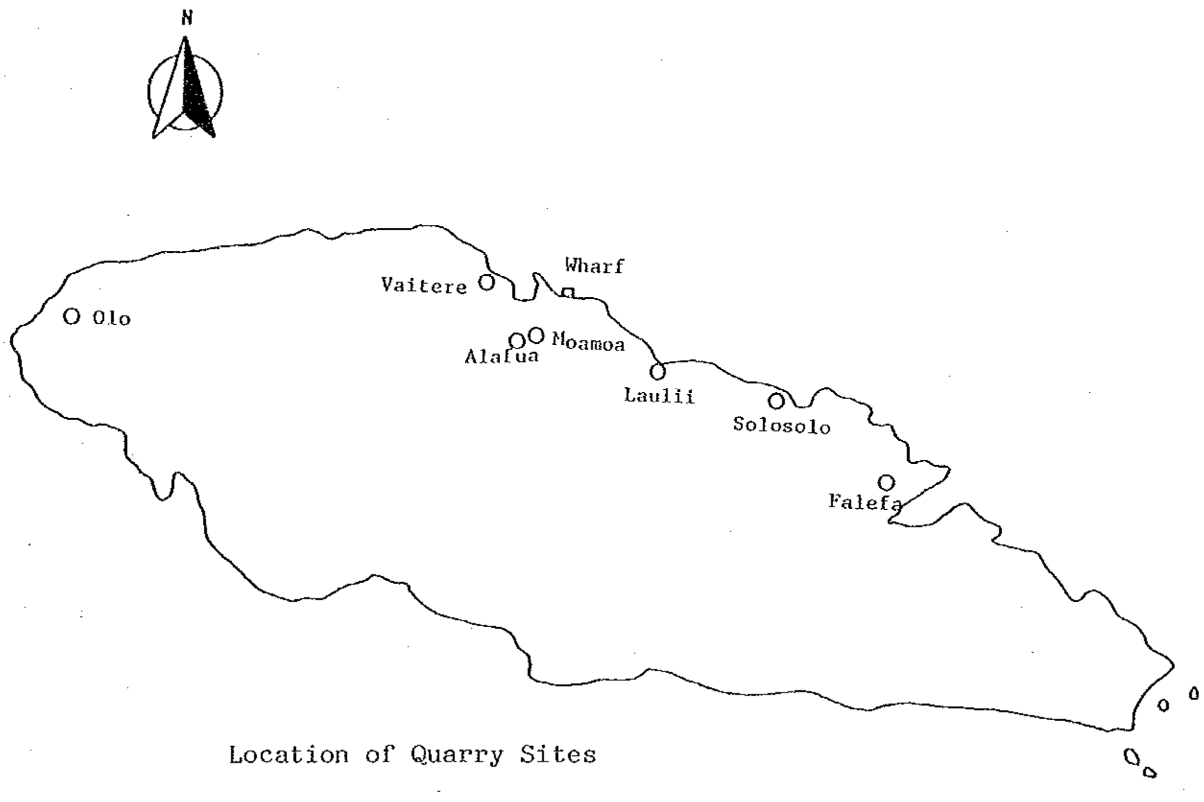
	Material	Quantity
①	Sand/Stone	130,000 m^3
②	Cement	2,400 t
③	Asphalt	280 t
④	Steel Pile	30 t
⑤	Steel Bar	80 t

⑥ Heavy Oil 270 kl
⑦ Light Oil 190 kl

34. As mentioned before, the project involves large-scale stone works. Major quarry sites in Upolu Island are listed below.

	Site	Material	Distance from the Wharf (km)
①	Moamoa	Agrregate and Crusher Run	8
②	Alafua	"	8
③	Olo	"	40
④	Falefa	"	32
⑤	Puipaa	Rubble Mound	10
⑥	Laulii	Scoria for Fill	13
⑦	Solosolo	Sand for Concrete	16
⑧	Vaitele	Sand for Fill	8

35. The quarry sites are noted on the following map.



Location of Quarry Sites

11) Construction Schedule

36. The construction schedule is shown in Fig. 7.1.1. The total project period is estimated at three years from the detailed engineering study to the completion of the construction work, and the actual construction work is estimated to be completed within a two year period. All the facilities will be in service in 1991.

37. In the above planning, the total number of working days per year is assumed to be 250 days excluding holidays and nonworkable days due to adverse weather. The total construction period is governed by the supply condition of stone material, and in planning the construction schedule special attention has been paid to spreading the project items requiring stone material evenly over the entire construction period.

Table 7.1.1 Construction Schedule for the First Stage Plan of Apia Port

No.	Description	Unit	Quantity	1 st Year						2 nd Year						3 rd Year					
				2	4	6	8	10	12	2	4	6	8	10	12	2	4	6	8	10	12
1.	Repair of Main Wharf	L.S.	1																		
2.	Breakwater																				
	(1) Rubble Mound	m^3	50,800																		
	(2) Armour Stone(1 ton)	m^3	10,500																		
	(3) Concrete Block(6 ton)	pcs	1,670																		
	(4) Lighthouse	pcs	1																		
3.	Ferry Terminal																				
	(1) Reclamation	m^3	10,300																		
	(2) Quaywall	m	45																		
	(3) Dolphin	m	50																		
	(4) Terminal Building	m^2	600																		
	(5) Pavement	m^2	3,600																		
4.	Expansion of Container Yard																				
	(1) Reclamation	m^3	59,000																		
	(2) Pavement	m^2	6,000																		
5.	Tugboat (1500 HP)	pcs	1																		
6.	Buoy Lighting	pcs	4																		
7.	Mobilization and Demobilization	L.S.	1																		
8.	Detailed Design	L.S.	1																		
9.	Construction Supervision	L.S.	1																		

7-2 Cost Estimation for the First Stage Plan

38. Fig. 7.2.1 shows the layout of the First Stage Plan. The detailed project cost and the annual investment cost are shown in Table 7.2.1 and Table 7.2.2. The prerequisites of the cost estimation are presented below.

1) Exchange Rates

39. The exchange rates between various currencies are based on the official rates at the time of the cost estimation are as follows:

1.0 WS$ = 0.48 US$

1.0 WE$ = 72 Japanese Yen

All the project costs are indicated in Western Samoa Dollars (WS$).

2) Tax Exemption

40. It is assumed that no import tax is levied on the construction material and equipment brought in from overseas.

3) Physical Contingency

41. The physical contingency is set at 15% for civil works. No contingency is assumed for building works and engineering services, and no contingency is made for inflation.

4) Project Cost

42. The main construction materials required for the project are various sizes of stone for the breakwater and reclamation work. It is confirmed that the stone material is locally available at a reasonable price and an acceptable daily supply rate for this project. This is also true for the sand and aggregate for concrete work. However, all the steel materials are not locally available, and must be imported.

43. The heavy construction equipment required for this type of large-scale marine work is not locally available and must be mobilized from overseas. A limited number of construction machines such as dump trucks and mobile

cranes are locally available.

44. The project cost has been estimated on the basis of the unit prices and quantities of each project item according to the construction schedule. The cost has been estimated for the foreign and local currency portions separately.

The local portion includes the cost of stone material, fuel and minor construction equipment.

45. As shown in Table 7.2.1, the total project cost is estimated at about 23 million WS$ consisting of about 16 million WS$ for the foreign portion (70%) and about 7 million WS$ for the local portion (30%). The four major items of the project comprise about 70% of the total project cost i.e. 22% for the breakwater, 14% for the ferry terminal, 22% for the expansion of the container yard and 12% for the tugboat.

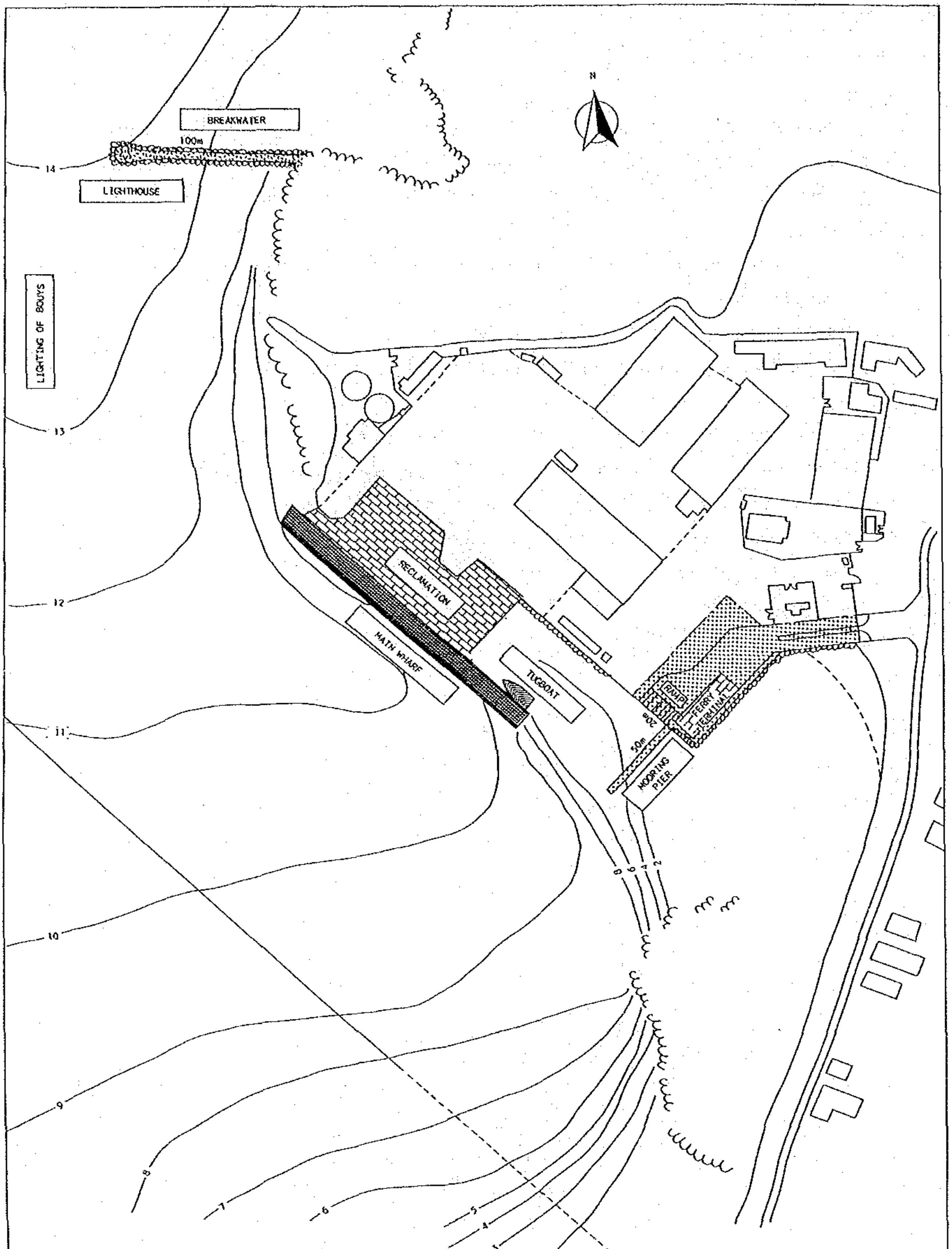


Fig. 7.2.1 First Stage Plan, Apia Port

Table 7.2.1 Breakdown of Construction Cost for the First Stage Plan

(Unit: 1,000 WS$)

		* Quantity	* U.Cost	* Forign	* Local	* Total
* Wharf Repaire				477	19	496
Anti-corrosion	PC	140	1.90	255	11	266
Fender	PC	10	22.40	217	7	224
Curbing	PC	10	.60	5	1	6
* Breakwater				2,590	2,382	4,972
Rubble Mound	M3	50,800	.05	1,087	1,352	2,439
Armour Stone	M3	10,500	.06	357	325	682
Armour Block	PC	1,668	1.07	1,080	704	1,784
Light House	PC	1	67.00	66	1	67
* Ferry Terminal				2,304	864	3,168
Dredging	M3	2,150	.02	34	11	45
Foundation	M3	1,350	.07	53	38	91
Quay	M	50	5.04	128	124	252
Backfill	M3	1,240	.05	32	34	66
Revetment	M3	2,780	.05	61	74	135
Reclamation	M3	4,960	.02	23	95	118
Pavement	M2	3,600	.06	55	172	227
Misc	LS	1	184.00	178	6	184
Building	M2	710	2.47	1,485	267	1,752
Dolphin	PC	5	59.60	255	43	298
* Container Yard				2,470	2,610	5,080
Access Dmlsh	M3	250	.16	23	18	41
Dredging	M3	32,400	.03	632	198	830
Foundation	M3	34,900	.05	872	833	1,705
Retaining Wall	M	170	5.74	493	483	976
Backfill	M3	4,500	.05	86	118	204
Reclamation	M3	20,400	.02	97	392	489
Pavement	M2	6,000	.11	166	518	684
Access Slab	M	120	1.26	101	50	151
* Tug Boat	PC	1	2,740.00	2,740	0	2,740
* Buoy Lighting	PC	4	4.00	16	0	16
* Mobilization	LS	1	2,877.00	2,877	0	2,877
S. Total				13,474	5,875	19,349
* Detailed Design	LS	1	712.00	678	34	712
* Supervision	LS	1	630.00	630	0	630
* Contingency	LS	1	2,057.00	1,176	881	2,057
S. Total				2,484	915	3,399
G. Total				15,958	6,790	22,748

Table 7.2.2 Construction Cost for the First Stage Plan

(Unit: 1,000 WS$)

Item.	1st Year						2nd Year						3rd Year						Total					
	For-eign	Local				Total	For-eign	Local				Total	For-eign	Local				Total	For-eign	Local				Total
		Other	U.S.L	S.L.	Total			Other	U.S.L	S.L.	Total			Other	U.S.L	S.L.	Total			Other	U.S.L	S.L.	Total	
1 Wharf repair	0	0	0	0	0	0	477	11	0	8	19	496	0	0	0	0	0	0	477	11	0	8	19	496
2 Breakwater	0	0	0	0	0	0	259	215	10	13	238	497	2,331	1,931	94	119	2,144	4,475	2,590	2,146	104	132	2,382	4,972
3 Ferry terminal	0	0	0	0	0	0	1,152	296	49	87	432	1,584	1,152	296	49	87	432	1,584	2,304	592	98	174	864	3,168
4 Yard expansion	0	0	0	0	0	0	1,235	1,161	60	84	1,305	2,540	1,235	1,161	60	84	1,305	2,540	2,470	2,322	120	168	2,610	5,080
5 Tug boat	0	0	0	0	0	0	2,740	0	0	0	0	2,740	0	0	0	0	0	0	2,740	0	0	0	0	2,740
6 Buoy lighting	0	0	0	0	0	0	16	0	0	0	0	16	0	0	0	0	0	0	16	0	0	0	0	16
7 Mobilization	0	0	0	0	0	0	2,877	0	0	0	0	2,877	0	0	0	0	0	0	2,877	0	0	0	0	2,877
S. Total	0	0	0	0	0	0	8,756	1,683	119	192	1,994	10,750	4,718	3,388	203	290	3,881	8,599	13,474	5,071	322	482	5,875	19,349
8 Detailed design	678	0	0	34	34	712	0	0	0	0	0	0	0	0	0	0	0	0	678	0	0	34	34	712
9 Supervision	0	0	0	0	0	0	315	0	0	0	0	315	315	0	0	0	0	315	630	0	0	0	0	630
10 Contingency	0	0	0	0	0	0	468	252	18	29	299	767	708	508	30	44	582	1,290	1,176	760	48	73	881	2,057
S. Total	678	0	0	34	34	712	783	252	18	29	299	1,082	1,023	508	30	44	582	1,605	2,484	760	48	107	915	3,399
G. Total	678	0	0	34	34	712	9,539	1,935	137	221	2,293	11,832	5,741	3,896	233	334	4,463	10,204	15,958	5,831	370	589	6,790	22,748

Abbreviation; U.S.L.: Unskilled Labor, S.L.: Skilled Labor

7-3 Cost Estimation for the Master Plan

46. The total project cost of the Master Plan for Apia, Asau, Salelologa and Mulifanua Harbours is tabulated in Table 7.3.2 - 7.3.4 and summarized in Table 7.3.1 below.

Table 7.3.1 Total Construction Cost of Master Plan

	Name of Harbour	Total Cost (1,000 WS$)
1.	Apia	85,616
2.	Asau (Alternative A)	19,609
3.	Salelologa and Mulifanua	4,358
	G. Total	109,583

47. Fig. 7.3.1 shows the layout of the Master Plan for Apia Harbour. The major project items for Apia Harbour under the Master Plan are the construction of the new wharf and the container terminal, and the total project cost is estimated at about 86 million WS$.

48. For Asau Harbour, the extension of the breakwater and the dredging work of the approach channel are the major items, and the total project cost is estimated at about 20 million WS$.

49. Salelologa and Mulifanua Harbours will require a total project cost of about 4 million WS$ mainly for the improvement of the existing ferry terminals.

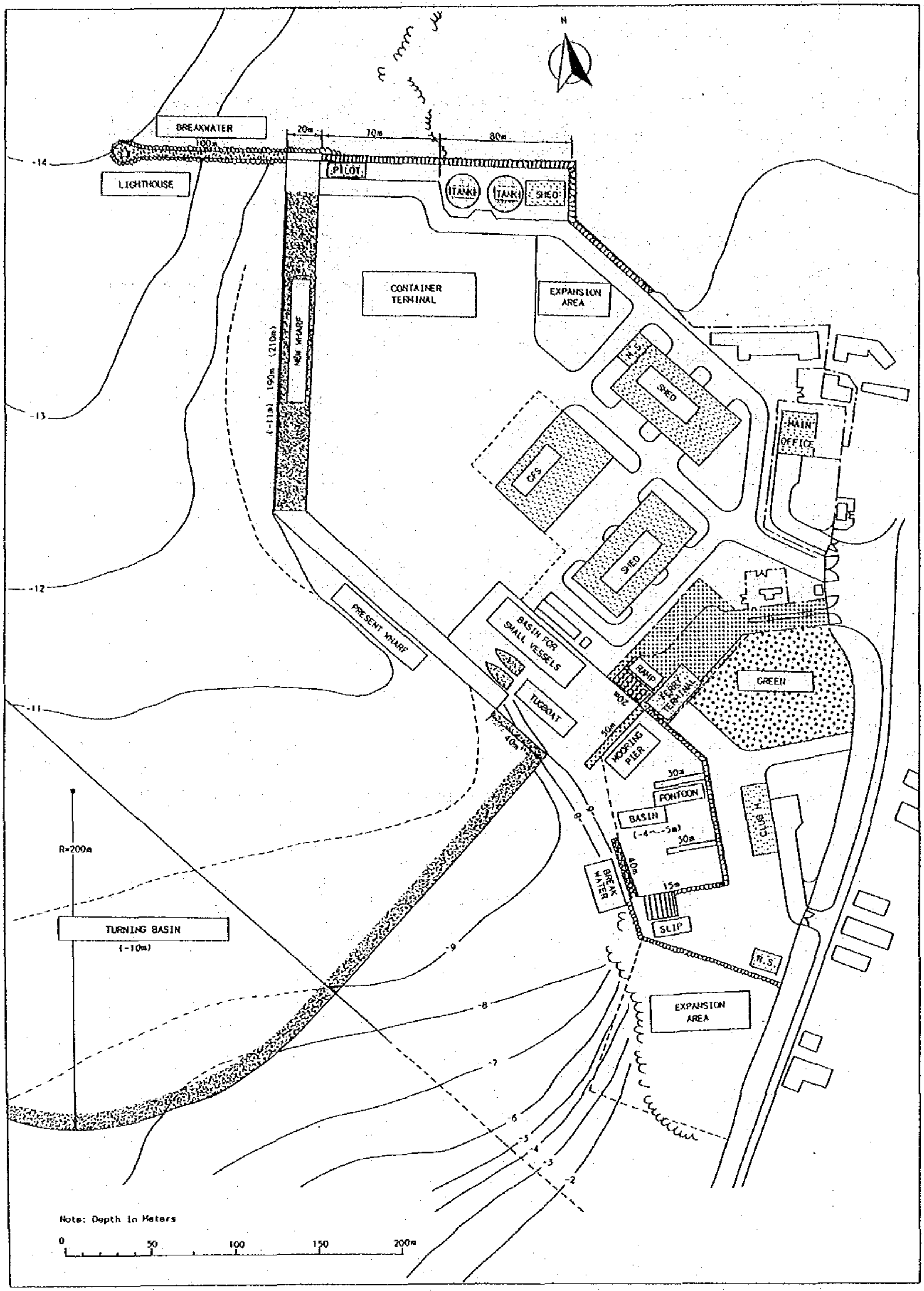


Fig. 7.3.1 Master Plan 2005, Apia Port

Table 7.3.2 Project Cost of Master Plan (Apia Harbour)

No.	Item	Unit	Quantity	Unit Cost WS$	Amount 1,000 WS$
1	Dredging	m^3	110,000	17	1,870
2	Breakwater	m	100	49,700	4,970
3	Wharf Repair	LS	1	496,000	496
4	New Wharf -11m	m	210	122,000	25,620
5	Ferry Terminal	m^2	3,600	880	3,168
6	Small Vessel Wharf	m	100	21,300	2,130
7	Buoy Lighting	LS	1	16,000	16
8	Buoy Resiting	LS	1	250,000	250
9	Container Yard	m^2	6,000	850	5,100
10	Container Terminal	m^2	25,000	130	3,250
11	Beacon Upgranding	PC	2	70,000	140
12	Marina	m^2	10,000	240	2,400
13	Green Area	m^2	5,000	90	450
14	C.F.S.	m^2	1,200	1,700	2,040
15	Maintenance Shop	m^2	200	1,400	280
16	Transit Shed	m^2	5,000	1,100	5,500
17	Main Office	m^2	1,500	2,700	4,050
18	Pilot Office	m^2	200	2,400	480
19	Co. Oil Tank & Shed	LS	1	463,000	463
20	Tug Boat	PC	2	2,740,000	5,480
21	Mobilization	LS	1	6,850,000	6,850
	S. Total				75,003
22	E. Services	LS	(1-19)x0.05		3,134
23	Contingency	LS	(1-13)x0.15		7,479
	S. Total				10,613
	G. Total				85,616

Table 7.3.3 Project Cost of Master Plan (Asau Harbour)

No.	Item	Unit	Quantity	Unit Cost WS$	Amount 1,000 WS$
	(Alternative A)				
1	Dredging of Channel	m^3	320,000	17	5,440
2	Dredging of Basin	m^3	420	170	71
3	Breakwater Extension	m	200	34,250	6,850
4	Open Storage Yard	m^2	5,730	113	647
5	Navigational Aids	LS	1	290,000	290
6	Mobilizaion	LS	1	3,650,000	3,650
	S. Total				16,949
7	E. Services	LS	(1-5)x0.05		665
8	Contingency	LS	(1-5)x0.15		1,995
	S. Total				2,660
	G. Total				19,609
	(Alternative B)				
1	Dredging of Channel	m^3	700,000	17	11,900
2	Dredging of Basin	m^3	420	170	71
3	Breakwater Extension	m	200	34,250	6,850
4	Open Storage Yard	m^2	4,800	540	2,592
5	Navigational Aids	LS	1	290,000	290
6	Mobilizaion	LS	1	3,650,000	3,650
	S. Total				25,353
7	E. Services	LS	(1-5)x0.05		1,085
8	Contingency	LS	(1-5)x0.15		3,256
	S. Total				4,341
	G. Total				29,694

Table 7.3.4 Project Cost of Master Plan
(Salelologa and Mulifanua Ports)

No.	Item	Unit	Quantity	Unit Cost WS$	Amount 1,000 WS$
	(Salelologa Port)				
1	Parking Area	m^2	3,500	320	1,120
2	Dredging	m^3	170	260	44
3	Navigational Aids	LS	1	260,000	260
	S. Total				1,424
	(Mulifanua Port)				
4	Parking Area	m^2	1,700	320	544
5	Navigational Aids	LS	1	180,000	180
6	Mobilization	LS	1	1,780,000	1,780
7	E. Services	LS	(1-5)x0.05		107
8	Contingency	LS	(1-5)x0.15		322
	S. Total				430
	G. Total				4,358

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