INDEX

APPENDIX－A	1. Result of Natural Condition's Survey
	2. Present Conditions of the Existing Facilities
	3. Evaluation of Safety of the Existing Structure
	4. Environmental Impact
APPENDIX－1.	Member of Study Team
APPENDIX－2.	Survey Schedule
APPENDIX－3.	List of Party Concerned in the Recipient Country
APPENDIX－4.	Minutes of Discussions
APPENDIX－5-1.	(1) General Plot Plan of Quelimane Dry Dock
APPENDIX－5-1.	(2) Typical Sections of Dock Wall
APPENDIX－5-1.	(3) Section of Pump Room and Buttress
APPENDIX－5-1.	(4) Pile Arrangement
APPENDIX－5-1.	(5) Details of Steel Sheet Pile Wall
APPENDIX－5-2-1.	(1) Sounding Chart (Sept., 1992 survey)
APPENDIX－5-2-1.	(2) Sounding Chart (July., 1997 survey)
APPENDIX－5-2-1.	(3) Sounding Chart (Dec., 1997 survey)
APPENDIX－5-2-1.	(4) River Cross Section in Front of the Dock
APPENDIX－5-2-1.	(5) Difference of Riverbed between July 97 and Dec.97
APPENDIX－5-2-1.	(6) Location of Cross Sections
APPENDIX－5-2-1.	(7) River Cross Section
APPENDIX－5-2-2.	Riverbank Observation Around the Project Site
APPENDIX－5-2-3.	Observation of Tide Level
APPENDIX－5-2-4.	(1) Record of Flow Direction and Velocity Measurement
APPENDIX－5-2-4.	(2) Maximum Flow Velocity and Direction at Each Point
APPENDIX－5-2-4.	(3) Flow Direction / Velocity and Tide Level
APPENDIX－5-2-5.	(1) Monthly Precipitation in Quelimane
APPENDIX－5-2-5.	(2) Analysis of Daily Precipitation in Quelimane (1993 - 1997)
APPENDIX－5-2-6.	(1) Location of Soil Investigation
APPENDIX－5-2-6.	(2) Result of Soil Investigation (Dynamic Cone Penetration Test)
APPENDIX－5-2-6.	(3) Result of Soil Investigation (Dutch Cone Penetration Test)
APPENDIX－5-3.	Present Conditions of Superstructure of Revetment
APPENDIX－5-4.	Yearly Erosion Rate of River Bank
APPENDIX－6.	Structural Stability Analysis of Circular Slip
APPENDIX－7.	Assessment of Local Scouring Area Due To Dock Facility
APPENDIX－8.	Assessment of Cover Range of the Erosion Protection Wall
APPENDIX－9.	Principal Design Criteria for Erosion Protection Structures

Appendix A

1. Result of Natural Condition Survey

1.1 Riverbed

In order to gain an understanding of the riverbed shape and changes in maximum water depth, based on progressive erosion, as well as to help to clarify the erosion mechanism, riverbed sounding was carried out in waters in front of the Plan site, along with cross sections above the point of river confluence upstream from the Plan site and at various downstream locations. Riverbed contours were evaluated based on comparisons with the cross sections surveys carried out in September, 1992 and in July and December, 1997.

The kinematic GPS survey was carried out by using echo sounder synchronized with 3 GPS receivers.

The survey findings are shown in Appendix 5-2-1 (1) ~ (7).

Based on a comparison of riverbed as of September, 1992 and July, 1997, it was confirmed that the riverbed in front of the dock was notably eroded but, based on a further comparison of the sounding findings for July and December, 1997. no major erosion progress was discovered.

Maximum water depth, as recorded in the depth measurements taken in July, 1997, was found to be -14 m in sections about 140 m upstream from the dock but, in the December survey, the -14m isobath contour had expanded to about 80 m offshore directly in front of the dock.. While a tendency toward increased maximum water depth has not been found, the area involved may be expected to evidence continued expansion in the future. As shown in Appendix 5-2-1 (5), based on water depth readings for July and December, 1997, and an examination of the extent to which depths rose and fell, by section, during this period, a tendency was seen for the river bottom contour to shift from the upstream to the downstream side. It would, therefore, be reasonable, in our judgment, to consider the possibility that the future maximum water depth in front of the dock may reach -14 m.

1.2. Condition of Riverbank Line

In order to grasp the contour of the long riverbank, observations were made and photos taken from an aircraft over a wide expanse, both upriver and downriver, centering on the dock. These photos were then compared with previous aerial photos taken in June 1996.

The findings from these surveys are presented in Appendix 5-2-2.

Based on a comparison of the two sets of photographs (the Air survey photos from June, 1996 and those taken from an aircraft in December, 1997), it was confirmed that 15~20 m of erosion had taken place over this 1.5 year period in an area centering around the fisherfolk village downstream from the dock. This finding concurred with the results of our interview survey in that village.

1.3. Soil Conditions

Data clarifying soil conditions in the Plan area already exist, based on boring surveys conducted in September, 1992 at three shore locations in the subject Plan area, and in February, 1987 at two locations along the river basin at the Quelimane Fishing Port, located about 700 m upriver from the Plan site. For this reason, the December, 1997 survey consisted of Dutch Double Corn Penetration Test (CPT) and Dynamic Cone Penetration Test (DPSH) tests, designed primarily to confirm soil layer composition and soil characteristics. The Dutch probes, targeted at clayey soil above the foundations, were conducted at 4 shore locations, while the DPSH tests, directed at both relatively hard and clayey soil in the lower section of the foundation, were carried out at 5 locations in the river area and 2 shore locations The shore locations used for the DPSH tests were the same as those used for the Dutch probes.

The locations for the soil surveys conducted in the Plan area in September, 1992 and December, 1997 are given in Appendix 5-2-6(1), while penetration resistance, representing the results of the Dutch probe tests, and the number of blows, reflecting the findings of the DPSH tests, are shown in Appendices 5-2-6(2), (3) respectively.

The findings obtained on soil layers and soil characteristics were little changed from those developed through previous boring surveys. As in these past surveys, the N values for the sandy layers at the base of the viscous layers showed considerable variation. In Dutch probe testing, correlation are well-established between penetration resistance and both N values and uniaxial compressive strength; and, in DPSH testing, between the number of blows and N values.

The following formulae are widely accepted for converting cone penetration resistance (qc) to cohesion and the N value of sandy soil.

The formula for conversion to N values is as proposed by Meyerhof, but the "4" coefficient therein fluctuates over a wide range, from 2~10 or more, depending on soil characteristics.

- cohesion (Cu) $qc(kg/cm2) = (14 \text{ to } 17)\,Cu$
- N value $qc = 4\,N$

For purposes of estimating the internal frictional angle of sandy soil from N values, the Peck and Meyerhofs method are well established. These correlation are as shown in Table 1.1.

Table 1.1 Relationships between N Values, Internal Frictional Angle, and Relative Density of Sandy Soil, based on Peck and Meyerhof

N-Value	Relative Density (Dr)	Internal Friction Angle φ (°)	
		Peck	Meyerhof
0～4	very loose (0～0.2)	28.5 or below	30 or below
4～10	loose (0.2～0.4)	28.5～30	30～35
10～30	medium (0.4～0.6)	30～36	35～40
30～50	dense (0.6～0.8)	36～41	40～45
over 50	very dense (0.8～1)	41 or more	45 or more

(Note : According to Peck and Meyerhof's Study)

Based on the above estimating methods along with empirical judgments, the soil characteristics, as estimated from the findings of the Dutch probe and DPSH tests, are shown in Table 1.2.

Table 1.2 Soil Characteristics

Dutch Cone Penetration Test				
Depth (m)	+5.5～+3.5	+3.5～-5.5	-5.5～-9.0	-9.0～
Cone Penetration Resistance (Mpa)	1.9～2.6	0～1.6	4.2～14.6	15 <
Kind of Soil	Sand (Filled)	Clay	Sand	Sand
Converted N-Value	2 ～ 5	< 5	10 ～ 40	50 <
Internal Friction Angle (cohesion)	30 ～ 32	(10 ～ 40 kPa)	32 ～ 38	38 <
Dynamic Cone Penetration Test				
Depth (m)	±0.0～-5.5	-5.5～-8.0	-8.0～-11.0	-11.0～-14.0
Converted N-Value	1 ～ 10	10 ～ 40	15 ～ 40	40 <
Kind of Soil	Clay	Sand	Sand	Sand
Internal Friction Angle	—	32 ～ 38	32 ～ 38	38 <

Note: All Depths are based on Chart Datum Level (C.D.L.)

Based on the above findings as well as the existing boring data, the foundation conditions at the Plan site, reflecting its location in a river area, were found to be a so-called soft foundation, made up of a clayey layer from 0 ~ 8 m, with an N value of N = 1. The relative density of the second layer, from 8 ~ 17 m, ranged from medium to firm, but with a wide dispersion in N values from 10~50. The depth of the basic foundation layer, with an N value of 50 or more, was confirmed as 25~30 m or lower. A cemented sand horizon is presumed to exist as a micro characteristic of the Plan site, particularly in view of the wide N dispersion in the second layer.

1.4. Tidal Levels

In order to determine whether variations in tidal levels are the principal reason for the accelerated erosion, we conducted a series of tidal observations in front of the dock gate over a 27-hour period on December 12~13, 1997 (moon age: 12.5 ~ 13.5). These findings were then compared and evaluated vis-a-vis those from a 25-hour series of tidal observations conducted during spring tide on September 12 ~ 13, 1992 (moon age : 15.8 ~ 16.8).

The findings from the above surveys are shown in Appendix 5-2-3.

Comparing the measurements obtained in December, 1997 with the estimated tide level values at Morrubune, located on the Bons Sinais estuary, as shown in the tidal tables at Quelimane, the tide level in front of the dock was found to rise about 20 cm at high tide and fall some 29 cm at ebb tide.

High tide at the Plan area was shown to lag by about 20 minutes, and ebb tide by about 26 minutes. These findings were extremely close those obtained in the 1992 survey, which showed a tidal rise of over 20 cm and a high-tide lag of 20 minutes. These slight discrepancies can be attributed to differences in average surface levels based on seasonal differences in survey times. It may be concluded, therefore, that there are no notable changes in tidal levels between the two sets of data.

1.5. River Flow Conditions

Observations were made on flow direction and flow speed at the locations shown in Figure 1.5.-1 in order to gain an understanding of flow conditions along the entire river and examine their relationship with the erosion process, while also estimating maximum river flow speed for use in designing the erosion protection work in front of the dock.

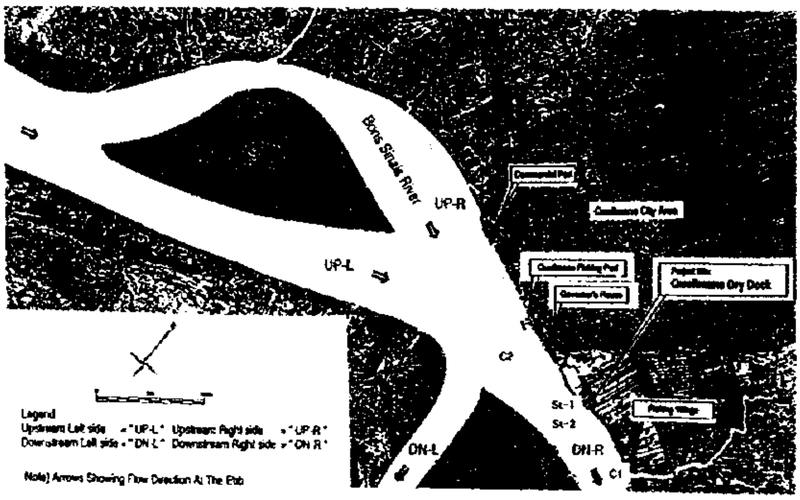


Figure 1.1 River Flow Observation Points

Observation records already exist from two previous surveys conducted at 2 locations in front of the dock (St. 1, St. 2) and 3 layers (surface, middle, and lower) over a period of 25 hours, as conducted on September 12~13, 1992 (moon age: 15.0~16.0) and July 17~18, 1997 (moon age: 12.2~13.2). In a subsequent survey on December 11~12, 1997 (moon age: 11.5~12.5), in addition to the two inspection points, a third location (St. 3) was established 50.0 m offshore in front of the dock, but, as the mooing buoy at this additional site sank during high tide, reliable measurement values could not be obtained. Comparing the results from the September, 1992 and July, 1997 surveys with the measurements obtained in December, 1997, in the case of both hourly and maximum flow speeds, the September, 1992 survey yielded the highest measurement values. This result may be due to the complex involvement of such factors as differences in moon age, changes in average surface levels reflecting seasonal differences, and changes in river flow volume. Maximum flow speeds (based on actual survey measurements) and flow direction observations are shown in Table 1.3

Table 1.3 Maximum Flow Velocity and Direction Among Actual Measured Data

Station	Layer	Observation Date	Flow Direction	Flow Velocity
St.2	-4m	92.9.13 97.7.18	90.0° 100.6°	114cm/sec. 92.4cm/sec.
St.2	-2m	97.12.11	95.6°	90.94cm/sec.

With respect to the September, 1992 observations in the above table for St. 2 at - 4 m, when the highest flow speed was recorded, the average eastward and westward flows and recurring tides at flood tide have been derived, as shown in Table 1.4.

Table 1.4 Maximum Flow Velocity on Observed on Sept. 13, 1992

Mean Max. Spring	St.2	- 4.00 Layer	271°	119.7cm/s	River Flow	273°	14.1cm/s	Total 133.8cm/s
Tropic Spring	St.2	- 4.00 Layer	271°	137.0cm/s	River Flow	273°	14.1cm/s	Total 151.1cm/s

From the above, we have concluded that it would be proper to use 150 cm /s as the maximum flow speed for designing the structure and other facilities in front of the dock, including increases in river flow speed resulting from heavy rainfall.

In addition, in order to grasp flow conditions in the sections upriver and downriver from the dock, we measured flow direction and speed at 3 layers (surface, middle, and lower) at navigation buoys in downstream area (C1) and upstream area (C2). We also measured flow direction and speed at 12 locations in 4 sections on the branch streams upriver and downriver from the dock (at 3 layers: surface, middle, and lower) during both fair tide (upriver → downriver) and back-tide (downriver → upriver) in order to measure respective flow volume in the 2 upstream and 2 downstream branches so as to gain an understanding of flow volume and balance in these branch streams during both fair and back-tide periods. These measurements were supplemented by visual flow observations on the river directly in front of the facility.

The survey findings on flow direction and speed, as measured at the above survey locations, along with their relationship to tidal levels at the time of measurement, are presented in Appendices 5-2-4 (1) ~ (3).

Cross-sectional stream area, maximum water depth, and flow volume, as derived from the observation data, are presented in Table 1.5.

Table 1.5. Cross Sectional Area & Discharge Volume at 4 Diversions

		Upstream Diversions		Downstream Diversions	
		UP-R	UP-L	DN-R	DN-L
Ebb	Cross Sectional Area of Flow (m2)	2,680 (1,540)	3,570 (3,020)	4,760 (4,660)	480 (460)
	Max. Depth (m)	5.34	7.71	10.64	7.69
	Average Flow Velocity (m/s) (Averaged 3 points)	0.33~0.60 (0.46)	0.58~0.69 (0.63)	0.51~0.92 (0.70)	0.52~1.31 (0.92)
	Discharge (m3/s)	710	1,900	3,260	420
Flood	Cross Sectional Area of Flow (m2)	2,980 (1,500)	3,320 (3,310)	4,570 (4,300)	420 (360)
	Max. Depth (m)	5.78	7.27	10.30	7.22
	Average Flow Velocity (m/s) (Averaged 3 points)	0.52~0.63 (0.58)	0.46~0.90 (0.62)	0.31~0.78 (0.50)	0.60~0.96 (0.81)
	Discharge (m3/s)	870	2,050	2,150	290

Comparing the 2 watercourses on the upriver side (UP-L and UP-R), UP -L (the watercourse on the south side) evidences a larger cross-sectional stream area, deeper water depth, and higher flow volume. This may be said to support, to some extent, the findings from our interview survey to the effect that "the main waterway on the upriver side used to be on the Commercial Port (northern) side, but, in recent years, the branch stream on the southern side has become the main course."

On the other hand, comparing the 2 watercourses on the downriver side (DN - L and DN - R), as is evident also from river width, DN-R (the watercourse on the dock side) was observed to be far dominant in the cross-sectional stream area, water depth, and flow pressure alike and so was verified as the main watercourse .

From the above, it has been confirmed that the main current in the area facing the dock is that flowing from UP-L (the watercourse on the south side of the upriver section) to DN-R (the course on the dock side (with the directions reversed during back-tide periods.); and that the flow direction in the main waterway directly strikes the shore side on which the dock has been constructed.

1.6. Meteorological

The Aerial Meteorological Observation Station at Quelimane Airport has compiled precipitation data over a 5-year period from 1993 through November, 1997. This data has been compared and analyzed vis-a-vis that for the period 1982 ~ 1991, as gathered during the 1992 survey.

Annual rainfall over the entire span (1982~1997) is given in Figure 1.2. (Cf. Appendices 5-2-5 (1), (2) at the back of this Report.)

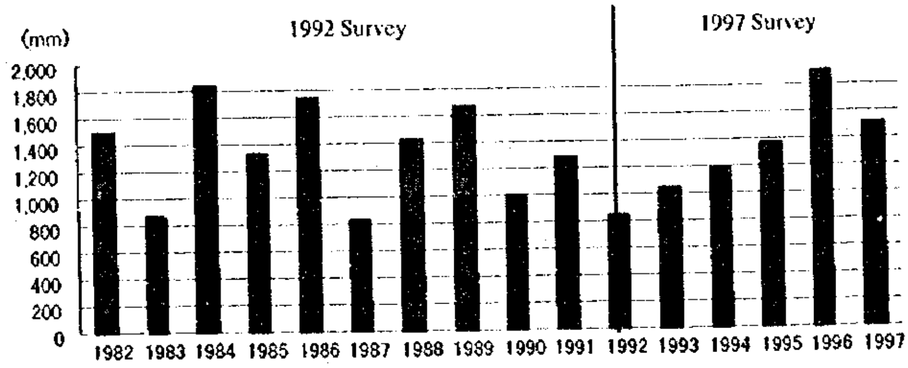


Figure 1.2 Yearly Precipitation

Annual precipitation from 1990~1995 did not exceed 1,300 mm. While it increased to 1,897 mm in 1996, this was due to the fact that average monthly rainfall during the rainy season (January through March) totaled 465.3 mm, 299 mm, and 308.6 mm, respectively some 1.5 times the average precipitation in previous years. Similarly, rainfall during February, 1997, at 588.1 mm, was double the historical average for this month.

But it is short-term daily rainfall that becomes a direct cause of flooding and other disasters. Recorded data show that, over the 5-year period from 1993~1997, daily precipitation exceeded 50 mm on 27 days, reaching record daily levels of 206 mm in January, 1996 and 145 mm in February, 1997.

2. Present Condition of the Existing Facilities

2.1. Joint Separation in the Upper

Increased joint separation in the upper concrete sections was first reported at the end of June, 1997. In an effort to measure the changes in the degree of separation from June until December, observation and measuring locations were set up at Point A on the upstream side and Point C on the downstream. These locations are shown in Figures 2.1 and 2.2.

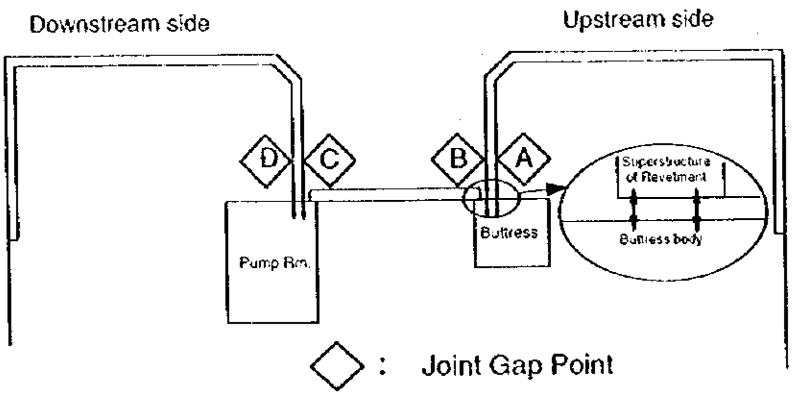


Figure 2.1 Measuring Locations

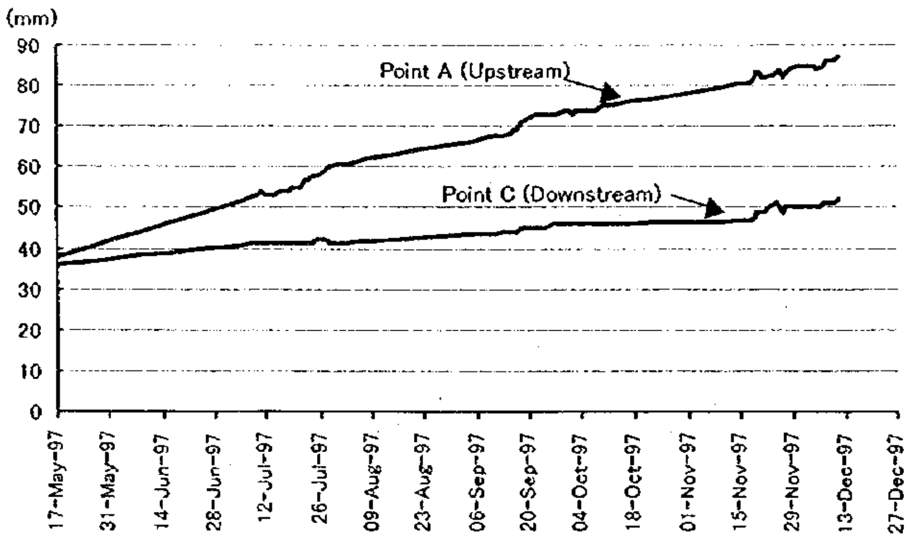


Figure 2.2 Development of Joint Separations

Measurements of joint separations in the buttress and upper concrete sections of the revetment were made at Points A and B on the upstream side, while those at the pump room and upper concrete sections were taken at Points C and D on the downstream side. The measurement values represent conversions of measurement data, based on the assumption that the concrete joint sections had been touching at the time the facilities were completed.

The amount of joint separation, as shown in Table 2.1, has been gradually and continuously increasing, and, from these movements, it may be concluded that the displacement of the wing sections on either side of the dock has definitely been progressing.

Table 2.1 Gap Measuring Record at the Both Wings

Measuring Point	1997/5/17	1997/7/25	1997/12/11
Point A (Upstream)	38 mm	58 mm	87 mm
Point C (Downstream)	36 mm	42 mm	52 mm

2.2. Inclination of the Steel Sheet Piles in the Front Revetment

In order to determine the degree of inclination (X) in the baseline direction and the extent of collapse (Y) at a right angle to the baseline direction, measurements were made from a boat during ebb tide at intervals of about 2.0m in a vertical direction, using a plumb bob. The measurement locations and findings are presented in Figure 2.3.

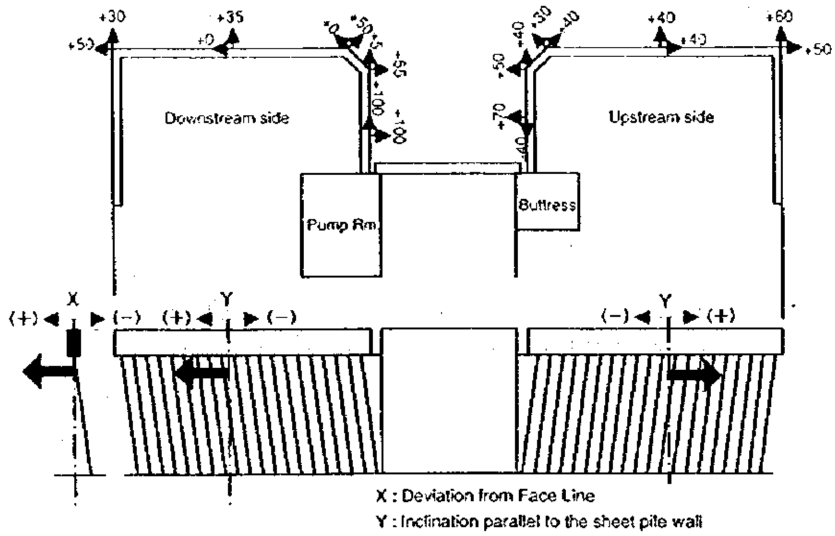


Figure 2.3 Measuring Record of Inclination of Steel Sheet Pile Wall

With respect to the baseline direction, as viewed from the offshore side in front of the dock, a inclination toward the outside from the dock center has been confirmed on both the upstream and downstream sides, with the upstream displacement larger than that on the downstream side. Generally speaking, the steel piles evidence a inclination in the pile-driving direction, owing to the frictional force generate at coupling between the sheet piles during pile driving operation. It is believed, therefore, that these differences resulted from the fact that the respective piles had been driven toward the outside from the center of the dock. However, since this was the first time that slope measurements were taken, changes and displacement in pile inclination since facility completion are not definitive.

While the baseline direction on the upstream side is, on average, collapsing toward the river, the degree of collapse cannot yet be measured visually. Moreover, displacement on the upriver side is larger than that on the downriver side.

2.3. Protective Sheet Piles for the Front Revetment.

The present condition of the protection sheet piles, which were driven to protect the front revetment, is not entirely clear since they are buried under river water of high turbidity. However, a portion of the piles in the external corner sections are gone or leaning heavily to the outside, and it has become clear that, even when the front depth measurements were taken, no reaction was clearly recorded on the sounder. In addition, the top of the protective piles in the middle section could not dry out even at the lowest ebb during the spring tide, raising the possibility that this portion too may be leaning toward the outside. And, in the case of the steel sheet piles in the downstream section, the corner portions are leaning outward toward the river side, with pile dispersion also increasing.

2.4. Concrete Slab in Front of the Dock Gate

The scouring in the middle of the dock is progressing, having now reached as far as the area below the concrete slab in front of the dock gate. As a consequence, the slab has been damaged from a point about 3 m in front of the gate, while the central portion is sinking. Directly in front of the gate, the edge section of the inclining slab has risen, so that, when the gate is opened, it touches the edge section of the slab and thus can only be opened to about 160 degree. The failure of the gate to fully open is a phenomenon that developed in late July, 1997 and so has been confirmed as one evidencing further advancement of sub-slab scouring.

2.5. Displacement of the Coping Concrete in the Upper Portion of the Front Revetment

Observations was carried out on the locations and widths of the cracks in the coping concrete sections of the upper revetment, and these measurements were then compared with those taken in July, 1997. Moreover, a line was spread to the front baseline of the revetment as well as that of the concrete in the upper part of both wing sections so as to measure relative baseline displacement. The measurement locations and finding are shown in Figure 2.4.

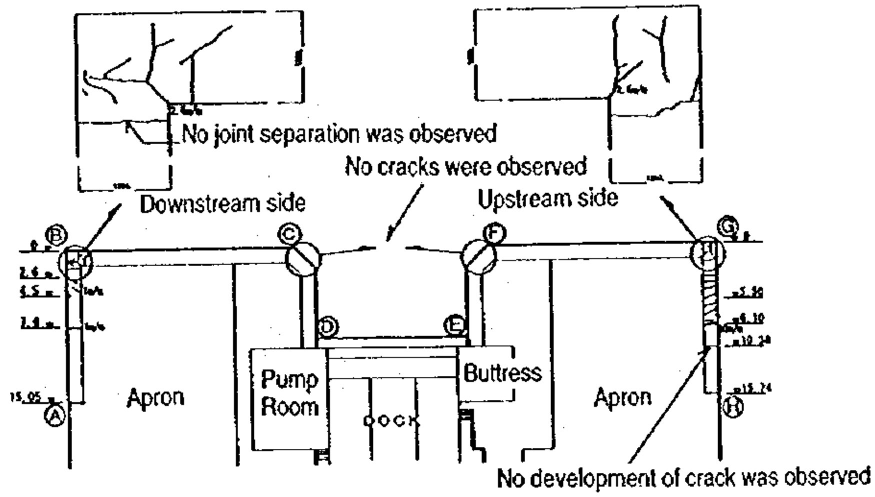


Figure 2.4 Crack Locations in the Coping Concrete Sections in the Upriver Revetment.

While the comparison with the findings of the July, 1997 survey revealed no increase in the number of cracks, since the concrete in the upper section of the revetment on the upstream side showed an enlarged displacement toward the outside in almost the center of the G-H section, the crack width in the center had advanced 5 mm → 30 mm.

It has been determined that, as this section is a self supporting revetment, in view of the progressive scouring action in the front, the pressure toward the outside on the side wall becomes relatively large, causing the cracks in the central portion to develop. Because of this, as the G portion is a corner section and thus subject to considerable bending distortion, the cracks have progressed to a substantial degree. The cracks in this portion and in the center of the G - H portion have penetrated the coping concrete.

No cracks were observed in the C or F portions, as was also the case at the time of the July, 1997 survey. Nor was there any conspicuous cracking between Points A -B on the downstream side. With respect to the revetment baseline, a line was spread to Points B-G to measure relative displacement. A 14 mm difference was observed at Point C, and 120 mm at Point F.

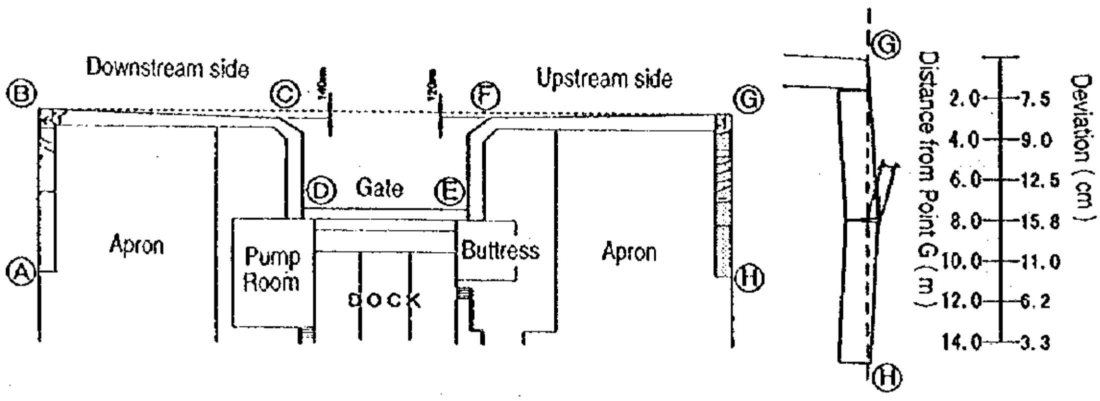


Figure 2.5 Displacement of the Coping Concrete

2.6 Changes in the Anchoring Sheet Piles and Tie-rod for the Front Revetment

Conditions over the full length of the anchoring sheet piles and tie-rod in the revetment on the upstream side were confirmed by excavating the fill soil. Measurement data are given in Figure 2.6.

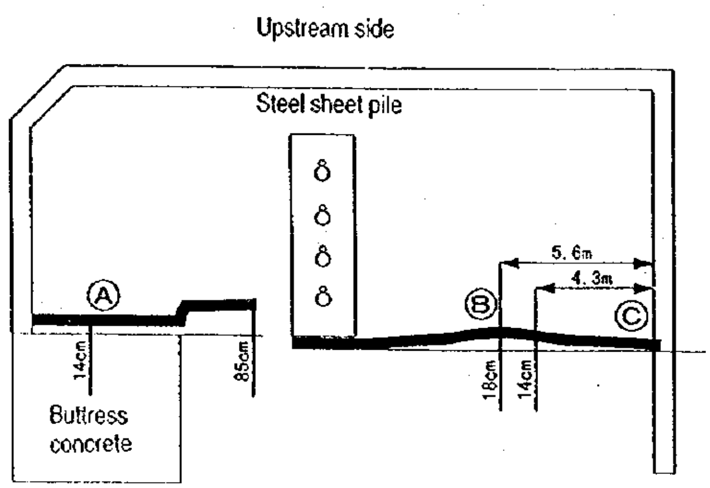


Figure 2.6 Shape Changes in the Anchoring Work

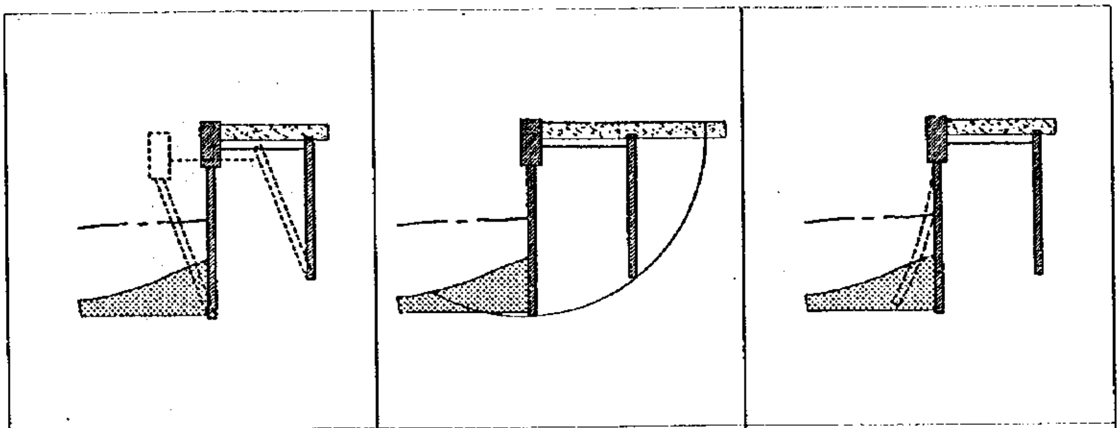
Vertical load appears to have been applied to the tie-rod along with the settlement of the fill material in the upper section but, in the anchoring sheet piles, considerable tensile load has been operating upon the river side, with shape changes at the top of the anchoring sheet piles reaching 140 mm in the buttress section and 180 mm and 140 mm, respectively, in the B and C portions. Based on these shape changes, the tie-rod has bent downward in the central portion, though no abnormalities were found in the pile installation section, ring joints, or turnbuckles.

3. Evaluation of the Safety of the Existing Structure

Based on findings from our survey concerning the existing facilities, the safety of the existing structure was made a careful examination.

Since the front foundation has been scoured, the front revetment is in unstable condition. The destructive mechanism is believed to be as follows :

① bending capitulation of the steel sheet piles and breakage of the tie-rods ;
② circular slip of the entire revetment; and
③ flipping out up of the base section in the lower part of the steel sheet piles.



Case ①	Case ②	Case ③
Bending capitulation of the steel sheet piles and breakage of the tie-rods	Circular slip of the entire revetment	Flipping out up of the base section in the lower part of the steel sheet piles

Fig. 3-1 Presumed Mechanism of Collapse

In case of the subject facilities, as it is feared that the case ② and ③ among above three types of collapse will especially be expected to occur, each case studies were examined.

3.1 Circular slip of the entire revetment

For the verification of the safety against the circular slip, an examination was made for the 4 sections of front revetment. As a result of the examination, it became clear that the safety factor against the section of the lowest downriver is F = 1.0, if the lowest internal friction angle is applied, so that foundation stability is barely being maintained, there is a possibility of causing land collapse owing to the progress of scouring. Careful attention should be given to the fact that a collapse from circular slip occurs much more suddenly than in the case of other collapse modes and that such a collapse would encompass the entire revetment.

3.2. Flipping out of the base section of the steel sheet

Since, as a result of scouring, the ground level of the front revetment has considerably lowered compared with what it was. The embedment length of the sheet piles to the foundation has shortened, the passive resistance of the front foundation has declined substantially. Furthermore, as discussed above, as the span has lengthened (owing to an increase in water depth from the original Design). the soil pressure applied to the sheet piles has increased. demanding considerable passive resistance in the base portion.

From an design standpoint, there is a clear danger of the base section of the steel sheet plates at any time.

From a design standpoint, the revetment structure has not maintained the necessary degree of safety. Accordingly, it is vital that the foundation level in front of the revetment of the dock be restored without delay.

4. Environmental Impact

4.1 Noise, Vibration, and Offensive Odors

The matter of noise, vibrations, and offensive odors arising during dock operations was carefully considered at the time the Basic Design for the subject facility prepared in 1992 . In fact, the present site was chosen in the first place because there was no concern over noise or security. Even after the dock initiated operations, no activities were ever undertaken that might give rise to significant noise, vibration, foul odors, or the like, nor did these operations have any adverse impact on local residents.

4.2 Drainage

Drainage from the subject facility comprises dirty water and miscellaneous drainage from the Administration Building as well as drainage resulting from hull washing operations inside the dock using water accumulated in the rainwater tank. The dirty water and miscellaneous drainage are processed via the soak-away pit, but, as a result of the clayey soil characteristics, permeation speed is slow, and so, in the event that drainage loads start to exceed present levels, countermeasures would be required, such as installation of a new soak-away.

The washing water is used chiefly to wash down hulls, so that the drainage does not include any large quantities of oil, paint, or similar substances.

4.3 Changes in River Flow Conditions Due to the Structure and Riverbank Erosion

Erosion has occurred over a wide area of the riverbank on the dock side, from the point at which the embankment wall ends at the eastern edge of the downtown area 500 m upstream from the dock to the edge of the fisherfolk village, about 1.1 km downstream. Local scouring and erosion that have been generated by a turbulent current, traceable to construction of the dock structure, have been noticed at both wing sections and in the center of the dock. While the erosion protection work under the subject Plan is intended to control scouring, it will be difficult to completely avoid the irregular currents over a wide area that are caused by the structure. While the possibility cannot be denied that, should riverbank erosion advance further in the future, there could be a negative impact on the natural environment, it should be recognized that such phenomena arise even at present at numerous locations extending to the estuary area, which require countermeasures separate and distinct from the prevention of localized turbulent currents induced by the Plan facility.

4.4 Impact on Cruising Vessels

Based on the erosion protection work under this Plan, a structure is to be built to protect the front of the dock embankment from erosion. In the vicinity of the Plan area, a tendency has been noted for the center of the flow axis to approach the bank on the dock side. Accordingly, as the cruising course too may well come close to the dock, it will be necessary, in the interest of vessel safety, to install light buoys on both the upstream and downstream sides of the structure .

Basic Design Study on
The Project for Maintenance of the Quelimane Dry Dock
in the Republic of Mozambique
Member of the Study Team

1. Leader	Mr. Kuniichi ASAOKA Deputy Director of Construction Div. Fishing Port Dept., Fisheries Agency, Ministry of Agriculture, Forest and Fisheries
2. Coordinator	Mr. Fumio TERASHIMA Second Project Study Division, Grant Aid Project Study Development, Japan International Cooperation Agency
3. Technical Adviser	Mr. Shin-ichi NAKAMURA Office of Overseas Fisheries Cooperation, International Affairs Division, Fisheries Policy Planning Dept., Fisheries Agency, Ministry of Agriculture, Forest and Fisheries
4. Chief Consultant cum Civil Engineer	Mr. Kunihiro WATANABE Fisheries Engineering Co. Ltd.
5. Physical Condition Researcher	Mr. Yoshiharu MATSUMOTO Fisheries Engineering Co. Ltd.
6. River System Analyst	Mr. Nobuo SATO Fisheries Engineering Co. Ltd.
7. Engineering Work Planner	Mr. Toshihito INKI, Fisheries Engineering Co. Ltd.
8. Interpreter	Ms. Keiko MITSUNAGA Fisheries Engineering Co. Ltd.
9. Executive Planner	Mr. Naohiko NAKAJIMA Fisheries Engineering Co. Ltd.

APPENDIX－2
Survey Schedule

	Date	Description		
		Gov. (Mr.Asaoka, Mr.Nakamura, Mr.Terashima)	Consultant	
1	06-Dec-97	Narita 16:55 to Singapore 21:55 (NH901)	Tokyo16:55 to Hong Kong20:55 (JL735)	
			Hong Kong2300 to Johannesburg(SA287)	
2	07-Dec-97	Singapore0110 to Johannesburg0715(SQ406)	Hong Kong to Johannesburg 0545(SA287)	
		Johannesburg15:45 to Maputo16:40 (SA142)	Johannesburg15:45 to Maputo16:40 (SA142)	
3	08-Dec-97	Ministry of Agriculture & Fisheries	same as left	
4	09-Dec-97	Ministry of Agriculture & Fisheries	same as left	
		Ministry of Foreign Affairs and Cooperation	same as left	
		Maputo Fishing Port	Member Meeting, Collection of Maps & Aerial Photos	
		Ministry of Agriculture & Fisheries	same as left	
5	10-Dec-97	Maputo11:00 to Quelimane13:40(TM146)	same as left	
		Quelimane Dry Dock	same as left	
			Precision Sounding (in front of the Dock : Area 300x100m)	
			Member Meeting (re: Natural Condition Field Survey, etc.)	
6	11-Dec-97	River Bank Inspection from Boat	same as left(Watanabe,Sato,Mitsunaga,Nakajima)	Precision Sounding (Shore Area 300x50m)
		Quelimane Port Authority	same as left(Mitsunaga,Nakajima)	River Cross Section Survey (2 sects.)
		Quelimane Dry Dock	same as left(Watanabe,Sato,Mitsunaga,Nakajima)	Current Measuring (25 hrs., in front of the Dock)
				Tide Measuring (25 hrs., in front of the Dock)
7	12-Dec-97	Quelimane Dry Dock	same as left(Watanabe,Sato,Mitsunaga,Nakajima)	Tide Measuring (25 hrs., in front of the Dock)
		Hearing at Fishing Village downstream of the Dock	same as left(Watanabe,Sato,Mitsunaga,Nakajima)	River Cross Section Survey (8 sects.)
		Quelimane City Council (Terashima,Nakamura)	same as left(Mitsunaga,Nakajima)	Current Measuring(D1,D2,B2,B3)
		River Reconnaissance by Airplane(Asaoka)	same as left(Watanabe,Sato)	Collection of Weather Condition Data
		Hearing at Fishery Company	same as left(Mitsunaga,Nakajima)	River Cross Section Survey (in front of the Dock)
		Quelimane Dry Dock･･@	same as left(Watanabe,Sato,Mitsunaga,Nakajima)	Inspection of Dock Structures
8	13-Dec-97	Quelimane Dry Dock･･@	same as left(Mitsunaga,Nakajima)	Current Measuring(D1,D2,B2,B3)
		Inspection of the Dock Operation	same as left(Mitsunaga,Nakajima)	Current Measuring(C1,C2)
		Hearing at Fishery Company	same as left(Mitsunaga,Nakajima)	Inspection of Dock Structures
		Inspection of Filling Material Source	same as left(Watanabe)	Survey for Construction Materials
		Quelimane Dry Dock		
9	14-Dec-97	Inspection of Dock Structures	same as left	same as left
		Quelimane16:30 to Maputo18:00(TM145)	same as left	same as left
10	15-Dec-97	Member Meeting	same as left	Collection of Maps & Aerial Photos
		Meeting at Ministry of Agriculture & Fisheries	same as left	Survey for Construction Materials
			same as left	Data Analysis, Member Meeting
11	16-Dec-97	Meeting at Ministry of Agriculture & Fisheries	same as left	Collection of Maps & Aerial Photos
		Hearing at Fishery Company	same as left(Nakajima)	Survey for Construction Materials
				Data Analysis, Member Meeting
12	17-Dec-97	Courtesy Call (Vice Minister of Agriculture & Fisheries)	same as left	Survey for Construction Materials
		Signing of Minutes of Meeting	same as left	(Matsumoto,Sato,Mitsunaga)
			Survey for Construction Materials	Maputo21:30 to Johannesburg23:25 (TM307)
13	18-Dec-97	Maputo7:30 to Harare10:25 (TM342)	same as left(Watanabe,Nakajima,Inki)	Johannesburg13:45 to (CX748) to
		Visit Embassy of Japan, JICA (Harare)	same as left	
14	19-Dec-97	Harare10:15 to Johannesburg1215 (UM769)	Harare10:15 to Johannesburg1215 (UM769)	to Hong Kong08:40 (CX748)
		Johannesburg1415 to Singapore(SQ405) to	Johannesburg1435 to Hong Kong(SA286) to	Hong Kong1025 to Tokyo1505(JL736)
15	20-Dec-97	to Singapore0620(SQ405)	to Hong Kong0925(SA286)	
		Singapore0825 to Tokyo1545(JL712)	Hong Kong1025 to Tokyo1505(JL736)	

APPENDIX—3

List of Party Concerned in the Recipient Country

Name	Title	Organization
Isidora Fazutudo	Vice Minister	Ministry of Agriculture and Fisheries(MAF)
Rodrigues Bila	Secretary General	MAF
Herminio Tembe	National Director	National Directorate of Fisheries, MAF
Gustavo Miranda	Assistant to Director	Specialist, FAO
Paulino Cumbane	Chief of Secretariat	National Directorate of Fisheries, MAF
Maria Ismail	Chief	Dept. of Int'l Cooperation, MAF
Alberto Andissene	Director	Civil Engineering Laboratory of Mozambique Ministry of Public Works and Housing
Antonio Alver	Engineer	Civil Engineering Laboratory of Mozambique Ministry of Public Works and Housing
Zacarias Kupela	Director	Asia & Oceania Division, Ministry of Foreign Affairs and Cooperation
Chico Mortar	Desk Officer for Japan	Asia & Oceania Division, Ministry of Foreign Affairs and Cooperation
Joaquim Tembe	Director	Quelimane Dry Dock
Jose Ali	Chief Accountant	Quelimane Dry Dock
Jaime Gerente	Mayor	City of Quelimane

MINUTES OF DISCUSSIONS

BASIC DESIGN STUDY ON THE PROJECT FOR MAINTENANCE OF THE QUELIMANE DRY DOCK IN THE REPUBLIC OF MOZAMBIQUE

In response to the request from the Government of the Republic of Mozambique, the Government of Japan decided to conduct a basic design study on the project for maintenance of the Quelimane Dry Dock in Mozambique and entrusted the study to the Japan International Cooperation Agency (JICA).

JICA sent to Mozambique a basic design study team (hereinafter referred to as "the Study Team"), which is headed by Mr. ASAOKA Kuniichi, Deputy Director, Construction Division, Fishing Port Department, Fisheries Agency, Ministry of Agriculture, Forestry and Fisheries, and scheduled to stay in Mozambique from December 7 to 18, 1997.

The Study Team held a series of discussions with the officials concerned of the Government of Mozambique and conducted field surveys at the study areas.

Through the discussions and field surveys, both parties have confirmed several important points described in the attached sheets. The Study Team will proceed to further works in Japan and prepare the Basic Design Study Report.

Maputo, December 17, 1997

浅岡 邦一	(signature)
Mr. ASAOKA Kuniichi Leader, Basic Design Study Team, JICA	Mr. Rodrigues Armando Bila Secretary General Ministry of Agriculture and Fisheries Mozambique

ATTACHMENT

1. Objective

The objective of the project is to maintain the Quelimane Dry Dock which is necessary for the fishing activities in Mozambique.

2. Responsible Organization and Implementing Agency

Responsible Ministry : Ministry of Agriculture and Fisheries

Implementing Agency : Ministry of Agriculture and Fisheries

3. Project Site

The project site is shown in ANNEX-1.

4. Major Items Requested by the Government of Mozambique

After the series of discussions, the items listed in ANNEX-2 are finally requested by the Government of Mozambique.

However, the final items and specifications covered under the project will be subject to further studies.

5. Management and Maintenance

Ministry of Agriculture and Fisheries will maintain and use the Dry Dock maintained under the Grant Aid properly and effectively, and to assign the necessary staff members for operation and maintenance of them as well as to bear all the expenses other than those to be borne by the Grant Aid.

6. Japan's Grant Aid System

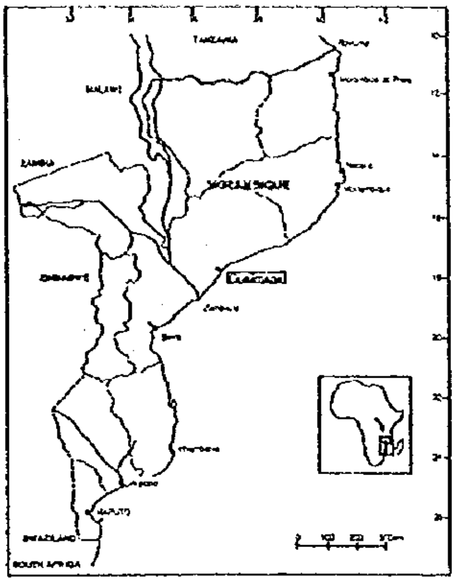
1) The Government of Mozambique has understood the system of the Japan's Grant Aid explained by the Study Team; the main feature is described in ANNEX-3.

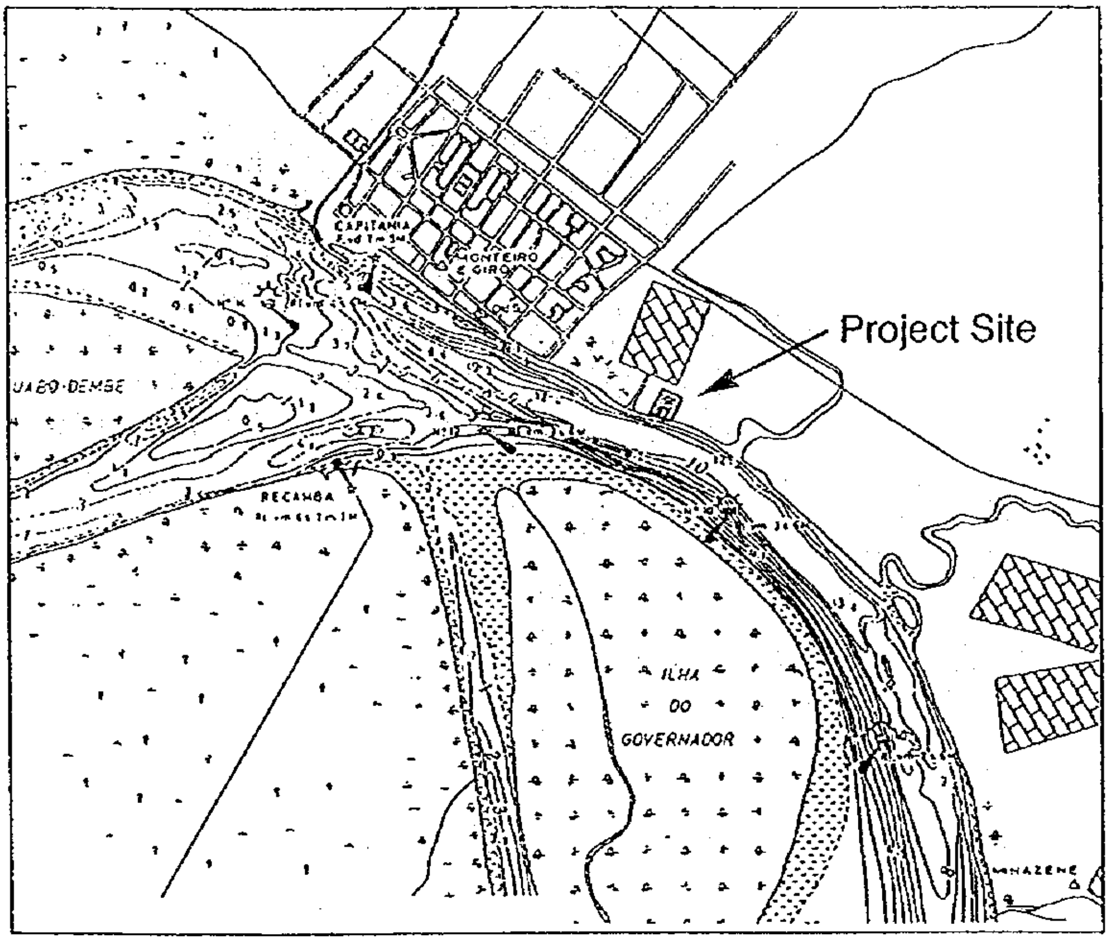
2) The Government of Mozambique will take the necessary measures, described in ANNEX-4 for the smooth implementation of the project on condition that the Grant Aid by the Government of Japan is extended to the project.

8. Further Schedule of the Study

Based on the results of the Basic Design Study, JICA will complete the Basic Design Study Report in accordance with the confirmed items, and forward it in its final form to the Government of Mozambique around March, 1998.

ANNEX-1: PROJECT SITE





ANNEX-2: ITEMS FINALLY REQUESTED BY THE GOVERNMENT OF MOZAMBIQUE

1. Formation of a mound at a height of -2.0m in front of the existing revetment.
2. Construction of an erosion protection wall with steel pipe piles, covering the front area and the part of both sides of the Dry Dock.
3. Renovation of the concrete slab in front of the dock gate.
4. Repair of other structural displacement of the Dry Dock, caused directly by erosion.
5. Necessary equipment for carrying out monitoring work.

ANNEX-3: JAPAN'S GRANT AID SCHEME

1. Grant Aid Procedure

1) Japan's Grant Aid Program is executed through the following procedures.

Application	(Request made by a recipient country)
Study	(Basic Design Study conducted by JICA)
Appraisal & Approval	(Appraisal by the Government of Japan & Approval by Cabinet)
Determination of Implementation	(The Notes exchanged between the Governments of Japan and the recipient country)

2) Firstly, the application or request for a Grant Aid project submitted by a recipient country is examined by the Government of Japan (the Ministry of Foreign Affairs) to determine whether or not it is eligible for Grant Aid. If the request is deemed appropriate, the Government of Japan assigns JICA to conduct a study on the request.

Secondly, JICA conducts the study (Basic Design Study), using Japanese consulting firms.

Thirdly, the Government of Japan appraises the project to see whether or not it is suitable for Japan's Grant Aid Program, based on the Basic Design Study report prepared by JICA and the results are then submitted to the Cabinet for approval.

Fourth, the project, once approved by the Cabinet, becomes official with the Exchange of Notes signed by the Government of Japan and the recipient country.

Finally, for the implementation of the project, JICA assists the recipient country in such matters as preparing tenders, contracts and so on.

2. Basic Design Study

1) Contents of the Study

The aim of the Basic Design Study (hereinafter referred to as "the Study"), conducted by JICA on the requested project (hereinafter referred to as "the Project"), is to provide a basic document necessary for the appraisal of the Project by the Government of Japan. The contents of the Study are as follows:

a) confirmation of the background, objectives and benefits of the Project and also institutional capacity of agencies concerned of the recipient country necessary for the Project's implementation;
b) evaluation of the appropriateness of the Project to be implemented under the Grant Aid Scheme from the technical, social and economic points of view;
c) confirmation of items agreed on by both parties concerning the basic concept

KA

BK

of the Project;

d) preparation of a basic design of the Project; and

e) estimation of costs of the Project.

The contents of the original request are not necessarily approved in their initial form as the contents of the Grant Aid project. The Basic Design of the Project is confirmed considering the guidelines of Japan's Grant Aid Scheme.

The Government of Japan requests the Government of the recipient country to take whatever measures are necessary to ensure its self-reliance in the implementation of the Project. Such measures must be guaranteed even though they may fall outside of the jurisdiction of the organization in the recipient country actually implementing the Project. Therefore, the implementation of the Project is confirmed by all relevant organizations of the recipient country through the Minutes of Discussions.

2) Selection of Consultants

For the smooth implementation of the Study, JICA uses a consulting firm selected through its own procedure (competitive proposal). The selected firm participates the Study and prepares a report based upon the terms of reference set by JICA.

At the beginning of implementation after the Exchange of Notes, JICA recommends the same consulting firm which participated in the Study be used for the services of the Detailed Design and Construction Supervision of the Project. This is necessary in order to maintain the technical consistency between the Basic Design and Detailed Design as well as to avoid any undue delay caused by the selection of a new consulting firm.

3. Japan's Grant Aid Scheme

1) What is Grant Aid?

The Grant Aid Program provides a recipient country with non-reimbursable funds to procure the facilities, equipment and services (engineering services and transportation of the products, etc.) for economic and social development of the country under principles in accordance with the relevant laws and regulations of Japan. Grant Aid is not supplied through the donation of materials as such.

2) Exchange of Notes (E/N)

Japan's Grant Aid is extended in accordance with the Notes exchanged by the two Governments concerned, in which the objectives of the project, period of execution, conditions and amount of the Grant Aid, etc., are confirmed.

3) "The period of the Grant" means the one fiscal year which the Cabinet approves the project for. Within the fiscal year, all procedure such as exchanging of the Notes,

KA B.K.

concluding contracts with consulting firms and contractors and final payment to them must be completed.

However, in case of delays in delivery, installation or construction due to unforeseen factors such as weather, the period of the Grant Aid can be further extended for a maximum of one fiscal year at most by mutual agreement between the two Governments.

4) Under the Grant, in principle, Japanese products and services including transport or those of the recipient country are to be purchased.

   When the two Governments deem it necessary, the Grant Aid may be used for the purchase of the products or services of a third country.

   However, the prime contractors, namely consulting, contracting and procurement firms, are limited to "Japanese nationals". (The term "Japanese nationals" means persons of Japanese nationality or Japanese corporations controlled by persons of Japanese nationality.)

5) Necessity of "Verification"

   The Government of the recipient country or its designated authority will conclude contracts denominated in Japanese yen with Japanese nationals. Those contracts shall be verified by the Government of Japan. This "Verification" is deemed necessary to secure accountability of Japanese taxpayers.

6) Undertakings required to the Government of the recipient country

   a) to secure a lot of land necessary for the construction of the Project and to clear the site;
   b) to provide facilities for distribution of electricity, water supply, drainage and other incidental facilities outside the site;
   c) to ensure prompt unloading, tax exemption and customs clearance at ports of disembarkation in the recipient country and internal transportation therein of the products purchased under the Grant Aid.
   d) to exempt Japanese nationals from customs duties, internal direct taxes and other fiscal levies which may be imposed in the recipient country with respect to the supply of the products and services under the verified contracts.
   e) to accord Japanese nationals whose services may be required in connection with the supply of the products and services under the verified contracts such facilities as may be necessary for their entry into the recipient country and stay therein for the performance of their work.
   f) to ensure that the facilities constructed and products purchased under the Grant Aid be maintained and used properly and effectively for the Project; and
   g) to bear all the expenses other than those covered by the Grant Aid, necessary for the Project.

7) "Proper Use"

The recipient country is required to maintain and use the facilities constructed and equipment purchased under the Grant Aid properly and effectively and to assign the necessary staff for operation and maintenance of them as well as to bear all the expenses other than those covered by the Grant Aid.

8) "Re-export"

The products purchased under the Grant Aid shall not re-exported from the recipient country.

9) Banking Arrangement (B/A)

a) The Government of the recipient country or its designated authority should open an account in the name of the Government of the recipient country in an authorized foreign exchange bank in Japan (hereinafter referred to as "the Bank"). The Government of Japan will execute the Grant Aid by making payments in Japanese yen to cover the obligations incurred by the Government of the recipient country or its designated authority under the verified contracts.

b) The payments will be made when payment requests are presented by the Bank to the Government of Japan under an authorization to pay issued by the Government of recipient country or its designated authority.

ANNEX-4: NECESSARY MEASURES TO BE TAKEN BY THE GOVERNMENT OF MOZAMBIQUE

The following necessary measures should be taken by the Government of Mozambique on condition that the Grant Aid by the Government of Japan is extended to the Project.

1. To ensure prompt unloading, tax exemption and customs clearance at ports of disembarkation in Mozambique and internal transportation therein of the products purchased under the Grant Aid.

2. To exempt Japanese nationals from customs duties, internal direct taxes and other fiscal levies which may be imposed in Mozambique with respect to the supply of the products and services under the verified contracts.

3. To accord Japanese nationals whose services may be required in connection with the supply of the products and services under the verified contracts such facilities as may be necessary for their entry into Mozambique and stay therein for the performance of their work.

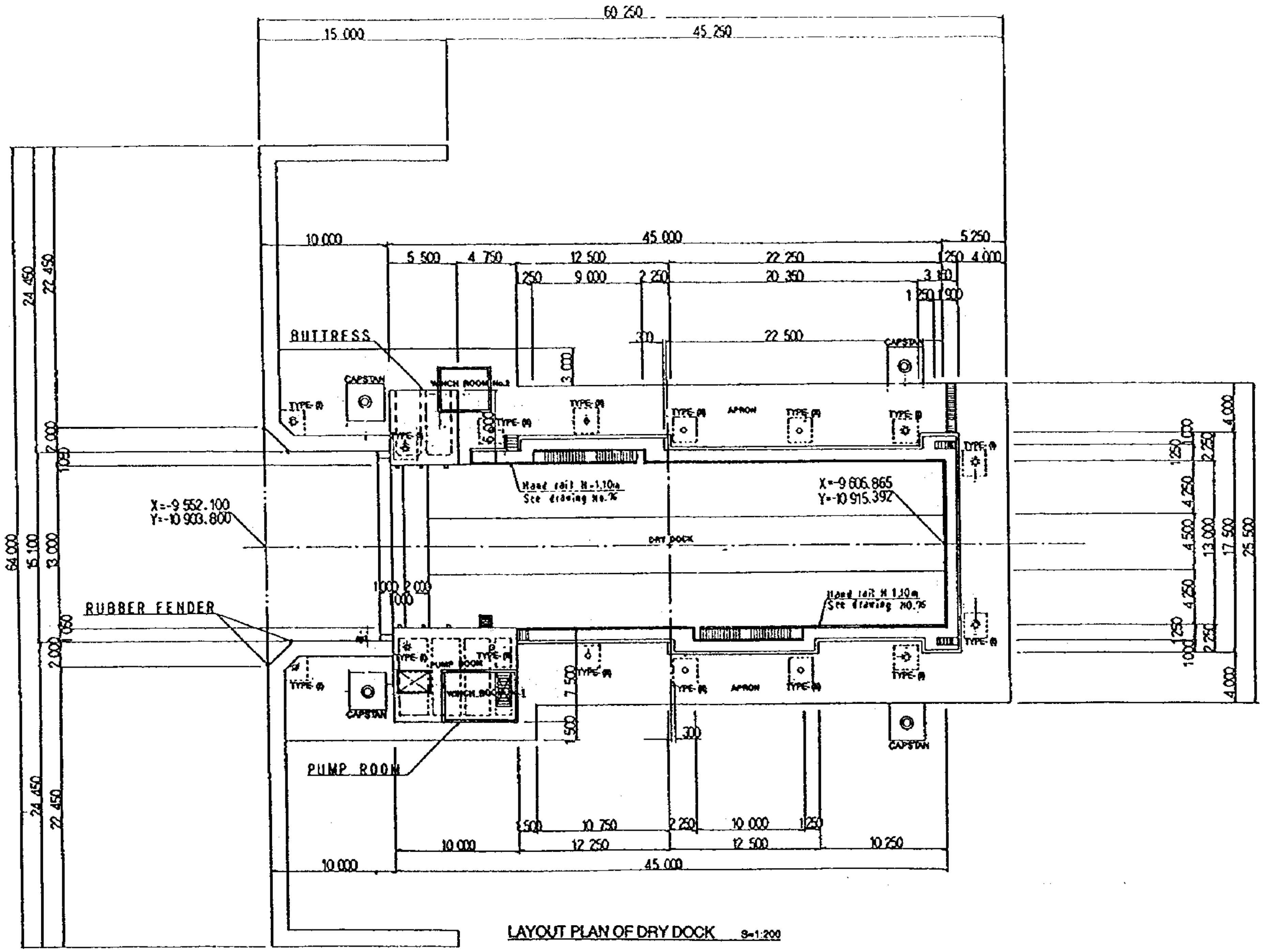
4. To maintain and use facilities constructed under the Grant Aid properly and effectively for the Project.

5. To bear commissions to the Japanese foreign exchange bank for its banking services based upon the Banking Arrangement, namely the advising commission of the "Authorization to Pay" and payment commissions.

6. To bear all the expenses, other than those covered by the Grant Aid, necessary for the Project.

APPENDIX -- 5-1 (1)

General Plot Plan of Quelimane Dry Dock

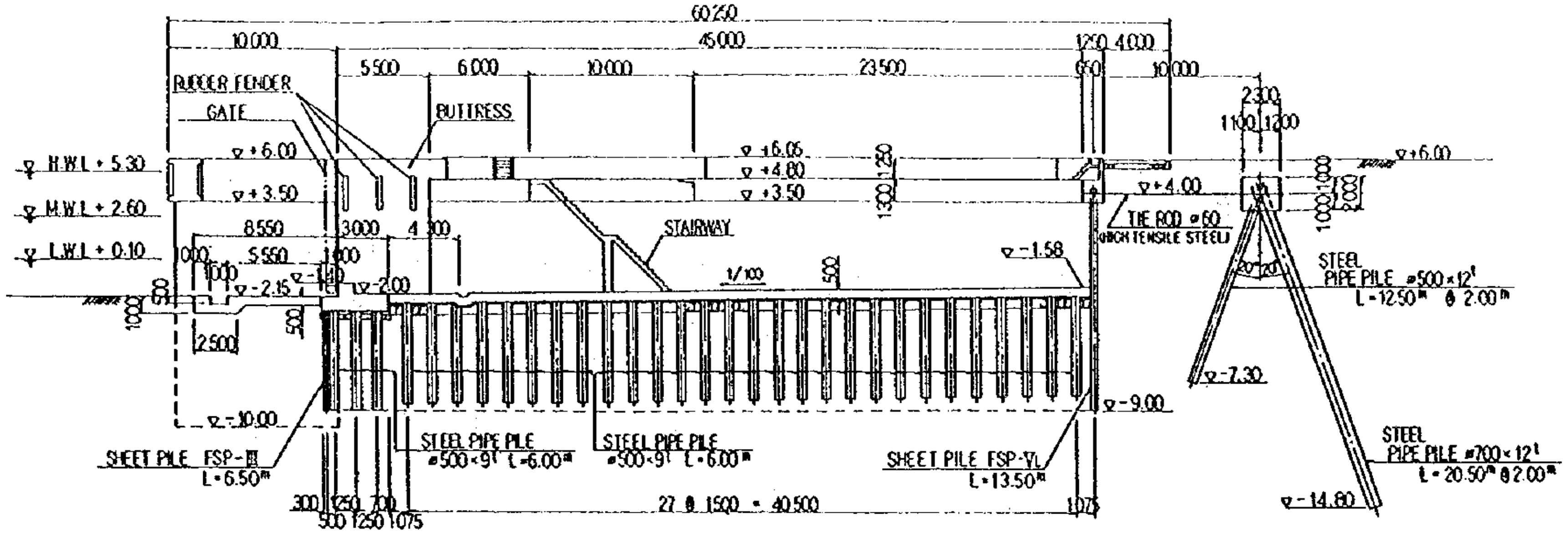


LAYOUT PLAN OF DRY DOCK S=1:200

NOTES: SYMBOLS OF MOORING INSTALLATIONS

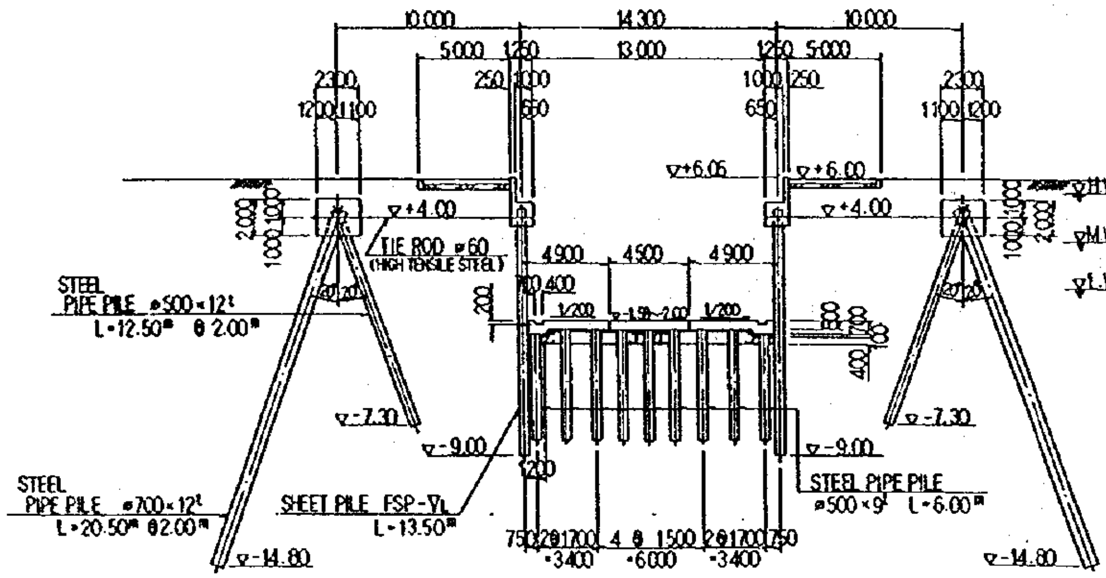
- CAPSTAN
- BIT/TYPE-(I)
- BIT/TYPE-(II)

A — A

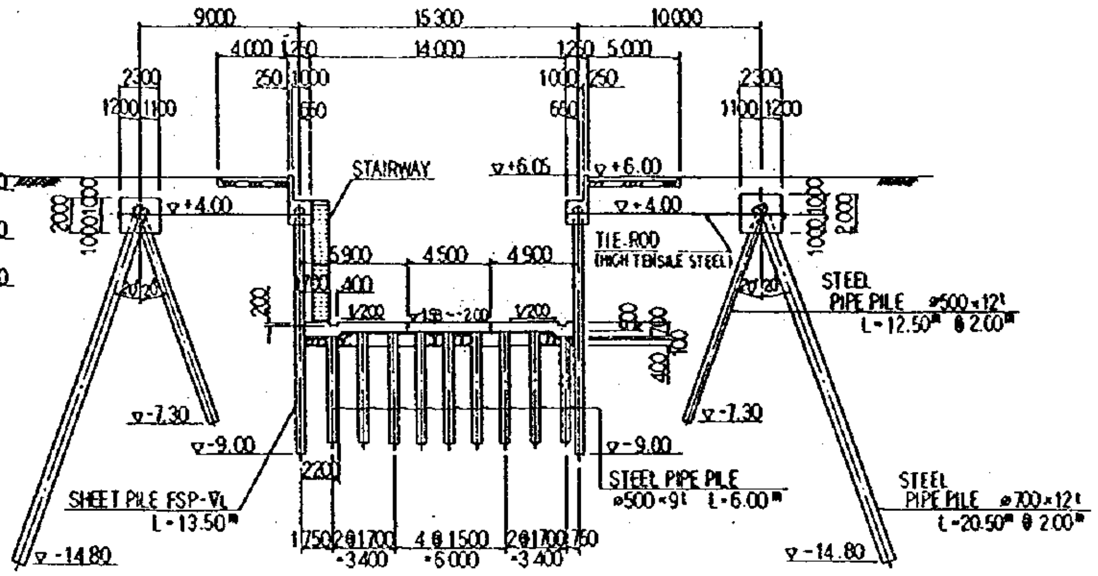


LONGITUDINAL SECTION OF DRY DOCK S=1:200

B — B

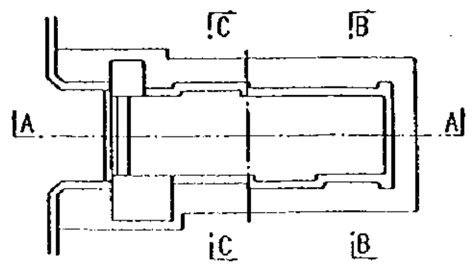


C — C

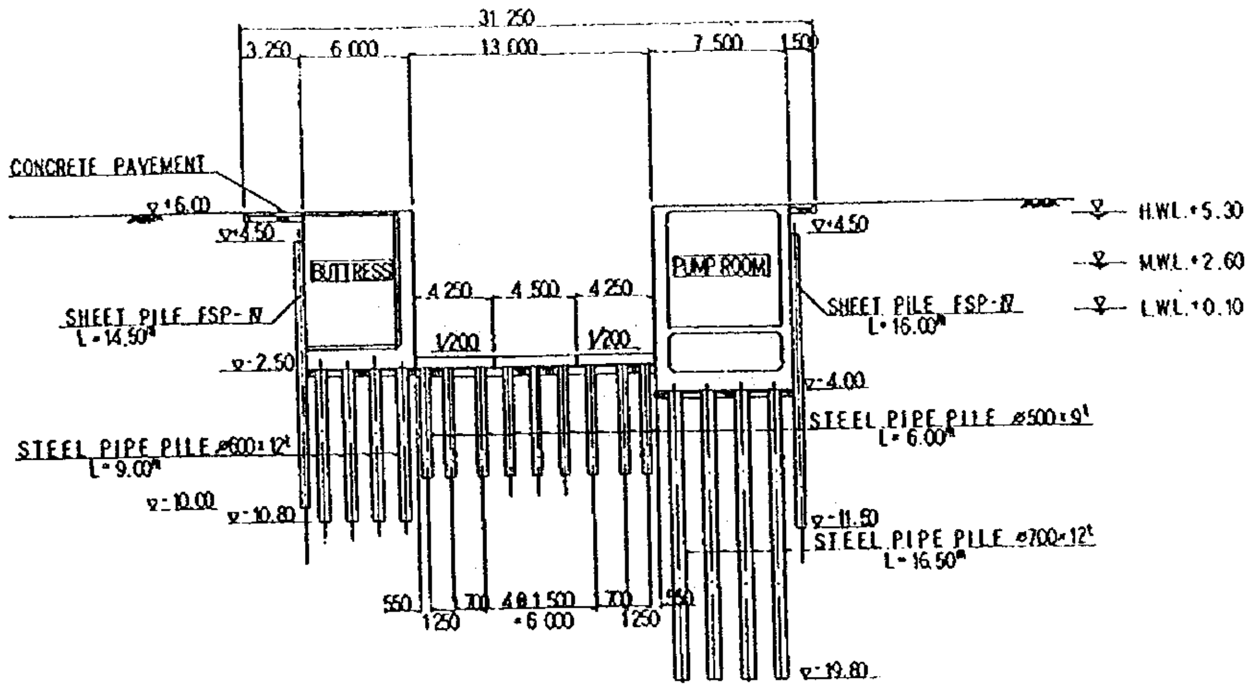


CROSS SECTION OF DRY DOCK S=1:200

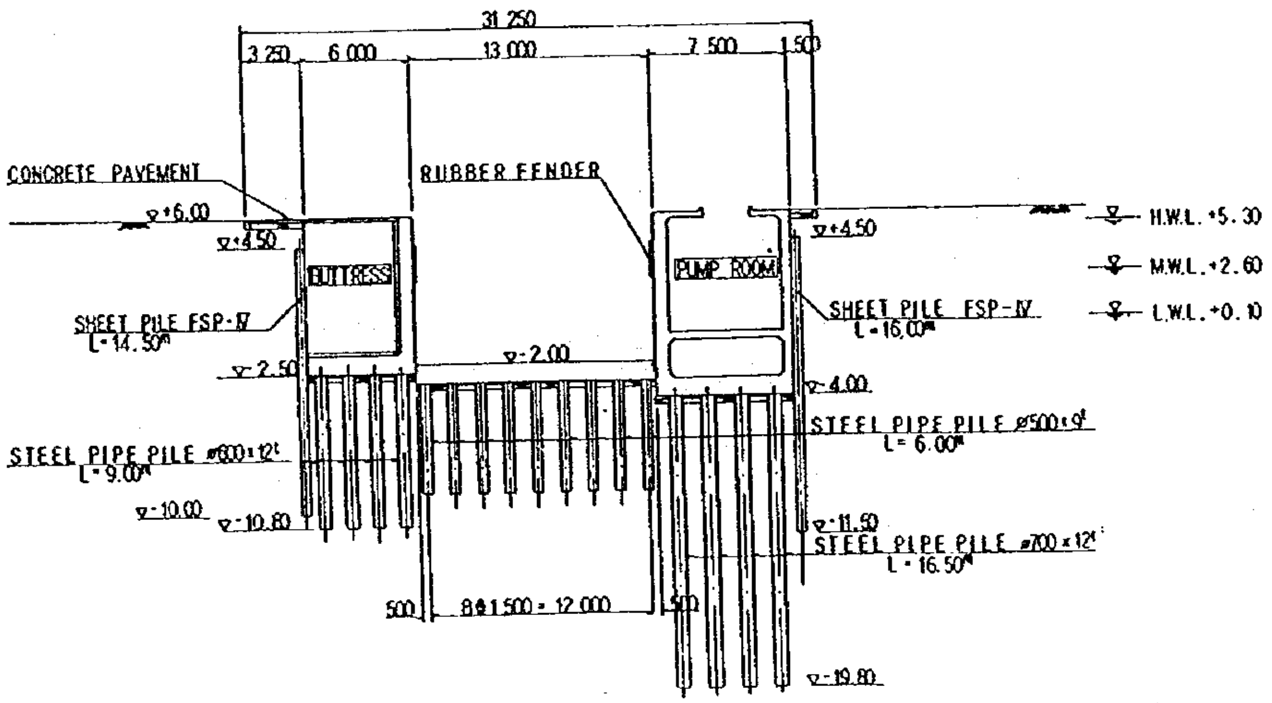
PLAN



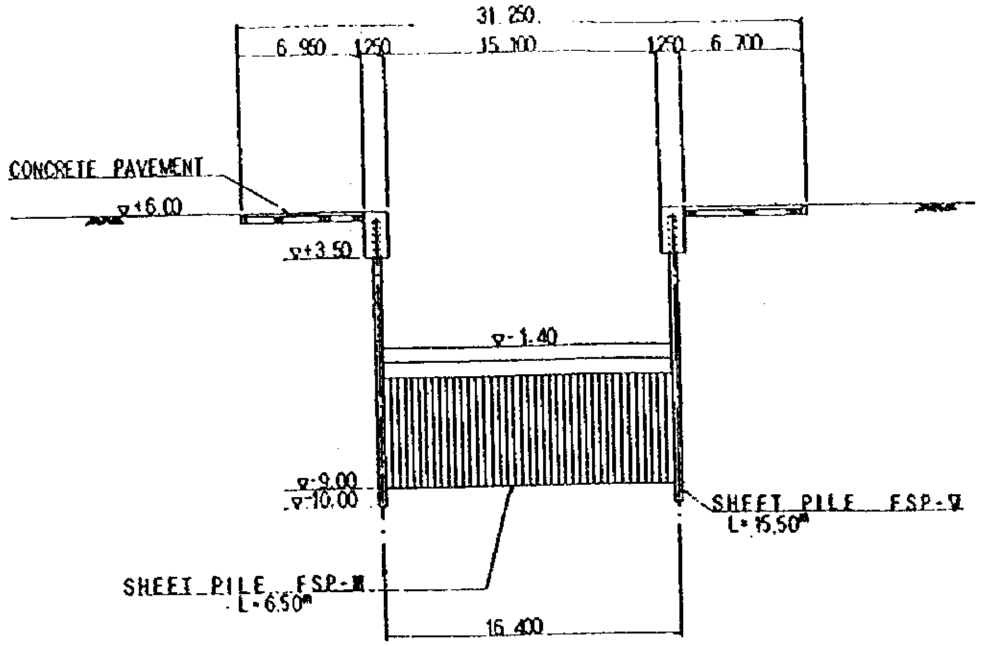
D – D

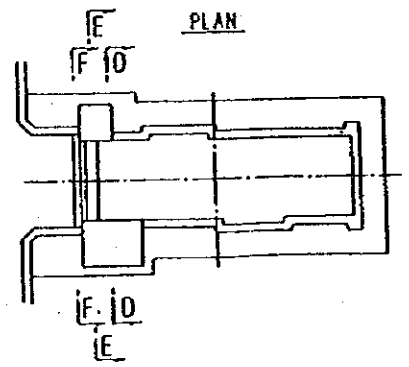


E – E

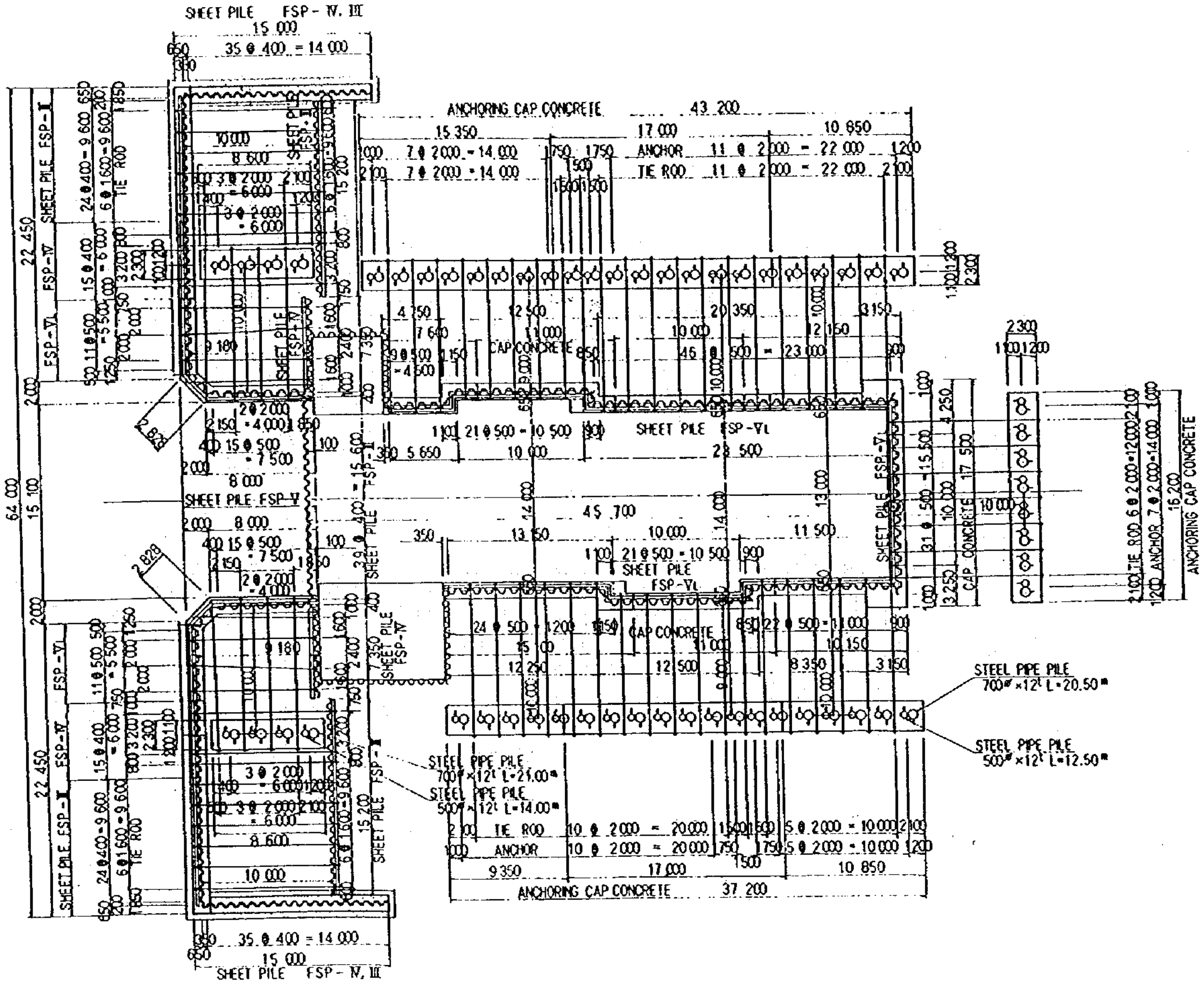


F – F

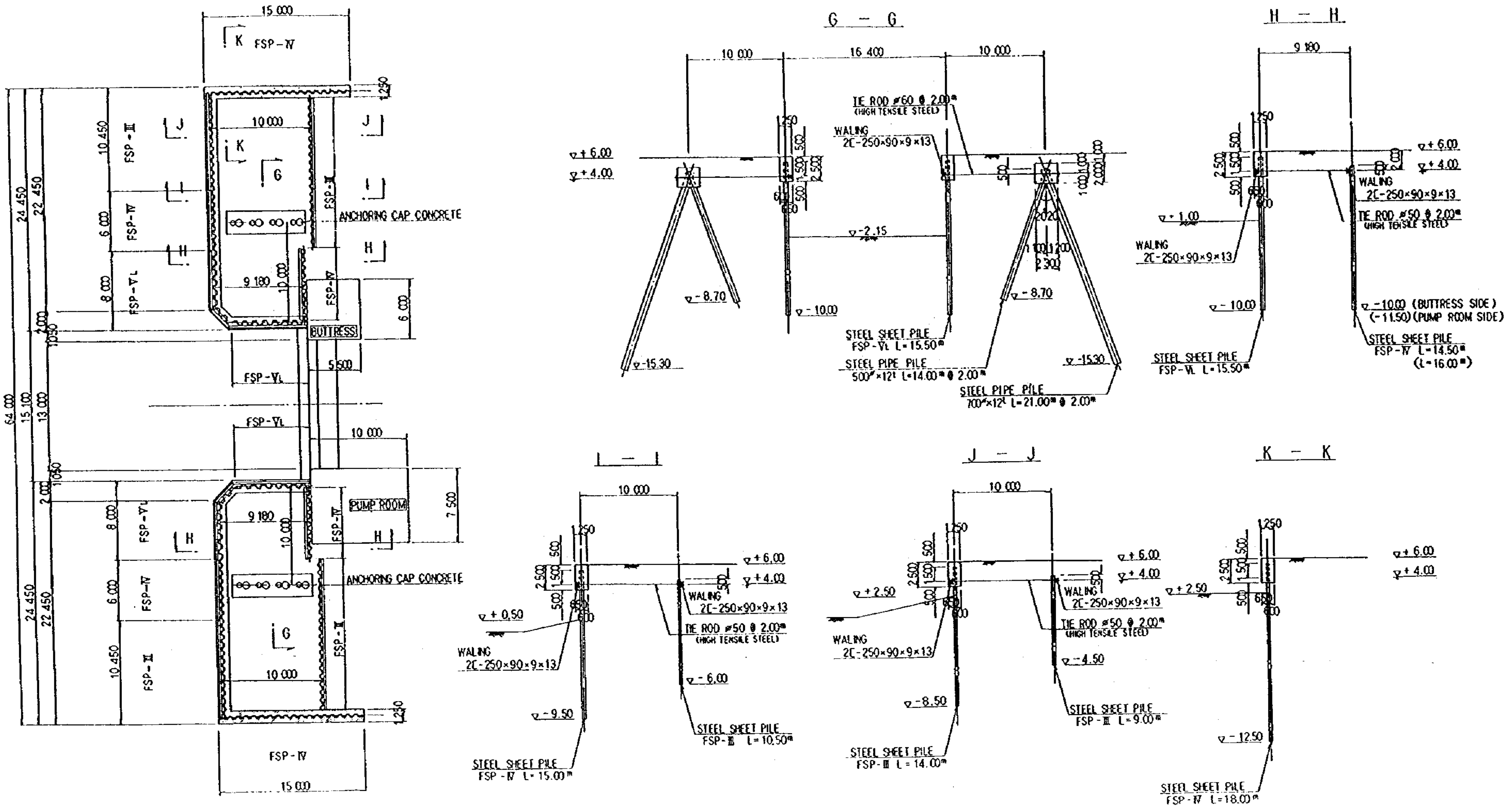




CROSS SECTION OF DRY DOCK S=1:200



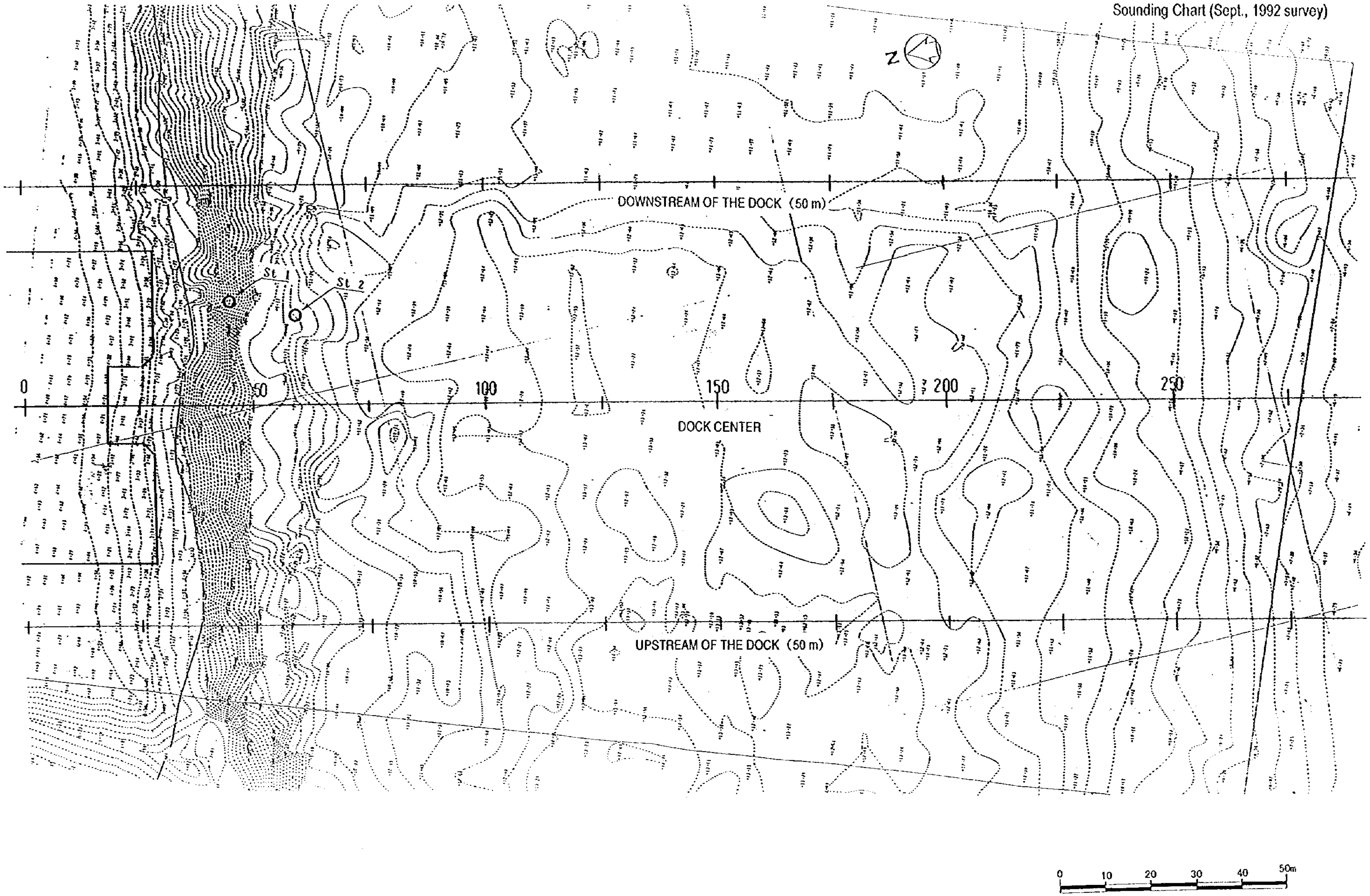
STEEL SHEET PILE AND
ANCHOR PILE ARRANGEMENT S=1:200



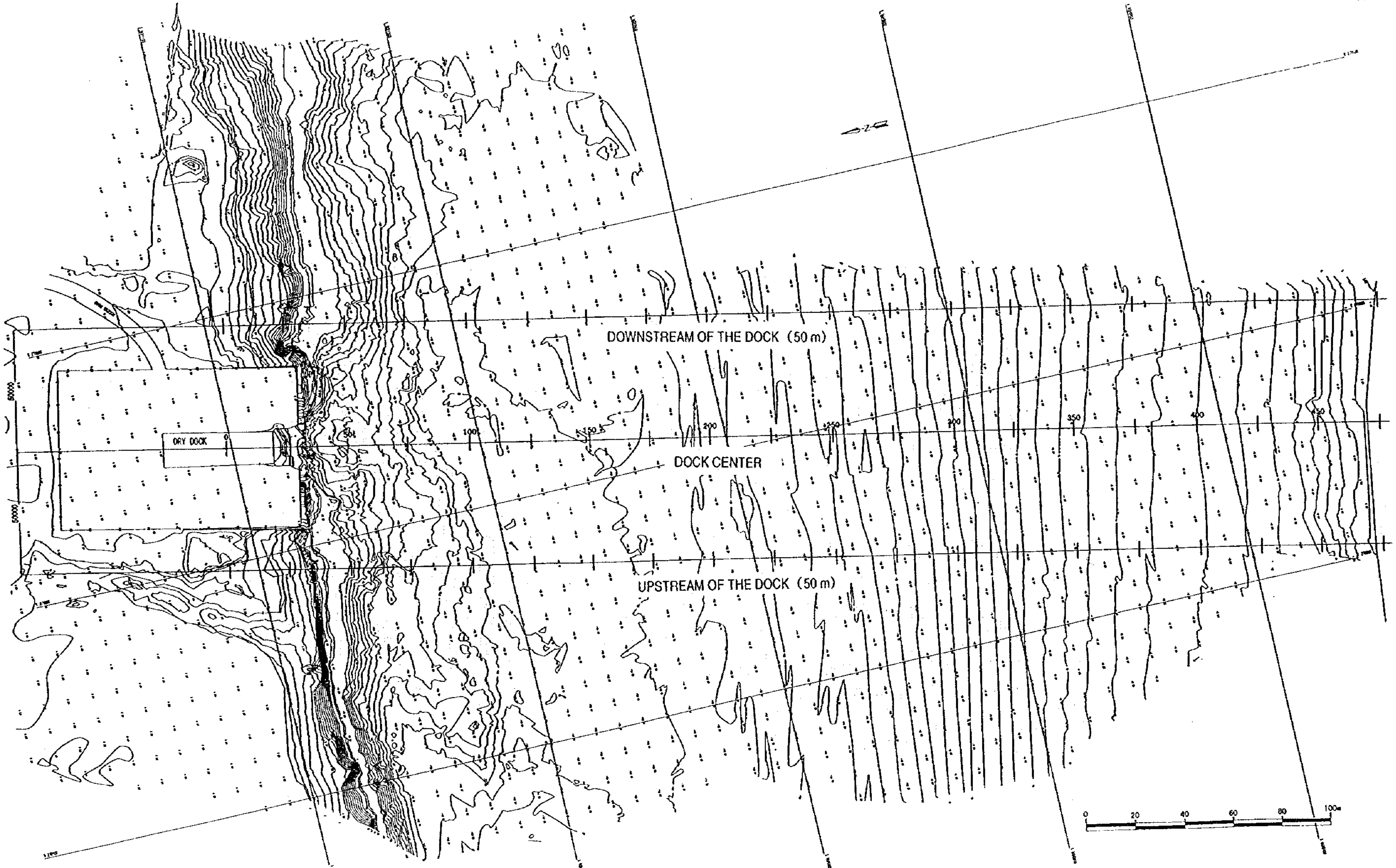
PLAN & CROSS SECTION OF ACCESS CHANNEL WALL S=1:200

APPENDIX--5-2-1 (1)

Sounding Chart (Sept., 1992 survey)

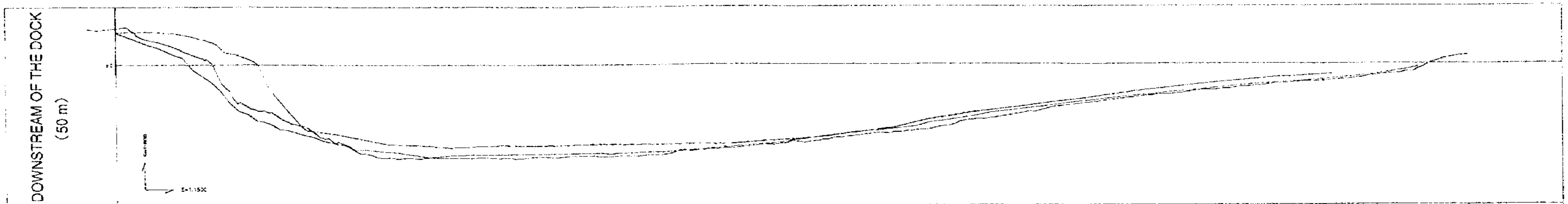


APPENDIX－5-2-1(2)
Sounding Chart (July., 1997 survey)

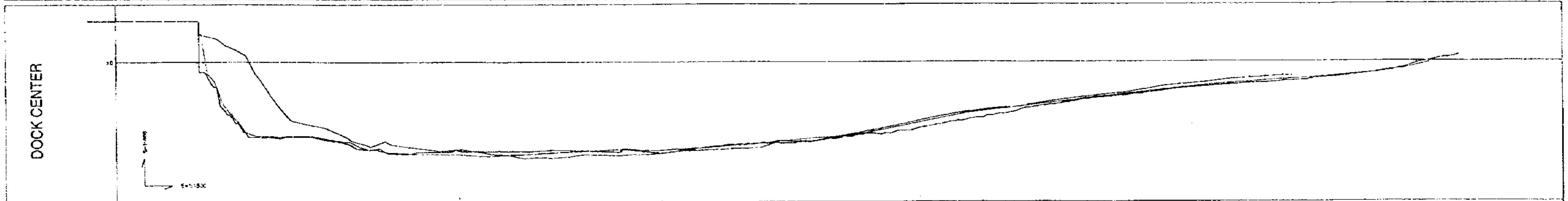


APPENDIX－5-2-1(3)
Sounding Chart (Dec., 1997 survey)

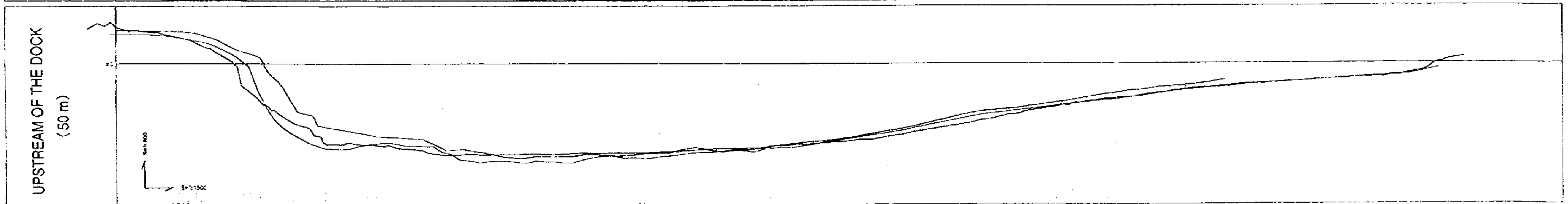
River Cross Section in Front of the Dock



92	DISTANCE	[illegible]
	LEVELS	[illegible]
97-7	DISTANCE	[illegible]
	LEVELS	[illegible]
97-12	DISTANCE	[illegible]
	LEVELS	[illegible]

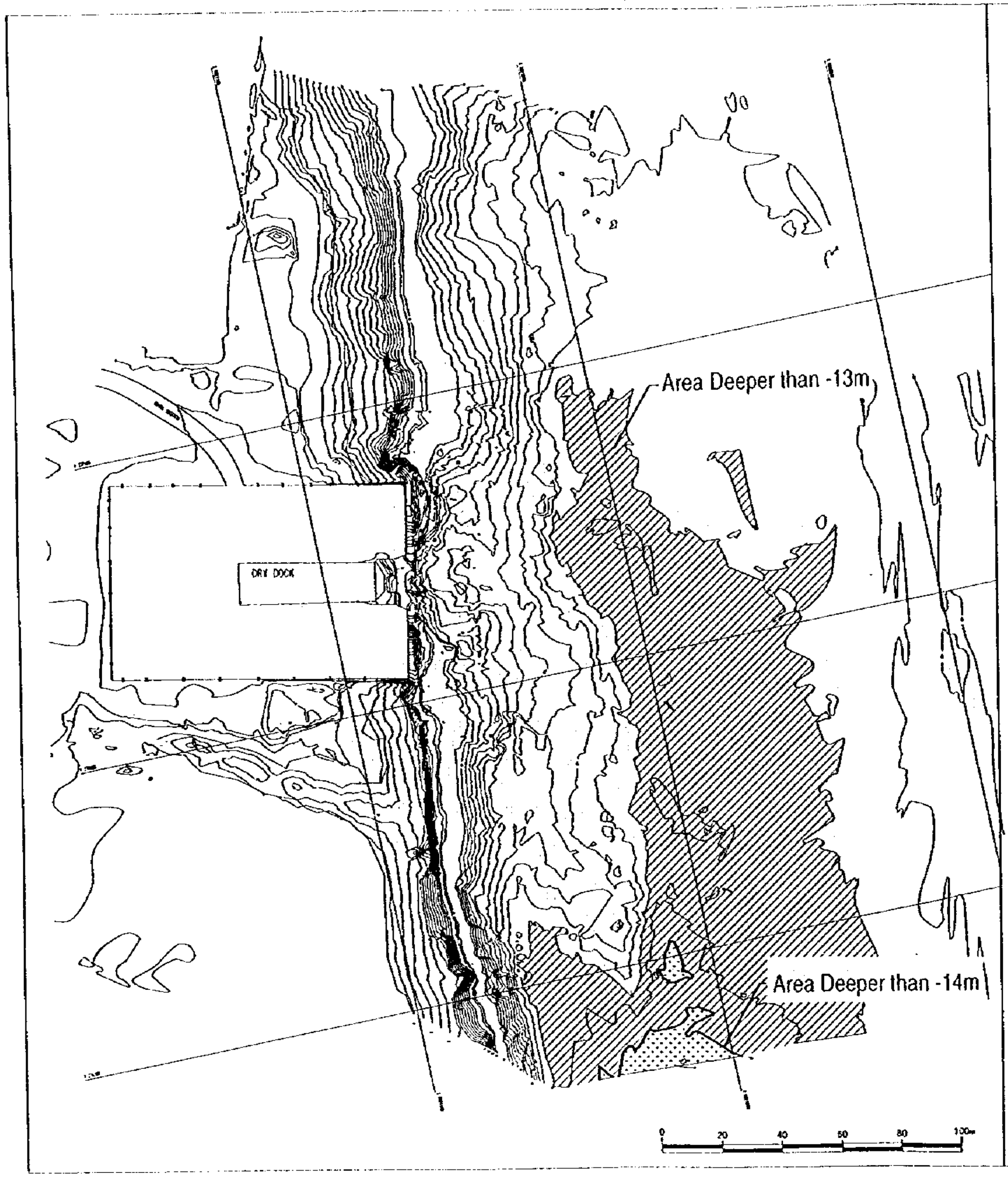


92	DISTANCE	[illegible]
	LEVELS	[illegible]
97-7	DISTANCE	[illegible]
	LEVELS	[illegible]
97-12	DISTANCE	[illegible]
	LEVELS	[illegible]

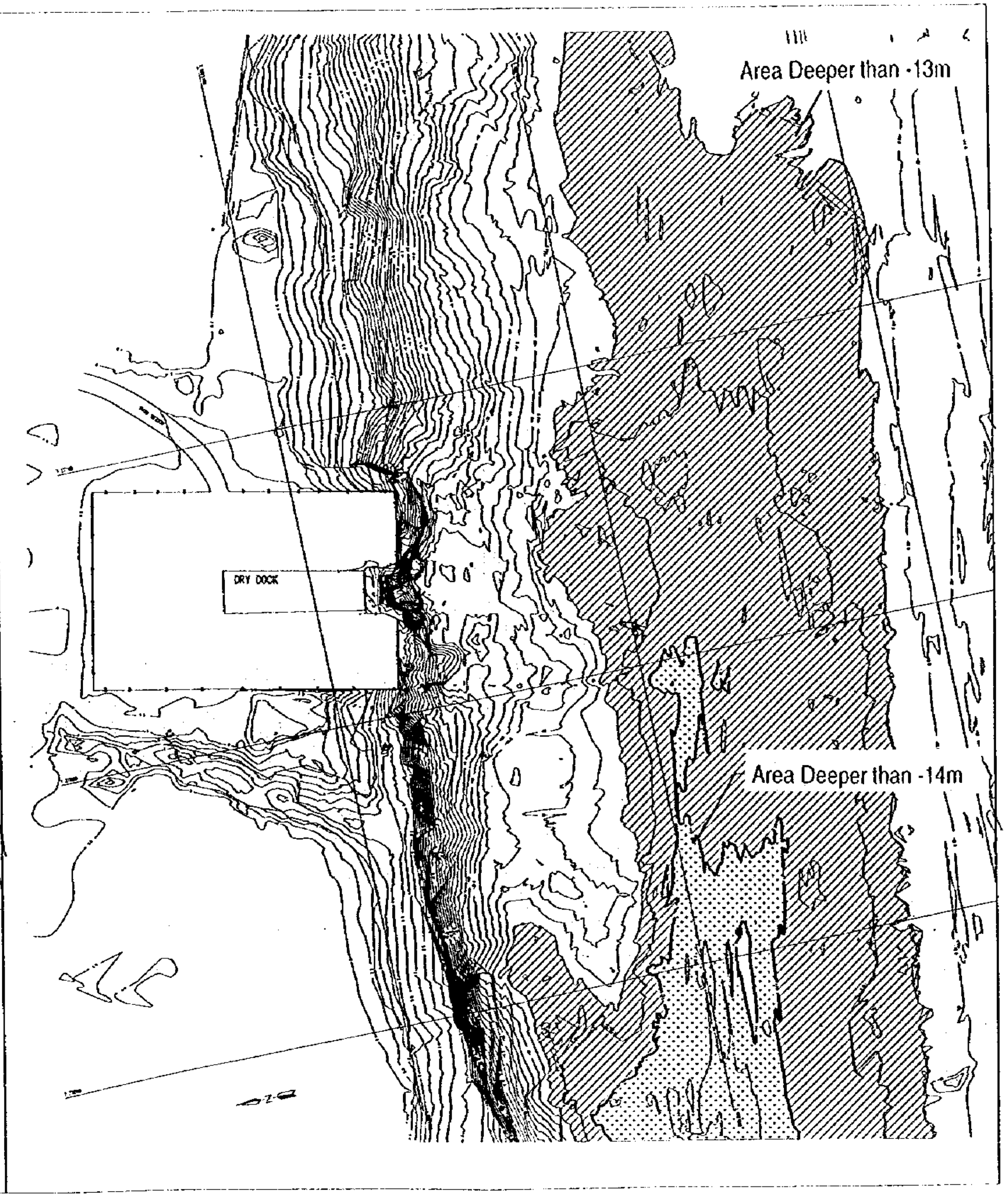


92	DISTANCE	[illegible]
	LEVELS	[illegible]
97-7	DISTANCE	[illegible]
	LEVELS	[illegible]
97-12	DISTANCE	[illegible]
	LEVELS	[illegible]

Difference of Riverbed between July 97 and Dec.97

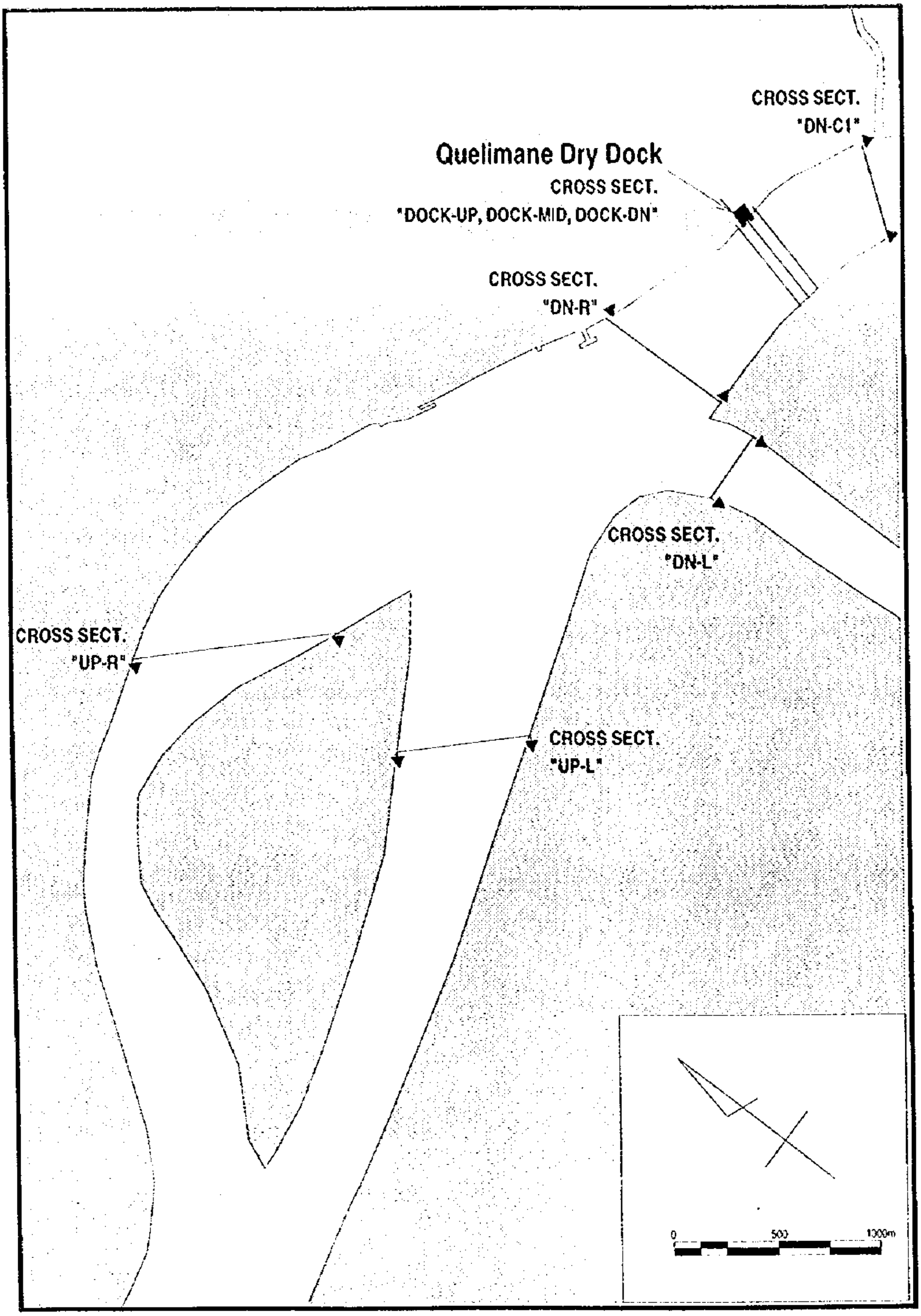


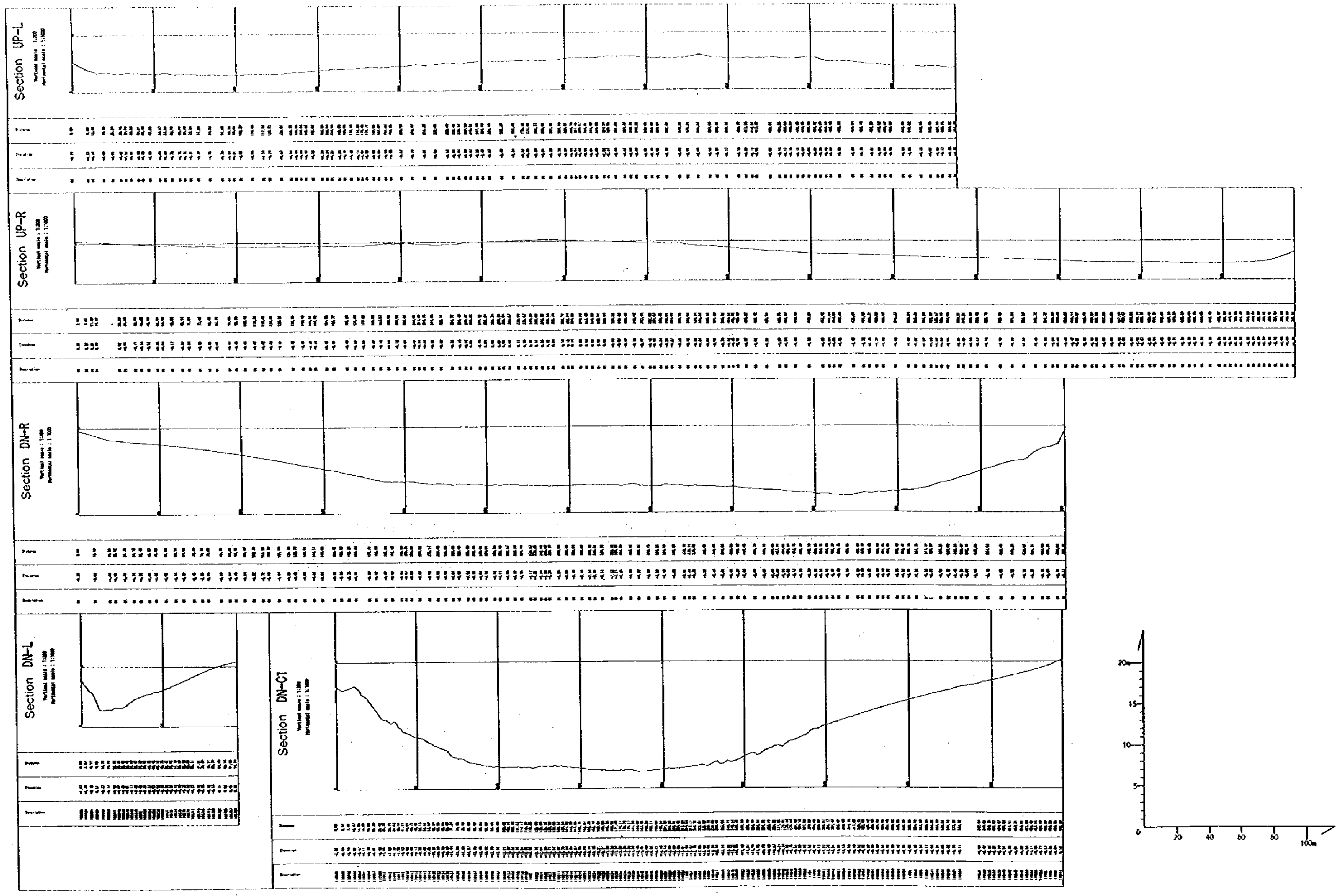
Sounding Chart (July., 1997 survey)



Sounding Chart (Dec., 1997 survey)

Location of Cross Sections




Section UP-L
Section UP-R
Section DN-R
Section DN-L
Section DN-C1

APPENDIX－5-2-2

Riverbank Observation Around the Project Site

Photo - 1
Upstream (UP-R) North
Gentle sedimentation area

Photo - 8
Downstream (UP-L) North
Heavily eroded area

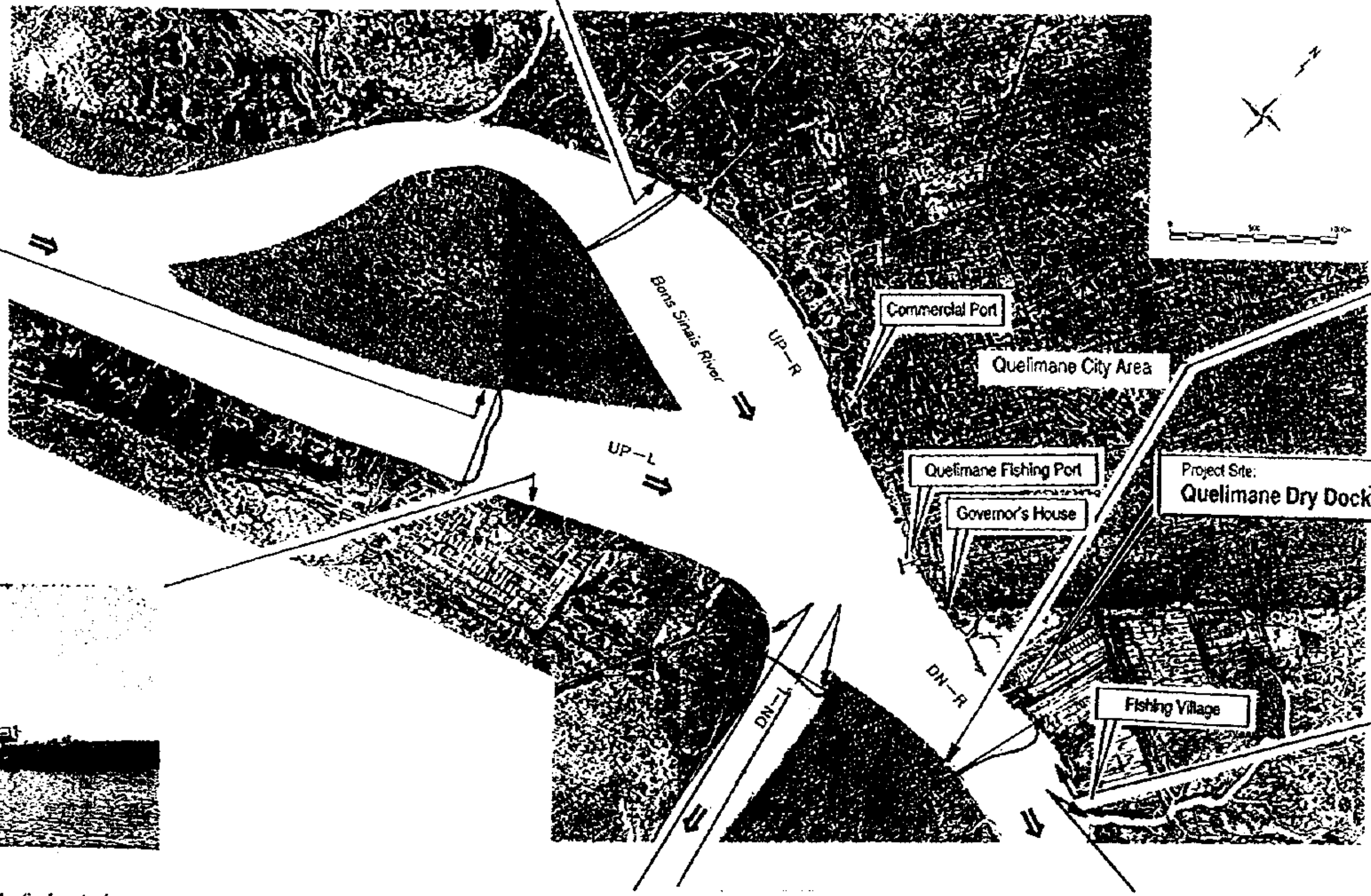


Photo - 2
Downstream (DN-R) South, Opposite of the Dock
Gentle sedimentation area

Photo - 3
Downstream (DN-R) North, Dock Side
View of the Dock from Fishing Village
Heavily eroded area

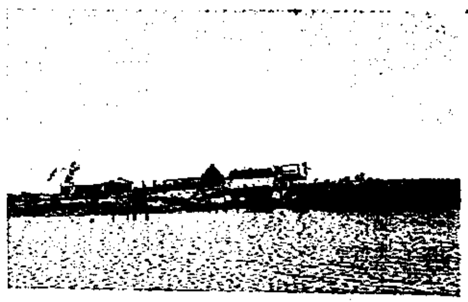


Photo - 7
Downstream (UP-L) South, (in front of pump station for aquaculture pond)
Heavily eroded area

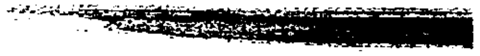


Photo - 6
Downstream (DN-L) West, at down-stream shoal.
Wide sedimentation area

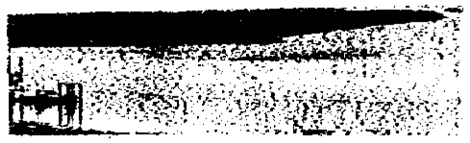
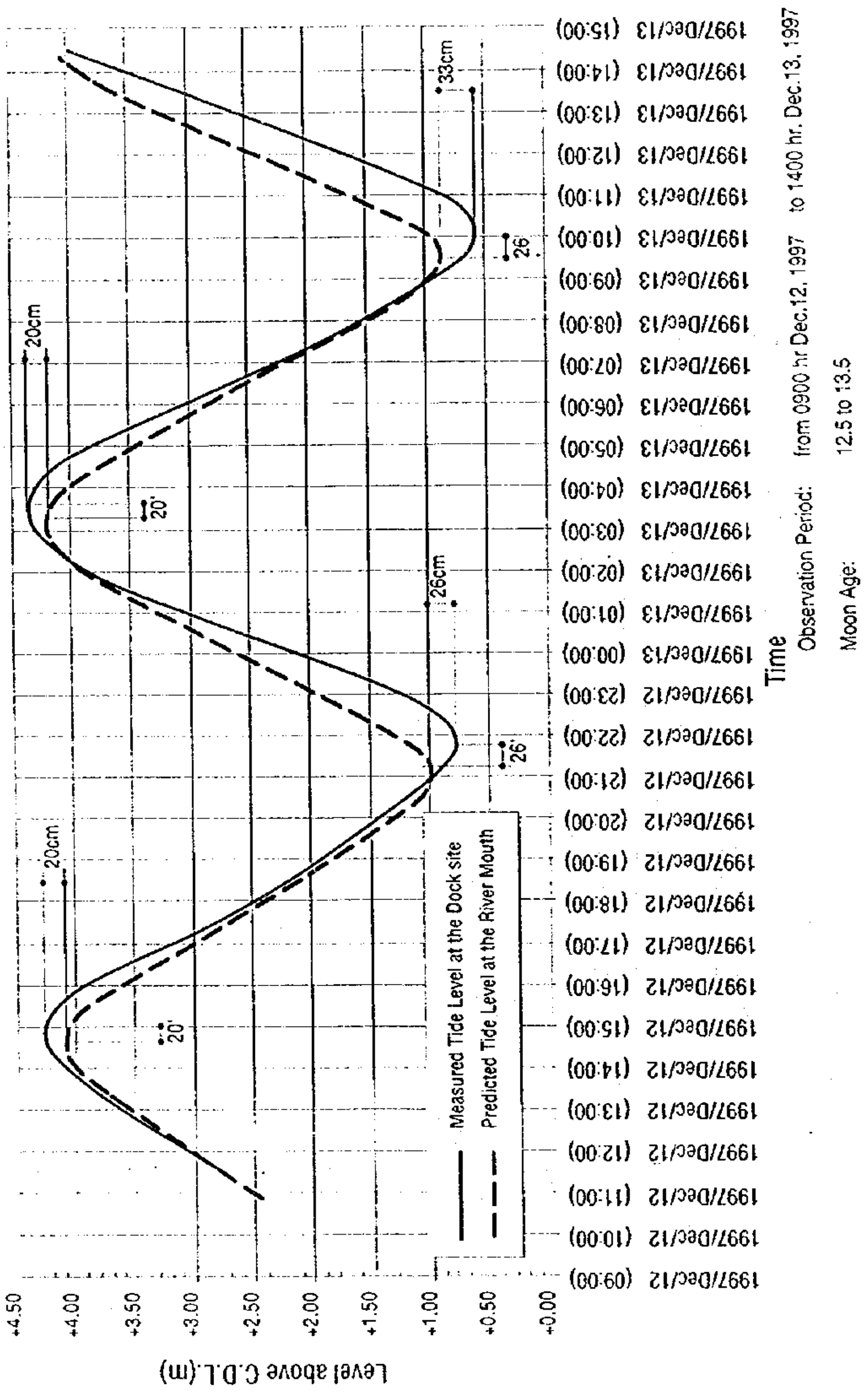


Photo - 5
Downstream (DN-L) East, at down-stream shoal.
Erosion is observed at center right

Photo - 4
Downstream (DN-R) North, Dock Side
View of the shore line of the downstream from Fishing Village.
Heavily eroded area

APPENDIX—5-2-3

Observation of Tide Level



Record of Flow Direction and Velocity Measurement

Observation Period: Dec.12, from 1997 to Dec.13, 1997

Moon Age: 12.5 - 13.5

Measuring Equipment: Doppler type self recoding Current Meter (Model RCM-9, Aanderaa Instruments, Inc.)

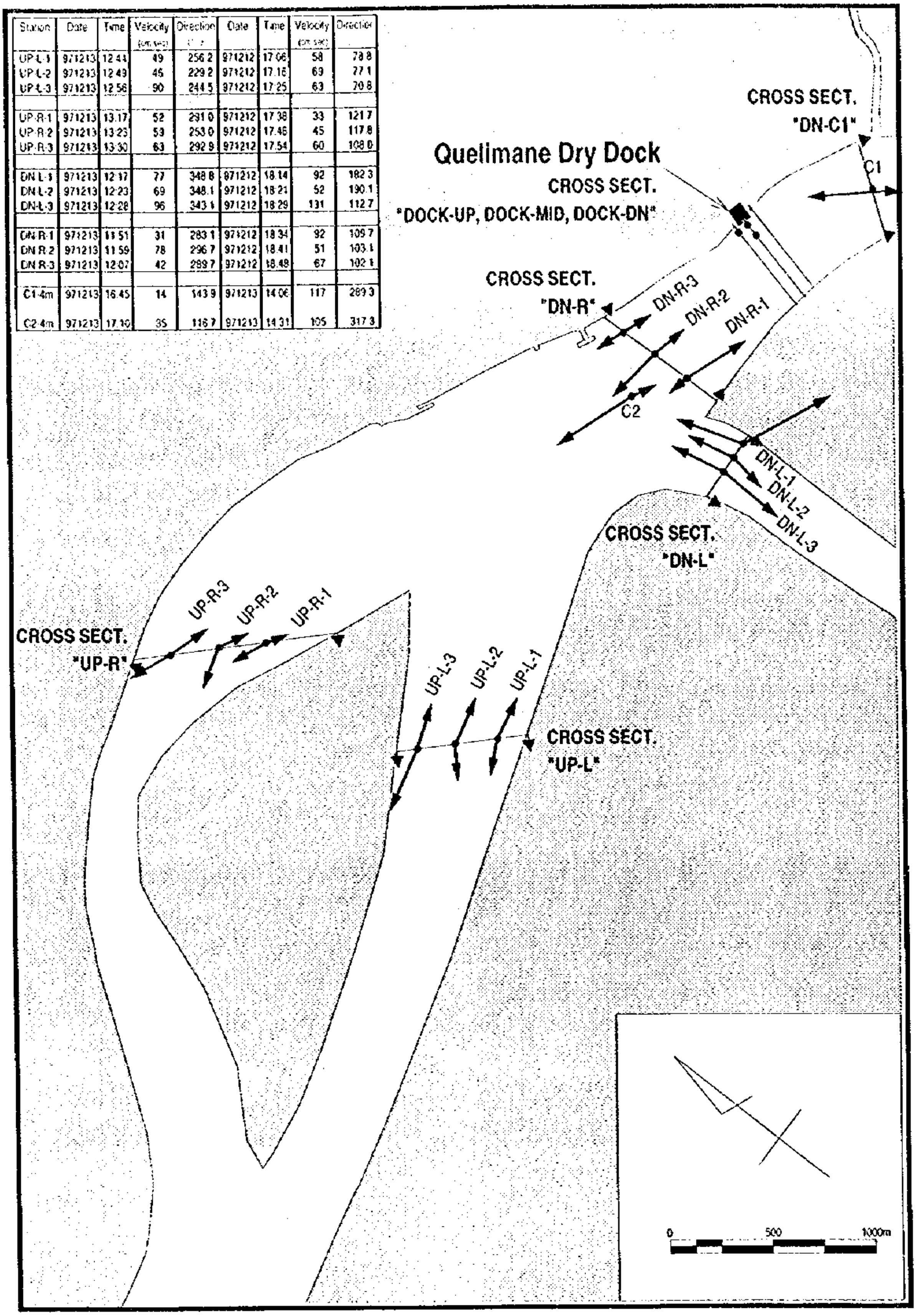
Station	Date	TIME (GMT+8hrs)	REFERENCE —	CURRENT SPEED CM/S	CURRENT DIRECTION Degrees Magnetic	TEMPERATURE Deg C	CONDUCTIVITY mS/cm	TURBIDITY FNU

SUMMARY OF AVERAGED CURRENT DATA

Station	Date	TIME	REFERENCE	CURRENT SPEED	CURRENT DIRECTION	TEMPERATURE	CONDUCTIVITY	TURBIDITY
UP-L-1	971212	17:06	946	58	78.8	30.6	54.1	23.4
UP-L-2	971212	17:16	946	69	77.1	30.6	53.9	21.0
UP-L-3	971212	17:25	946	63	70.8	30.6	53.8	21.9
UP-R-1	971212	17:38	946	33	121.7	30.6	54.1	23.4
UP-R-2	971212	17:46	946	45	117.8	30.6	53.7	22.8
UP-R-3	971212	17:54	946	60	108.0	30.5	52.9	18.1
DN-L-1	971212	18:14	946	92	182.3	30.6	53.8	23.3
DN-L-2	971212	18:21	946	52	190.1	30.5	53.7	23.4
DN-L-3	971212	18:29	946	131	112.7	28.5	53.7	23.4
DN-R-1	971212	18:34	946	92	109.7	30.6	53.5	23.4
DN-R-2	971212	18:41	946	51	103.1	30.6	53.6	23.4
DN-R-3	971212	18:48	946	67	102.1	30.4	53.4	23.4
DN-R-1	971213	11:51	946	31	283.1	30.4	51.8	23.4
DN-R-2	971213	11:59	946	78	296.7	30.3	51.4	23.3
DN-R-3	971213	12:07	946	42	289.7	30.3	38.6	17.6
DN-L-1	971213	12:17	946	77	348.8	30.6	53.0	23.4
DN-L-2	971213	12:23	946	69	348.1	30.7	53.4	23.4
DN-L-3	971213	12:28	946	96	343.1	30.7	53.4	23.4
UP-L-1	971213	12:44	946	49	256.2	30.5	51.9	23.4
UP-L-2	971213	12:49	946	46	229.2	30.4	51.5	23.4
UP-L-3	971213	12:56	946	90	244.5	30.3	51.9	23.4
UP-R-1	971213	13:17	946	52	291.0	30.5	52.6	23.4
UP-R-2	971213	13:23	946	59	253.0	30.5	52.5	23.4
UP-R-3	971213	13:30	946	63	292.9	30.4	52.3	23.4
C1-2m	971213	14:00	946	113	289.1	30.5	53.8	23.4
C1-4m	971213	14:06	946	117	289.3	30.4	54.1	23.4
C1-BT	971213	14:11	946	95	291.1	30.4	54.2	23.4
C2-2m	971213	14:29	946	104	319.9	30.3	54.6	23.4
C2-4m	971213	14:31	946	105	317.3	30.3	54.8	23.4
C2-BT	971213	14:35	946	99	309.5	30.3	54.9	23.4
C1-2m	971213	16:40	946	26	117.5	30.3	55.2	11.7
C1-4m	971213	16:45	946	14	143.9	30.2	55.8	15.6
C1-BT	971213	16:49	946	16	143.8	30.2	55.8	19.3
C2-2m	971213	17:06	946	37	109.5	30.3	54.9	10.4
C2-4m	971213	17:10	946	35	116.7	30.3	55.1	11.8
C2-BT	971213	17:15	946	27	128.3	30.3	55.5	15.2

APPENDIX－5-2-4. (2)

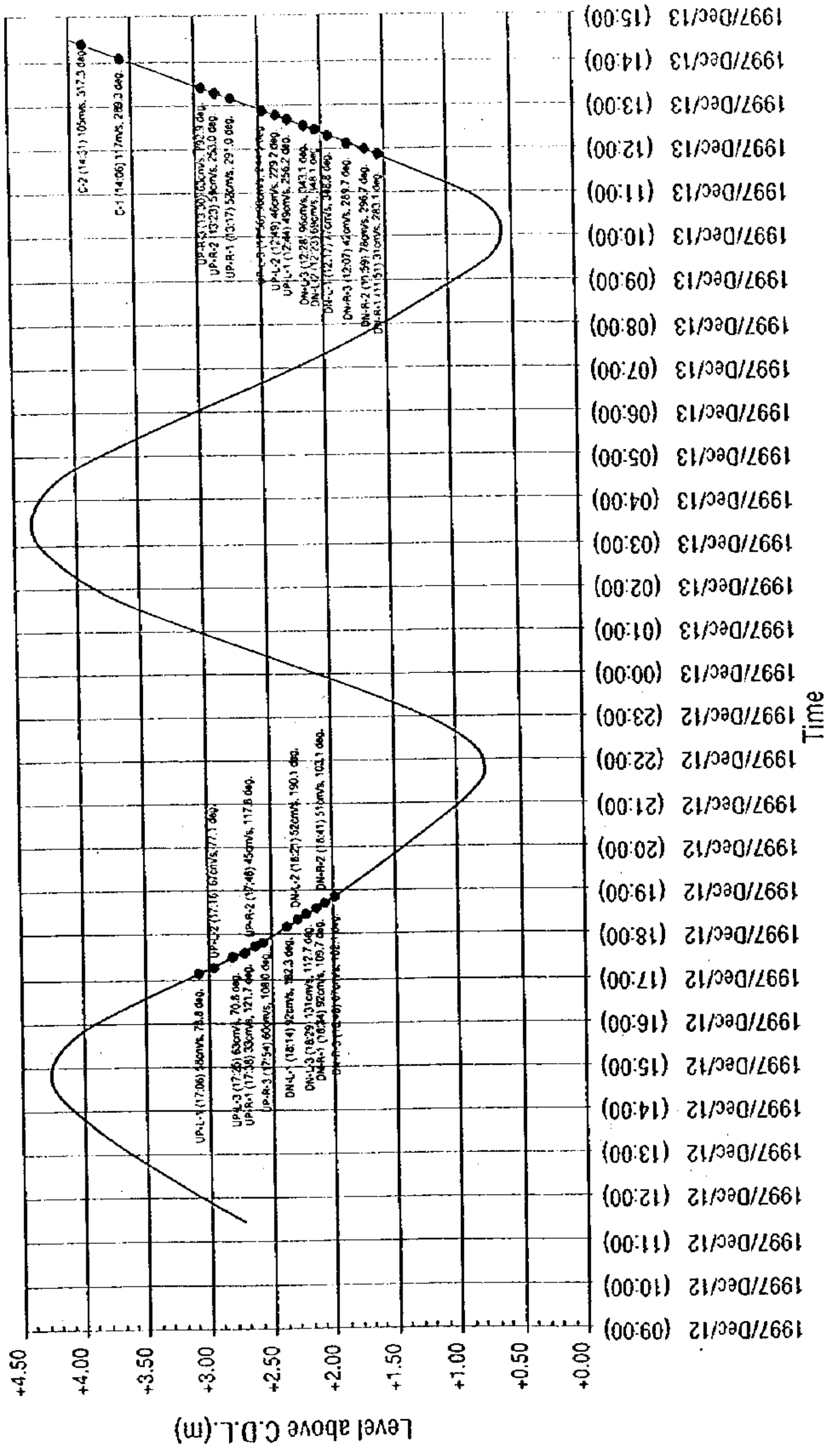
Maximum Flow Velocity and Direction at Each Point



Station	Date	Time	Velocity (cm/sec)	Direction [illegible]	Date	Time	Velocity (cm/sec)	Direction
UP-L-1	971213	12.44	49	256.2	971212	17.06	58	78.8
UP-L-2	971213	12.49	45	229.2	971212	17.16	69	77.1
UP-L-3	971213	12.56	90	244.5	971212	17.25	63	70.8
UP-R-1	971213	13.17	52	291.0	971212	17.38	33	121.7
UP-R-2	971213	13.23	53	253.0	971212	17.46	45	117.8
UP-R-3	971213	13.30	63	292.9	971212	17.54	60	108.0
DN-L-1	971213	12.17	77	348.8	871212	18.14	92	182.3
DN-L-2	971213	12.23	69	348.1	971212	18.21	52	190.1
DN-L-3	971213	12.28	96	343.1	971212	18.29	131	112.7
DN-R-1	971213	11.51	31	283.1	971212	18.34	92	109.7
DN-R-2	971213	11.59	78	296.7	971212	18.41	51	103.1
DN-R-3	971213	12.07	42	289.7	971212	18.48	67	102.1
C1-4m	971213	16.45	14	143.9	971213	14.06	117	289.3
C2-4m	971213	17.10	35	116.7	971213	14.31	105	317.3

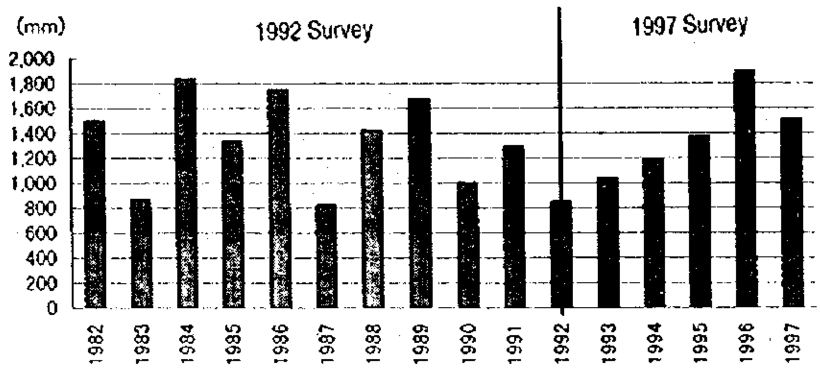
APPENDIX—5-2-4. (3)

Flow Direction / Velocity and Tide Level



APPENDIX－5-2-5. (1)
Monthly Precipitation in Quelimane

YEAR/MONTH	1	2	3	4	5	6	7	8	9	10	11	12	TOTAL
1982	483.1	245.1	23.4	171.2	50.8	26.6	62.8	23.4	86.8	159.4	49.6	112.7	1,494.9
1983	86.6	193.8	140.7	32.9	60.9	20.4	95.7	47.1	0.6	29.5	8.4	150.7	867.3
1984	403.4	341.6	365.0	88.5	113.1	73.8	41.1	29.3	0.2	73.1	237.1	71.0	1,837.2
1985	252.4	195.9	104.0	218.0	54.8	42.5	14.0	29.7	1.9	74.2	140.3	206.0	1,333.7
1986	385.7	289.0	337.2	269.9	47.8	44.5	59.9	1.0	4.3	105.5	12.3	189.0	1,746.1
1987	279.9	96.8	160.2	93.2	35.5	52.9	10.8	9.4	6.6	20.2	12.4	44.8	822.7
1988	217.7	293.2	239.9	92.5	29.8	59.2	73.2	24.6	1.9	32.6	104.2	254.5	1,423.3
1989	150.8	410.8	357.2	100.1	37.9	72.7	31.0	2.8	18.7	35.9	178.5	281.1	1,677.5
1990	272.5	127.7	43.2	56.4	163.3	96.6	22.1	46.8	19.6	2.5	70.6	78.7	1,000.0
1991	149.3	315.6	291.7	178.3	29.0	41.9	52.2	19.0	59.5	0.0	83.0	73.2	1,292.7
1992	237.9	191.7	120.3	26.1	25.2	75.0	49.8	25.4	0.0	7.4	35.8	54.8	849.4
1993	348.9	138.0	81.7	38.3	88.7	115.8	35.2	40.6	0.0	2.9	97.4	51.3	1,038.8
1994	228.9	168.7	296.5	110.1	33.0	97.7	52.3	32.9	11.4	14.1	38.0	104.1	1,187.7
1995	301.2	263.8	34.2	106.4	193.8	60.1	56.9	77.8	0.2	0.3	24.3	251.5	1,370.5
1996	465.3	299.0	308.6	81.7	30.7	76.7	407.1	3.8	0.1	9.2	7.8	207.1	1,897.1
1997	156.5	586.1	117.9	94.6	40.8	2.4	91.7	17.5	40.7	32.3	116.9	213.9	1,511.3
MEAN	276.3	259.8	188.9	109.9	64.7	59.9	72.2	26.9	15.8	37.4	76.0	146.5	1,334.4



APPENDIX – 5-2-5. (2)
Analysis of Daily Precipitation in Quelimane (1993 - 1997)

1993	1	2	3	4	5	6	7	8	9	10	11	12	Total	Average	Max
avg /month	343.9	138.0	81.7	38.3	88.7	115.8	35.2	40.6	0.0	2.9	97.4	51.3	1,033.3	86.6	343.9
max /daily	70.7	29.9	17.0	22.1	4.2	25.8	9.6	15.6	0.0	1.9	51.5	26.3	---	22.9	70.7
Rainy Days 0.1 mm<=	16	18	15	5	4	16	7	10	0	2	6	5	104		18
1.0 mm<=	13	14	13	5	2	11	6	6	0	2	4	4	80		14
10.0 mm<=	9	4	4	2	0	3	0	1	0	0	3	2	28		9
50.0 mm<=	3	0	0	0	0	0	0	0	0	0	1	0	4		3
Rain continuously over 25mm	153.0	26.3	0.0	0.0	0.0	25.8	0.0	0.0	0.0	0.0	51.8	26.3	283		153
days for ditto	3	1	0	0	0	1	0	0	0	0	1	1	7		3

1994	1	2	3	4	5	6	7	8	9	10	11	12	Total	Average	Max
avg /month	228.9	163.7	295.5	110.1	33.0	97.7	52.3	32.9	114.0	14.1	33.0	104.1	1,290.3	107.5	295.5
max /daily	62.9	61.1	88.2	23.9	12.7	39.9	11.3	15.0	4.9	2.2	14.5	43.0	---	31.6	88.2
Rainy Days 0.1 mm<=	15	15	18	13	8	12	13	8	3	5	6	7	123		18
1.0 mm<=	14	11	14	12	5	11	8	6	3	2	3	6	95		14
10.0 mm<=	8	6	8	5	2	3	1	1	0	0	2	3	39		8
50.0 mm<=	1	1	2	0	0	0	0	0	0	0	0	0	4		2
Rain continuously over 25mm	62.9	61.1	88.2	0.0	0.0	39.9	0.0	0.0	0.0	0.0	0.0	43.0	295		88
days for ditto	1	1	1	0	0	1	0	0	0	0	0	0	4		1

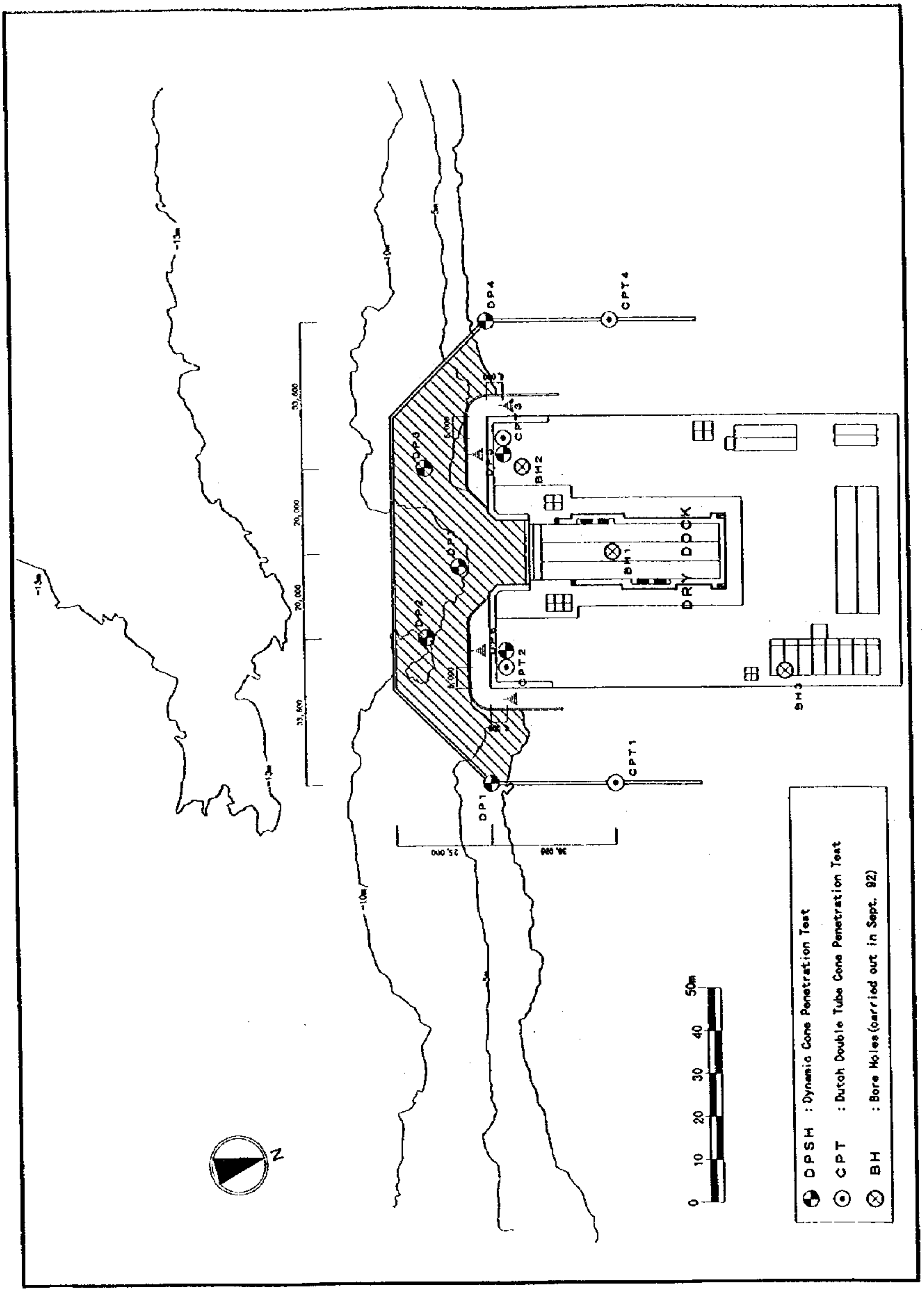
1995	1	2	3	4	5	6	7	8	9	10	11	12	Total	Average	Max.
avg /month	301.2	263.8	34.2	106.4	193.8	60.1	56.9	7.7	0.2	0.3	24.3	251.5	1,300.4	108.4	301.2
max /daily	76.3	74.8	34.4		28.5	23.9		4.8	0.2	0.3	20.7	86.3	---	35.0	86.3
Rainy Days 0.1 mm<=	13	18	12		19	8		4	1	1	4	12	92		19
1.0 mm<=	13	14	10		16	4		2	0	0	3	11	73		16
10.0 mm<=	8	7	4		8	2		0	0	0	2	7	38		8
50.0 mm<=	2	2	0		0	0		0	0	0	0	1	5		2
Rain continuously over 25mm	136.4	152.0	34.4		28.5	0.0		0.0	0.0	0.0	0.0	86.3	437.6		152.0
days for ditto	2	2	1		1			0	0	0	0	1	7		2

1996	1	2	3	4	5	6	7	8	9	10	11	12	Total	Average	Max.
avg /month	465.3	299.0	308.6	81.7	30.7	76.7	407.1	3.8	0.1	9.2	7.8	207.1	1,897.1	158.1	465.3
max. /daily	206.0	83.7	70.4	31.0	12.8	30.1	133.5	1.7	0.1	8.9	7.1	52.4	---	53.1	206.0
Rainy Days 0.1 mm<=	15	18	21	16	7	9	8	6	1	2	3	12	118		21
1.0 mm<=	11	13	17	11	6	6	8	1	0	1	1	11	86		17
10.0 mm<=	8	6	9	2	1	3	4	0	0	0	0	7	40		9
50.0 mm<=	2	1	1	0	0	0	4	0	0	0	0	1	9		4
Rain continuously over 25mm	254.6	83.7	70.4	31.0	0.0	158.0	397.8	0.0	0.0	0.0	0.0	102.4	1098		398
days for ditto	2	1	1	1	0	1	5	0	0	0	0	2	13		5

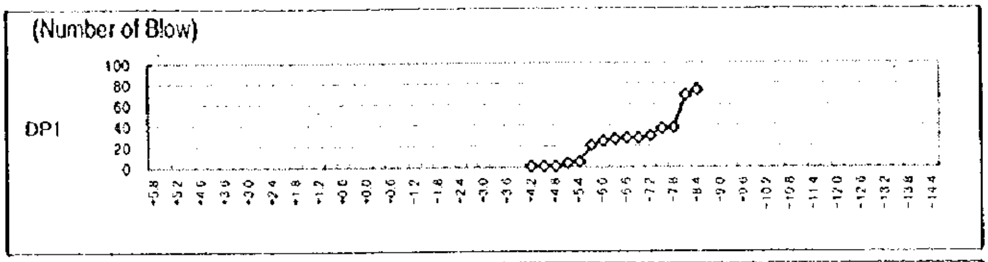
1997	1	2	3	4	5	6	7	8	9	10	11	12	Total	Average	Max.
avg /month	156.5	586.1	117.9	94.6	40.8	2.4	91.7	17.5	40.7	32.3	116.9	213.9	1,511.3	125.9	586.1
max. /daily	36.2	144.8	43.7	26.2	10.0	2.4	21.2	15.4	16.2	21.8	43.8		---	35.2	144.8
Rainy Days 0.1 mm<=	17	24	10	15	7	1	21	2	6	6	7		116		24
1.0 mm<=	13	24	7	13	6	1	16	2	4	4	6		96		24
10.0 mm<=	7	15	4	3	1	0	9	1	1	1	5		47		15
50.0 mm<=	0	2	0	0	3	0	0	0	0	0	0		5		3
Rain continuously over 25mm	36.2	184.4	43.7	26.2	0.0	0.0	0.0	0.0	0.0	0.0	43.8		339		184
days for ditto	1	4	1	1	0	0	0	0	0	0	1		8		4

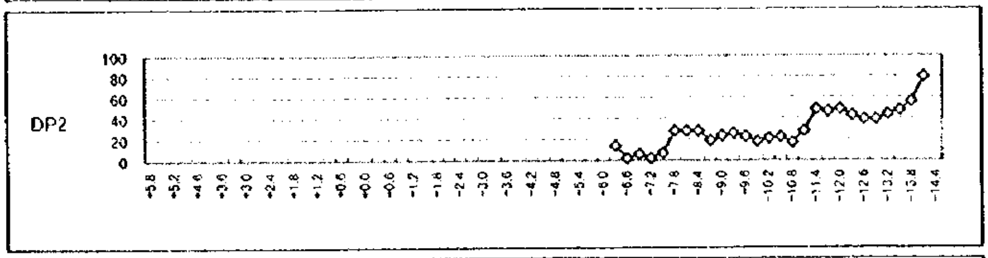
Summery 1993-1997	1	2	3	4	5	6	7	8	9	10	11	12	Total	Average	Max.
avg /month	300.2	291.1	167.8	86.2	77.4	70.5	128.6	20.5	31.0	11.8	56.9	165.6	1,407.6	117.3	300.2
max. /daily	206.0	144.8	83.2	31.0	23.5	39.9	133.5	15.6	16.2	21.8	51.8	86.3	---	72.0	206.0
Rainy Days 0.1 mm<=	15.2	18.6	15.2	12.3	9.0	9.2	12.3	6.0	2.2	3.2	5.2	9.0	117		19
1.0 mm<=	12.8	15.2	12.2	10.3	7.0	6.6	9.5	3.4	1.4	1.8	3.4	8.0	92		15
10.0 mm<=	8.0	7.6	5.8	3.0	2.4	2.2	3.5	0.6	0.2	0.2	2.4	4.8	41		8
50.0 mm<=	1.6	1.2	0.6	0.0	0.6	0.0	1.0	0.0	0.0	0.0	0.2	0.5	6		2
Rain continuously over 25mm	254.6	184.4	83.2	31.0	28.5	158.0	397.8	0.0	0.0	0.0	51.8	102.4	1297		398
days for ditto	3	4	1	1	1	1	5	0	0	0	1	2	19		5

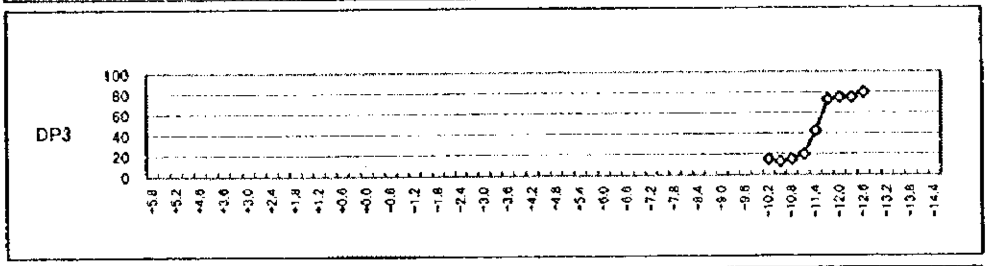
APPENDIX—5-2-6. (1)
Location of Soil Investigation

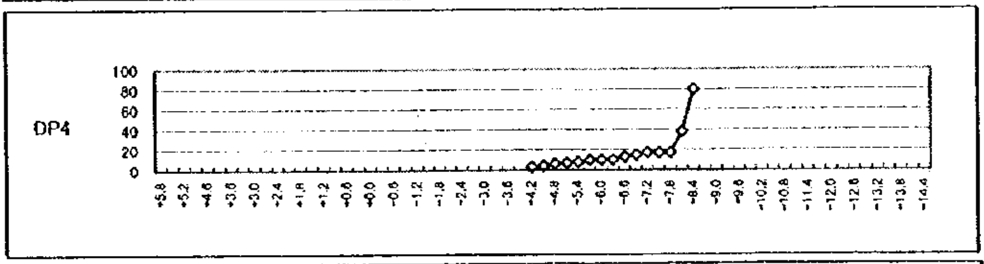


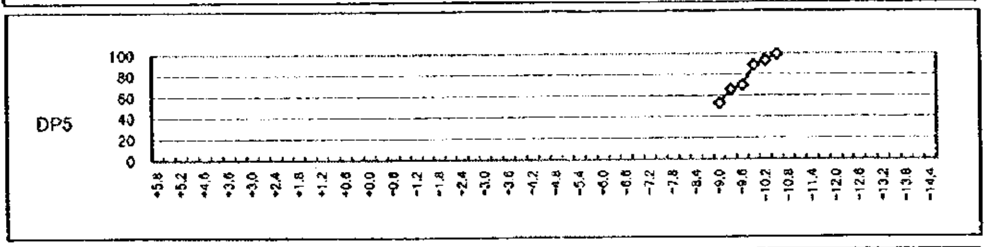
Result of Soil Investigation (Dynamic Cone Penetration Test)

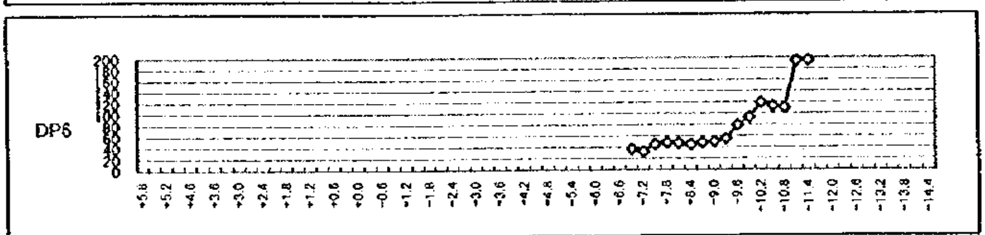


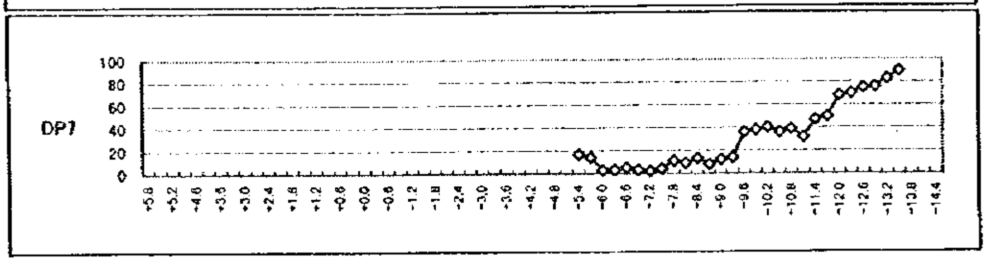




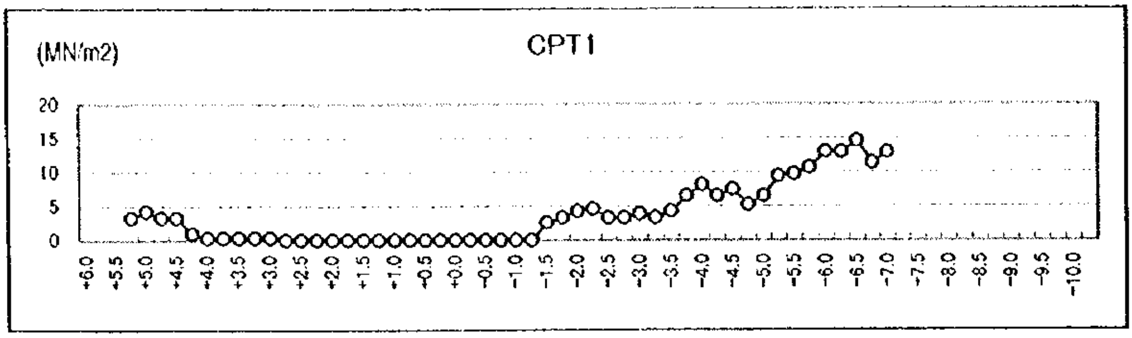


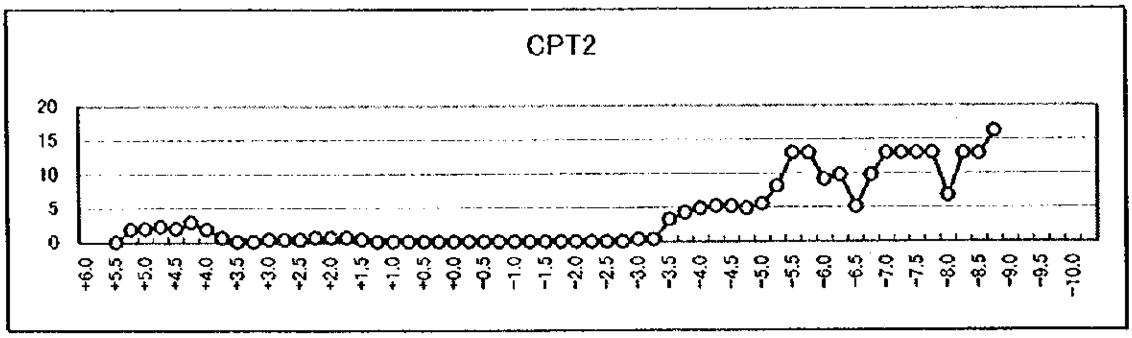


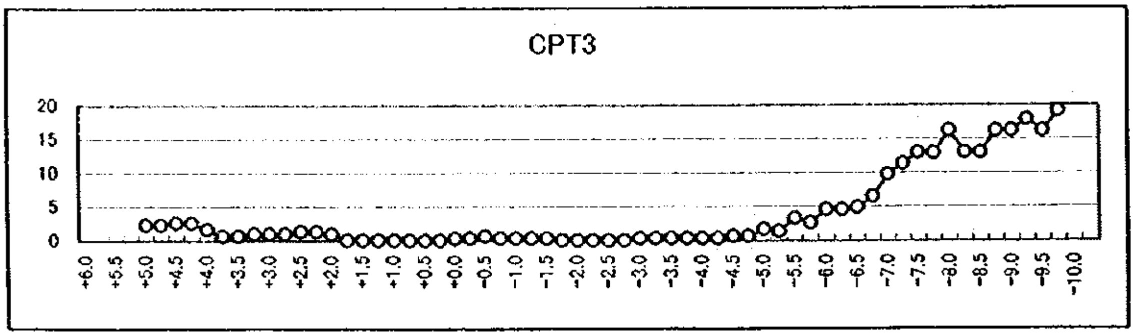




Result of Soil Investigation (Dutch Cone Penetration Test)







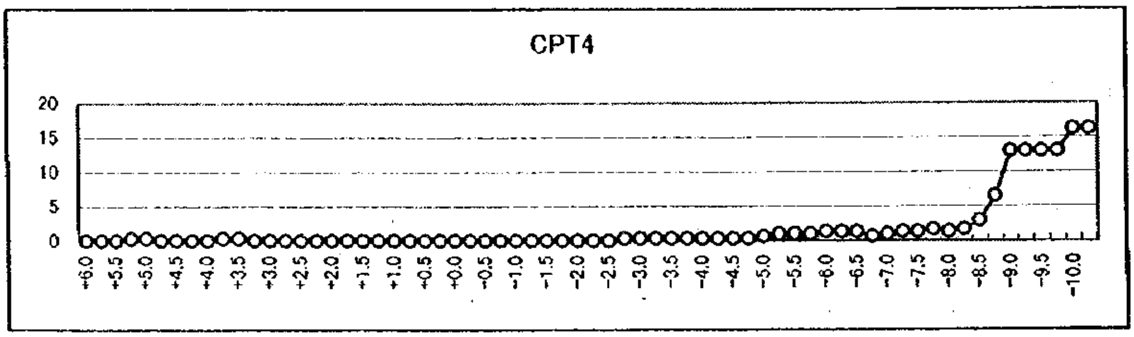


Photo - 9
Gap between Buttress & Revetment superstructure. (Upstream side)
Value on the board showing distance between two marking points.
(actual gap at Point "A" is 87 mm)

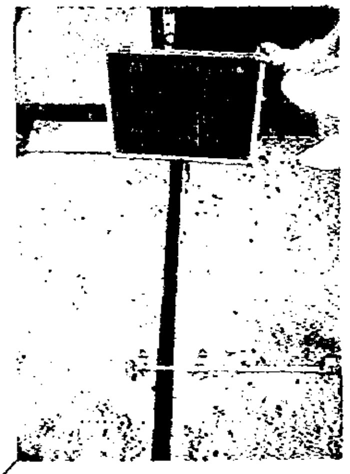


Photo -1
Gap between Pump Room & Revetment superstructure. (Downstream side)
Value on the board showing distance between two marking points.
(actual gap at Point "C" is 52 mm.)

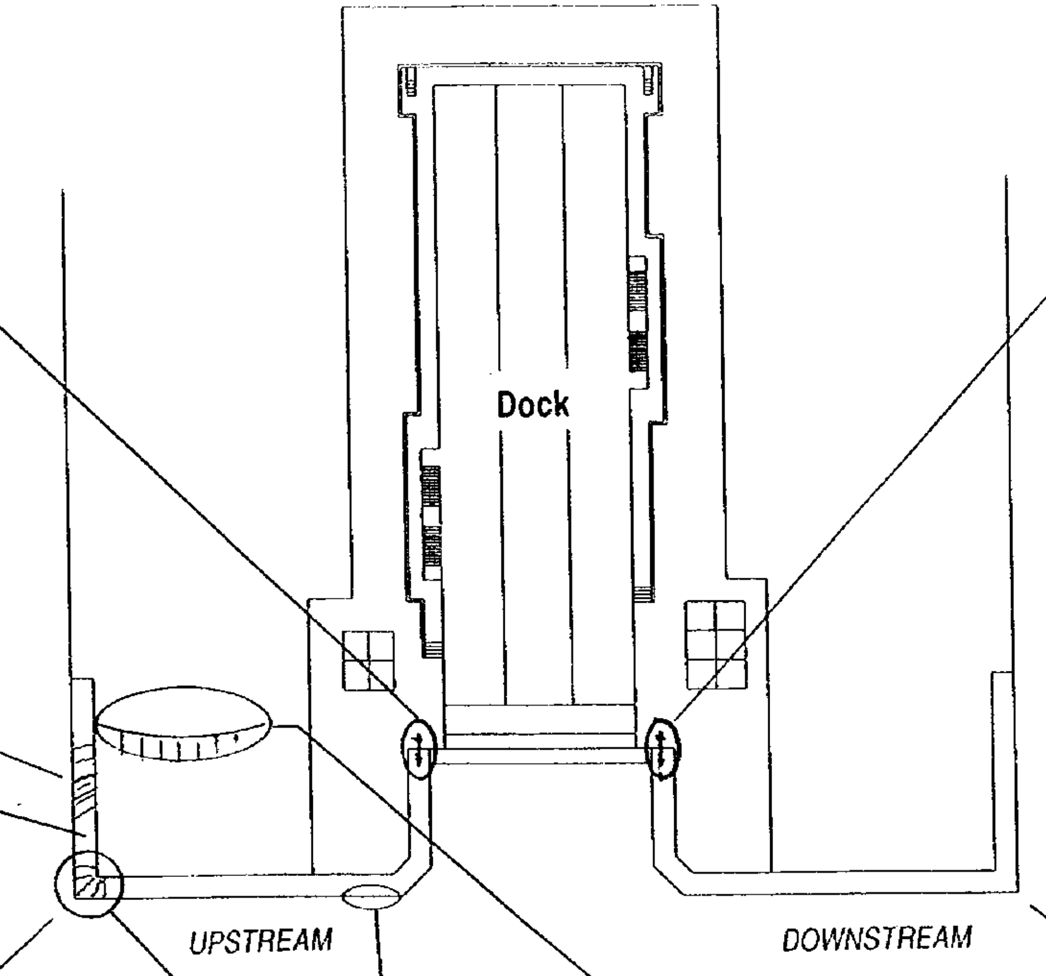


Photo - 8
Superstructure concrete of Steel Sheet Pile Wall at upstream wing
Crack developed at center part.

Photo - 7
Superstructure concrete of Steel Sheet Pile Wall at upstream wing
Distorted at center part.

Photo - 6
View of the Dock from Upstream
Heavily eroded part is observed.

Photo - 5
Collapsed superstructure concrete of Steel Sheet Pile Wall
(at corner part of upstream wing)
Upstream (UP-R) North

Photo -4
Measuring Inclination of Steel Sheet Piles

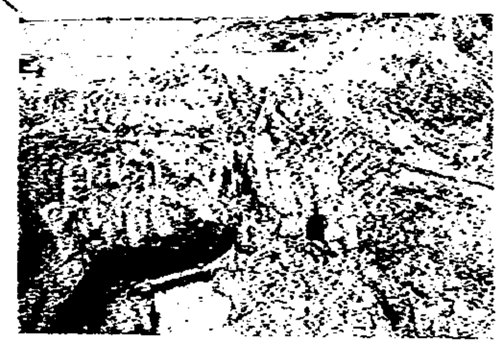


Photo -3
Anchoring Steel Sheet Pile Wall
(at upstream wing)

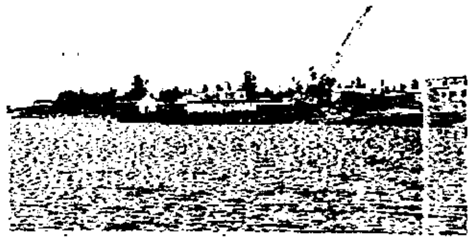


Photo - 2
View of the Dock from Downstream
Heavily eroded part is observed.

Yearly Erosion Rate of River Bank

1. Erosion Rate for Various Isobaths (from Sept.1992 to December 1997):

Survey Line No	1	2	3	4	5	6		19	20	21	22	23	24	No Obs	Total	Average
+2.0 Isobath																
Sept. 1992 Measurements	+4.2	+4.8	+4.8	+4.4	+3.8	+2.2	....	-2.8	-2.4	-2.8	-4.5	+1.0	+5.0	12	+17.7	+1.5
July, 1997 Measurements	+22.6	+21.2	+18.0	+19.2	+17.2	+17.6	....	+5.8	+6.0	-	-	-	+1.8	9	+129.4	+14.4
Dec. 1997 Maesurement	+23.3	+22.0	+18.0	+16.8	+17.3	+18.0		+8.5	+7.5	+7.0	+6.2	+6.4	+12.0	12	+163.0	+13.6
Dif.=(Sept.1992)-(Jul.1997)	-18.4	-16.4	-13.2	-14.8	-13.4	-15.4	....	-8.6	-8.4				+3.2			-12.9
Dif.=(Sept.1992)-(Dec.1997)	-19.1	-17.2	-13.2	-12.4	-13.5	-15.8	....	-11.3	-9.9	-9.8	-10.7	-5.4	-7.0			-12.1
+0.0 Isobath																
Sept. 1992 Measurements	-1.8	-1.4	-0.8	-1.0	-4.4	-5.8	....	-11.6	-11.2	-11.4	-11.0	-17.0	-18.6	12	-96.0	-8.0
July, 1997 Measurements	+9.4	+10.0	+8.2	+5.6	+3.6	+5.6	....	-2.2	-4.8	-4.6	-5.0	-6.2	-8.8	12	+10.8	+0.9
Dec. 1997 Maesurement	+14.0	+10.0	+9.0	+7.0	+5.0	+7.5		+3.2	-1.5	-1.0	-4.0	-5.0	-4.5	12	+39.7	+3.3
Dif.=(Sept.1992)-(Jul.1997)	-11.2	-11.4	-9.0	-6.6	-8.0	-11.4	....	-9.4	-6.4	-6.8	-6.0	-10.8	-9.8			-8.9
Dif.=(Sept.1992)-(Dec.1997)	-15.8	-11.4	-9.8	-8.0	-9.4	-13.3	....	-14.8	-9.7	-10.4	-7.0	-12.0	-14.1			-11.3
-3.0 Isobath																
Sept. 1992 Measurements	-9.0	-7.2	-9.0	-9.6	-10.6	-11.4	....	-16.0	-16.2	-18.0	-14.8	-17.2	-20.4	12	-159.4	-13.3
July, 1997 Measurements	-3.4	+1.0	+1.8	+1.2	+1.6	+1.0	....	-6.0	-6.2	-7.0	-7.0	-7.0	-7.6	12	-37.6	-3.1
Dec. 1997 Maesurement	+4.5	+3.0	+2.0	+0.5	-2.4	+0.0		-1.5	-5.5	-6.0	-7.8	-8.0	-9.0	12	-30.2	-2.5
Dif.=(Sept.1992)-(Jul.1997)	-5.6	-8.2	-10.8	-10.8	-12.2	-12.4	....	-10.0	-10.0	-11.0	-7.8	-10.2	-12.8			-10.2
Dif.=(Sept.1992)-(Dec.1997)	-13.5	-10.2	-11.0	-10.1	-8.2	-11.4	....	-14.5	-10.7	-12.0	-7.0	-9.2	-11.4			-10.8
-5.0 Isobath																
Sept. 1992 Measurements	-13.0	-12.4	-14.0	-14.8	-15.4	-17.2	....	-19.2	-20.4	-20.6	-17.2	-25.0	-27.0	12	-216.2	-18.0
July, 1997 Measurements	-0.4	-2.8	-3.8	-6.0	-7.0	-6.6	....	-7.8	-9.8	-9.6	-10.0	-11.8	-10.0	12	-85.6	-7.1
Dec. 1997 Maesurement	+0.0	-2.4	-3.8	-6.0	-8.0	-5.0		-4.5	-8.0	-9.8	-10.0	-10.8	-12.0	12	-80.3	-6.7
Dif.=(Sept.1992)-(Jul.1997)	-12.6	-9.6	-10.2	-8.8	-8.4	-10.6	....	-11.4	-10.6	-11.0	-7.2	-13.2	-17.0			-10.9
Dif.=(Sept.1992)-(Dec.1997)	-13.0	-10.0	-10.2	-8.8	-7.4	-12.2	....	-14.7	-12.4	-10.8	-7.2	-14.2	-15.0			-11.3
-7.0 Isobath																
Sept. 1992 Measurements	-22.0	-22.6	-22.0	-22.4	-23.2	-21.6	....	-25.6	-28.6	-27.0	-27.0	-29.2	-30.0	12	-301.2	-25.1
July, 1997 Measurements	-7.0	-9.0	-12.2	-13.4	-15.0	-14.0	....	-17.6	-13.8	-14.2	-14.8	-13.0	-18.0	12	-168.0	-14.0
Dec. 1997 Maesurement	-7.0	-9.0	-10.5	-13.5	-15.5	-15.0		-9.8	-13.0	-15.0	-14.4	-15.2	-17.2	12	-155.1	-12.9
Dif.=(Sept.1992)-(Jul.1997)	-15.0	-13.6	-9.8	-9.0	-8.2	-7.6	....	-8.0	-14.8	-12.8	-12.2	-10.2	-12.0			-11.1
Dif.=(Sept.1992)-(Dec.1997)	-15.0	-13.6	-11.5	-8.9	-7.7	-6.6	....	-15.8	-15.6	-12.0	-12.6	-14.0	-12.8			-12.2
-10.0 Isobath																
Sept. 1992 Measurements	-43.8	-41.0	-41.0	-39.6	-36.2	-39.8	....	-44.4	-46.4	-38.8	-37.2	-35.8	-41.8	12	-485.8	-40.5
July, 1997 Measurements	-30.0	-29.4	-31.2	-34.0	-33.4	-32.0	....	-32.0	-29.6	-23.0	-23.6	-26.0	-25.4	12	-349.6	-29.1
Dec. 1997 Maesurement	-26.4	-29.5	-31.0	-30.4	-32.8	-31.5		-30.5	-27.4	-25.0	-23.8	-24.0	-25.5	12	-337.8	-28.2
Dif.=(Sept.1992)-(Jul.1997)	-13.8	-11.6	-9.8	-5.6	-2.8	-7.8	....	-12.4	-16.8	-15.8	-13.6	-9.8	-16.4			-11.4
Dif.=(Sept.1992)-(Dec.1997)	-17.4	-11.5	-10.0	-9.2	-3.4	-8.3	....	-13.9	-13.0	-13.8	-13.4	-11.8	-16.3			-12.3

2. Summary:

Sep-92	Day	Year
Jul-97	1,764	4.8
Dec-97	1,917	5.3

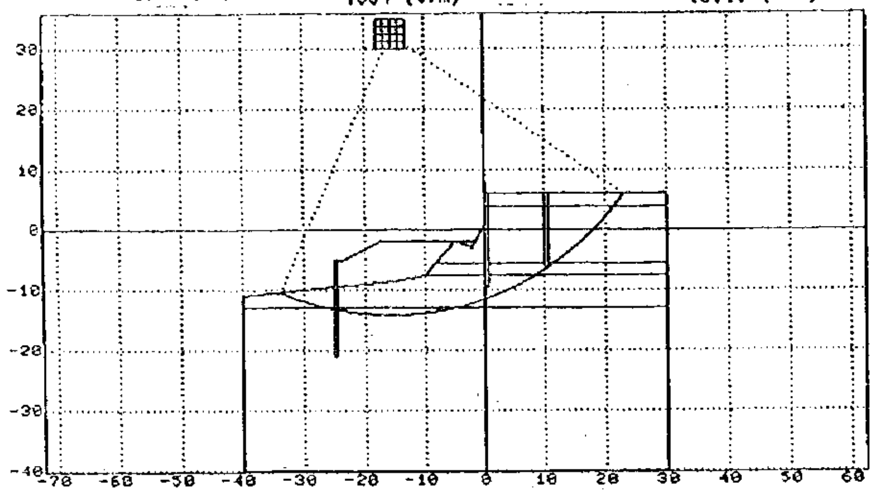
	+2.0	+0.0	-3.0	-5.0	-7.0	-10.0	Average	Duration	Yearly Erosion Rate
Dif.=(Sep.'92)-(Jul.'97)	-12.9	-8.9	-10.2	-10.9	-11.1	-11.4	-10.90	4.8	-2.27
Dif.=(Sep.'92)-(Dec.'97)	-12.1	-11.3	-10.8	-11.3	-12.2	-12.3	-11.67	5.3	-2.20

3. Result:

Thus yearly erosion rate has been set at : **2.3 m.**

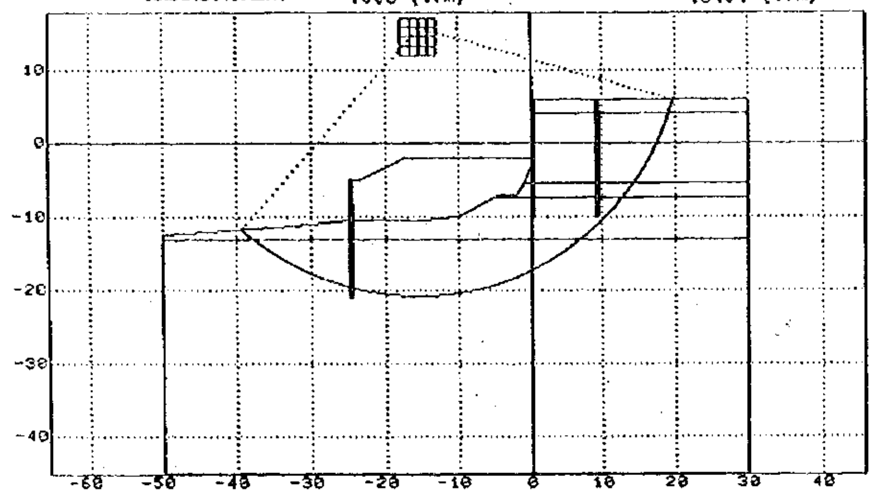
Section at ①

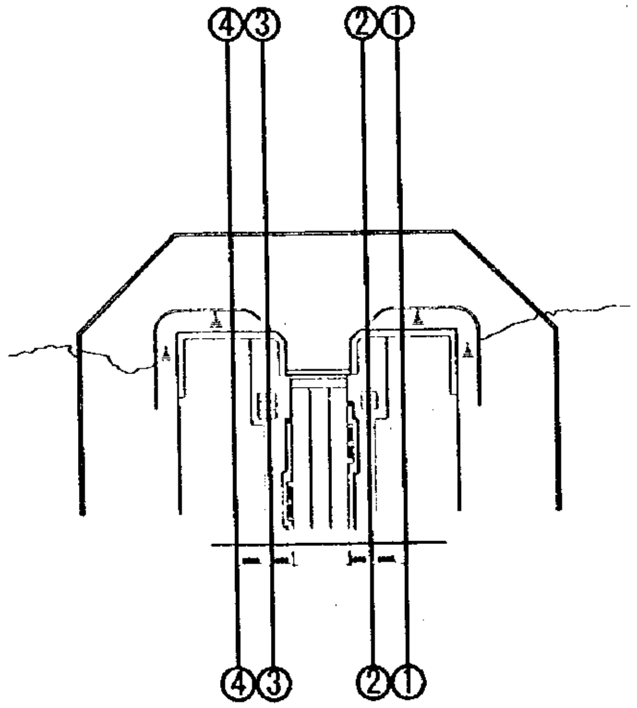
CENTER (-15.500 , 32.500) RADIUS 46.500 (m) SAFETY FACTOR 1.422
SLIDING MOMENT 7394 (tfm) RESISTANCE MOMENT 10518 (tfm)



Section at ③

CENTER (-15.500 , 15.750) RADIUS 36.500 (m) SAFETY FACTOR 1.334
SLIDING MOMENT 7860 (tfm) RESISTANCE MOMENT 10484 (tfm)

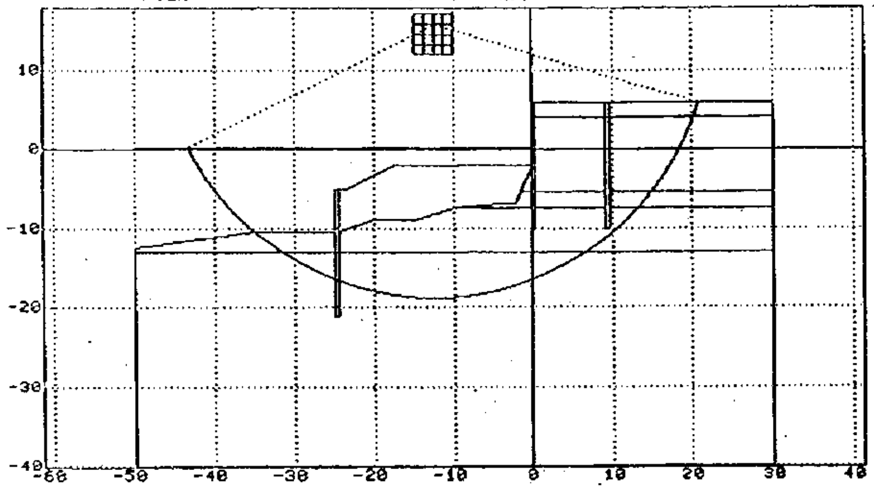




Location of Sections

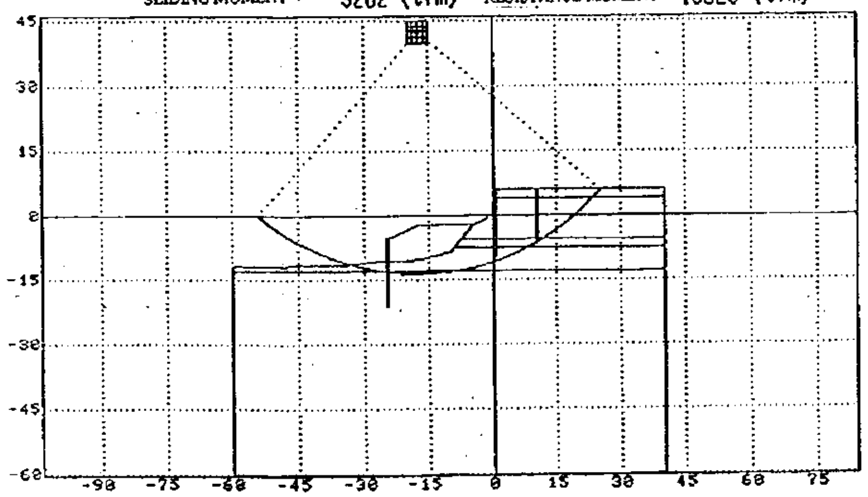
Section at ②

CENTER (-12.500 , 15.750) RADIUS 34.500 (m) SAFETY FACTOR 1.361
SLIDING MOMENT 6545 (tfm) RESISTANCE MOMENT 8909 (tfm)



Section at ④

CENTER (-17.500 , 42.500) RADIUS 58.100 (m) SAFETY FACTOR 1.416
SLIDING MOMENT 9202 (tfm) RESISTANCE MOMENT 13028 (tfm)



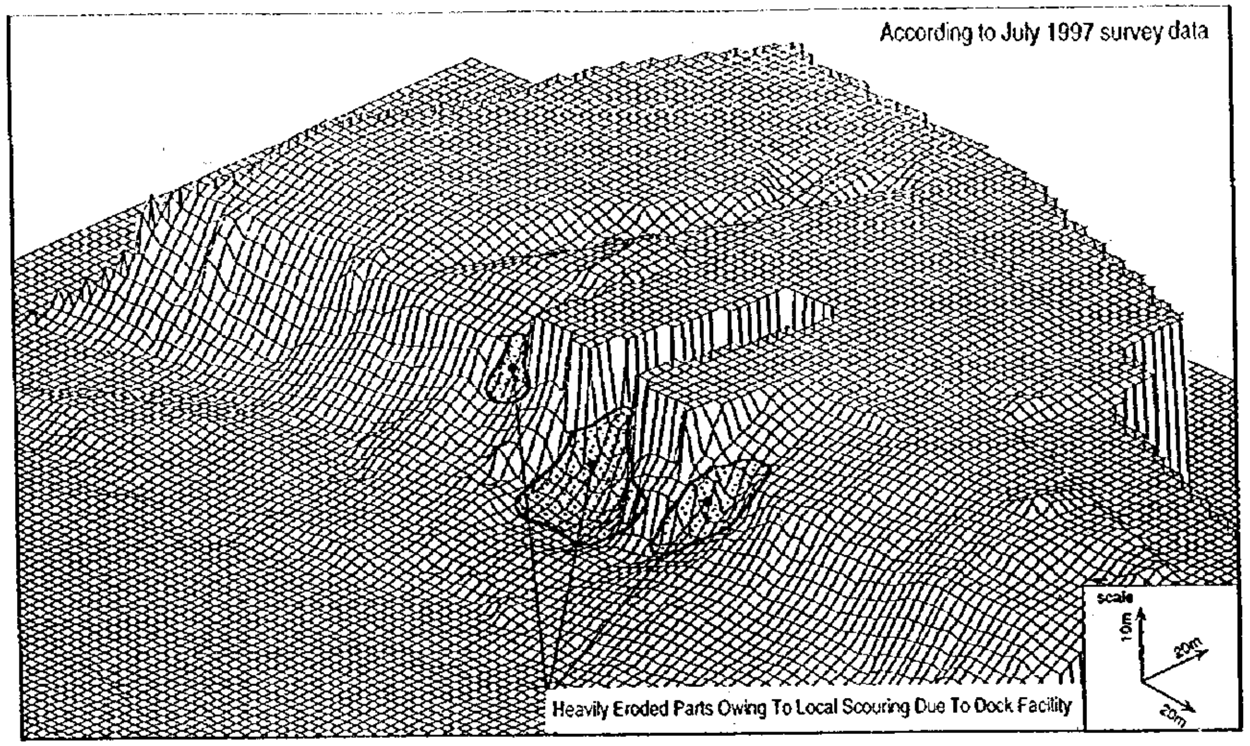


Fig.-1 Heavily Eroded Parts Owing To Local Scouring Due To Dock Facility
(According to July 1997 survey data)

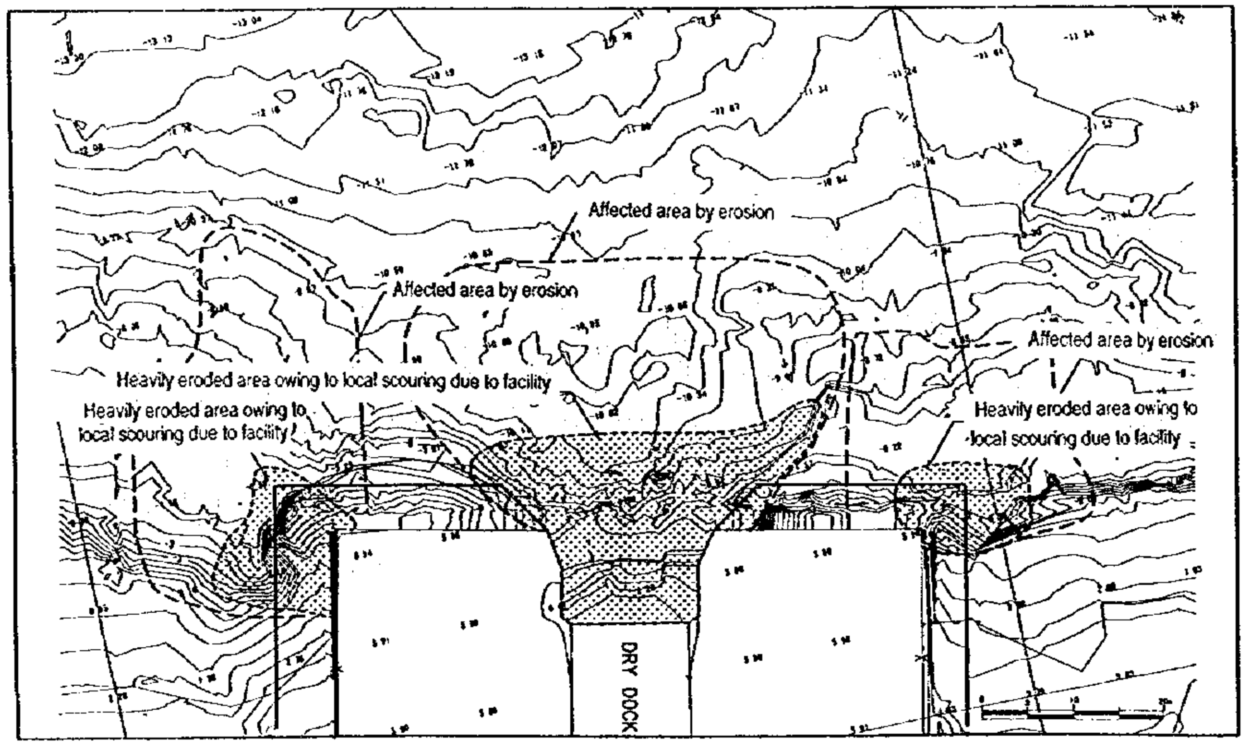


Fig.-2 Local Scouring Areas Due To Dock Facility
(According to July 1997 survey data)

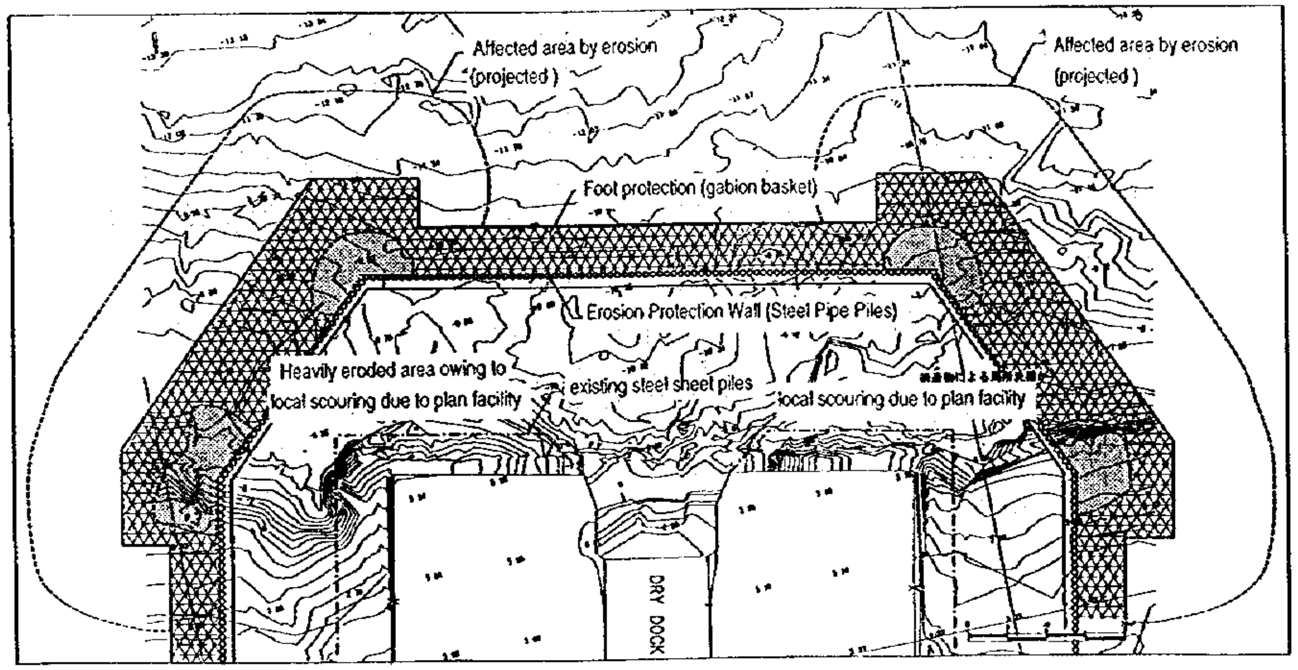
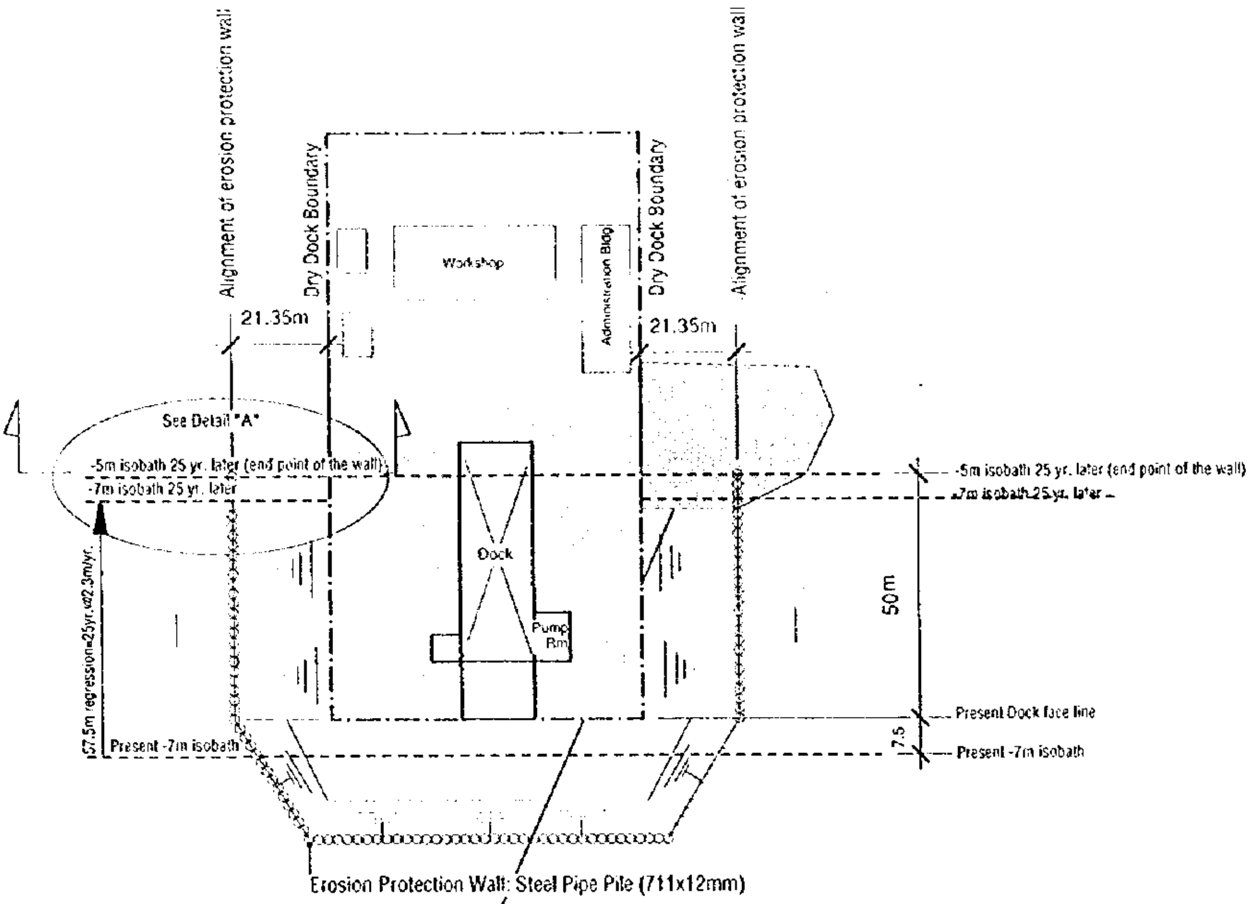
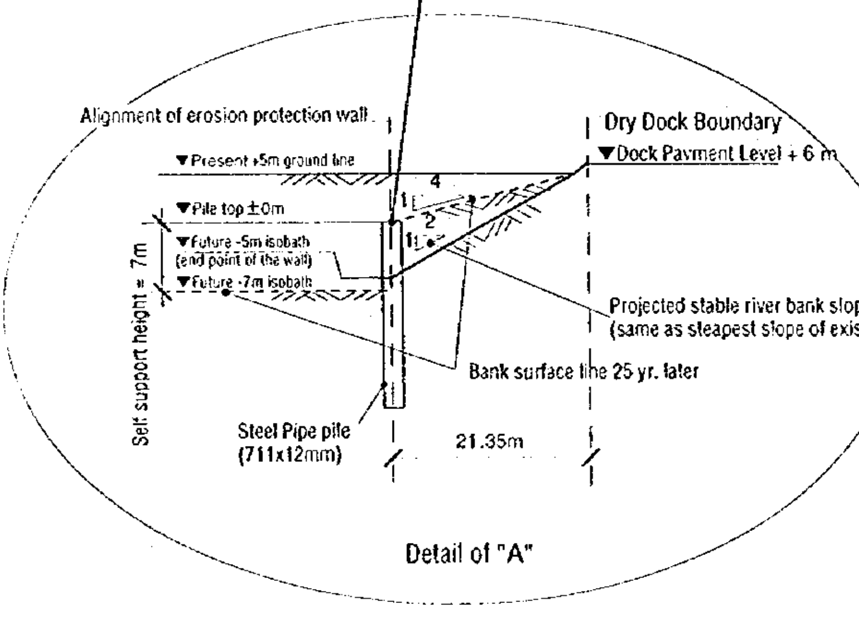


Fig.-3 Projected Local Scouring Areas Due To Plan Facility

APPENDIX−8.
Assessment of Cover Range of the Erosion Protection Wall



Counter Measures To Be Taken (25 years later):
(1) To place bank erosion protection materials (such as gabions or sand bags)
(2) To drive piles lower upto stable level necessary for self supporting.
(However, Care shall be given for keeping stable slope within the protection wall)



River bank isobath and Existing Stable Slope

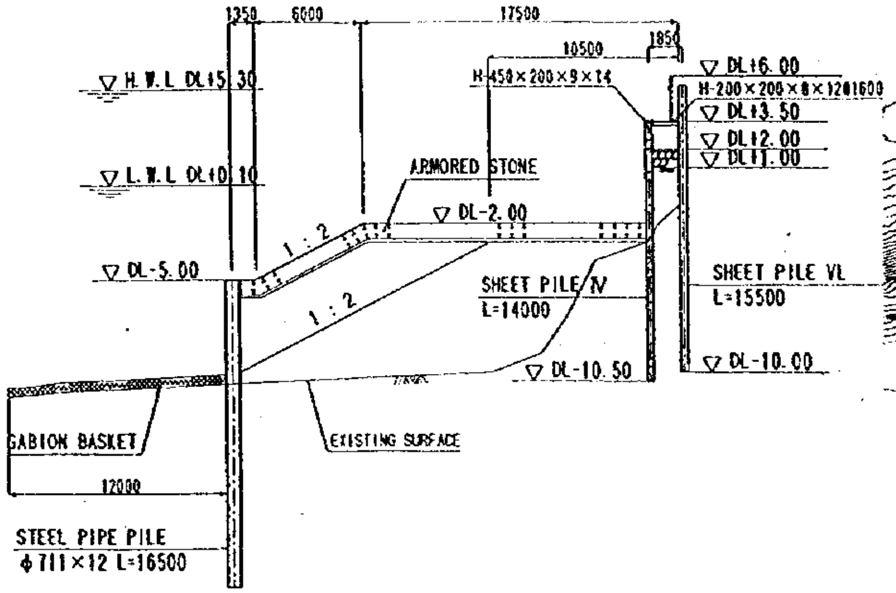
～ ±0m	1 : 5
±0m ～ −7m	1 : 2
−7m ～ −13m	1 : 7
−13m ～	1 : 30

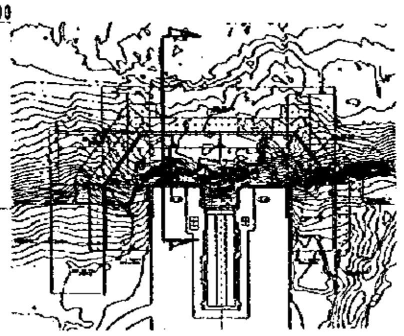
1. Basic Concept

The basic erosion protection measures are to restore the eroded riverbed to a level that provides structural stability to the revetment by way of constructing a mound, and to install self-supporting steel pipe piles at the toe of the mound in order to protect the mound against erosion and to reduce volume of filling sand.

2. Analyzed Section

A typical section of the erosion protection structures is shown in the following figure.





Analyzed Section

3. Design Conditions

(1) Tidal Levels

Highest High Water Level	D. L. +5.30 m
Lowest Low Water Level	D. L. +0.10 m
Residual Water Level	D. L. +4.00 m

(2) Seismic Conditions — Seismic force is not taken into consideration

(3) Corrosion Margin — 25 years of corrosion margin applied to steel members

(4) Allowable Stress of Steel Pile — $\sigma sa = 1{,}400 kgf/cm^2$

(5) Soil Conditions

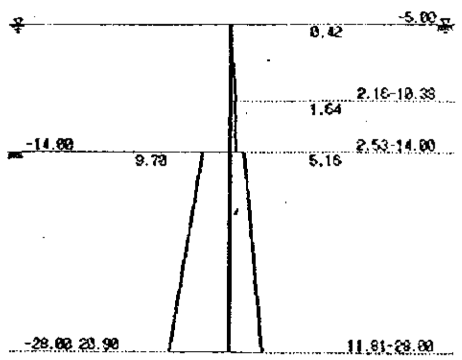
D.L.+6.0m～+2.5m	Filled Soil	$\gamma = 1.8 tf/m^3$	$\gamma' = 0.9 tf/m^3$	$\phi = 30°$
D.L.+2.5m～-5.5m	Cohesive Soil	$\gamma = 1.6 tf/m^3$	$\gamma' = 0.6 tf/m^3$	$c = 0.7 tf/m^2$
D.L.-5.5m～-7.5m	Sandy Soil	$\gamma = 1.8 tf/m^3$	$\gamma' = 1.0 tf/m^3$	$\phi = 28°$
D.L.-7.5m～-14.0m	Sandy Soil	$\gamma = 1.8 tf/m^3$	$\gamma' = 1.0 tf/m^3$	$\phi = 34°$
D.L.-14.0m～-28.0m	Cohesive Soil	$\gamma = 1.8 tf/m^3$	$\gamma' = 1.0 tf/m^3$	$c = 4.85 tf/m^2$ N=20
Soil for filling		$\gamma = 1.8 tf/m^3$	$\gamma' = 1.0 tf/m^3$	$\phi = 27.5°$ N=5

4. Design of Self-Supporting Steel Pipe Pile Structure

(1) Flow Chart of Design

Calculation of earth pressure	Coulomb's earth pressure theory
↓ Setting of virtual riverbed level	The point that sum of active earth pressure and residual water pressure equals to passive earth pressure
↓ Calculation of working load and point	Resultant of active earth pressure derived from the virtual riverbed level
↓ Calculation of stress of pile	Based on Chang's formula (as an exposed pile from surface)
↓ Calculation of embedded length of pile	Based on Chang's formula. Embedded length = 3/β, where β is a specific coefficient
↓ Computation of deformation at pile head	Allowable deformation is within 10 cm

(2) Distribution Diagram of Working Earth Pressure



(3) Results of Design Analysis

Item	Ordinary Condition		
Calculation Basis	ϕ600 x 14t	ϕ700 x 12t	ϕ711 x 12t
Section area A (per meter of wall)	$392.8 cm^2/m$	$348.3 cm^2/m$	$348.4 cm^2/m$
Moment of area (before corrosion)	$141{,}917 cm^4/m$	$174{,}444 cm^4/m$	$180{,}808 cm^4/m$
Moment of area (after corrosion)	$136{,}493 cm^4/m$	$166{,}807 cm^4/m$	$172{,}891 cm^4/m$
Section modulus Z (per meter of wall)	$4{,}557 cm^3/m$	$4{,}773 cm^3/m$	$4{,}869 cm^3/m$
Virtual riverbed level (m)	-14.00		
Pile head level (m)	-5.00		
Pile width B(cm)	100		
Stiffness against bend EI(kgf/cm^2) Before corrosion (for calculation of embedded length) After corrosion (for calculation of section area)	2.980×10^{11} 2.866×10^{11}	3.663×10^{11} 3.503×10^{11}	3.797×10^{11} 3.631×10^{11}
Working point of horizontal force (m)	3.61		
Pile head condition	free		
Horizontal force (tf)	14.49		
Constant of lateral resistance of ground Kh(kgf/cm^3)	4.00		
Specific coefficient $\beta = (Kh/B/4EI)1/4 (cm^{-1})$ Before corrosion (for calculation of embedded length) After corrosion (for calculation of area)	4.280×10^{3} 4.322×10^{3}	4.065×10^{3} 4.110×10^{3}	4.029×10^{3} 4.074×10^{3}
Calculation Results			
Surface deformation (cm)	0.80	0.74	0.73
Deflection angle of surface (rad)	0.00558	0.00486	0.00474
Maximum moment of bend underground (tf·m)	56.02	56.34	56.40
First zero point of moment of bend lm_1 (m)	6.00	6.33	6.39

(4) Summary of Calculation Results

Steel Pipe Pile	ϕ600 x 14t	ϕ700 x 12t	ϕ711 x 12t
Stress of steel member $\sigma s = M/Z$	$56.02 \times 10^5/4{,}557 = 1{,}229 kgf/cm^2 < \sigma_{sa} = 1{,}400 kgf/cm^2$	$56.34 \times 10^5/4{,}773 = 1{,}180 kgf/cm^2 < \sigma_{sa} = 1{,}400 kgf/cm^2$	$56.40 \times 10^5/4{,}869 = 1{,}158 kgf/cm^2 < \sigma_{sa} = 1{,}400 kgf/cm^2$
Embedded length of pile $L = 3/\beta$ *	3/0.4280 = 7.01m, -14m(riverbed level) - 7.01m = -21.01m → -21.5m	3/0.4065 = 7.38m, -14m(riverbed level) - 7.38m = -21.38m → -21.5m	3/0.4029 = 7.45m, -14m(riverbed level) - 7.45m = -21.45m → -21.5m
Deformation of pile head	δ=9.20cm<10.0cm	δ=7.80cm<10.0cm	δ=7.20cm<10.0cm
Pile weight	5.247t/m	4.670t/m	4.215t/m
Number of pile	1.282pc/m	1.136pc/m	1.122pc/m

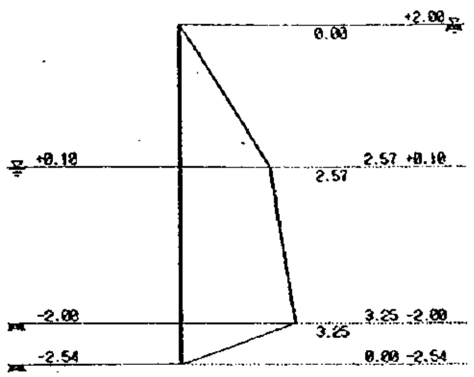
* Embedded length has been calculated with β value before corrosion, which is resulted in longer embedded length due to greater stiffness against bend.

5. Design of Front Revetment

(1) Flow Chart of Design

Calculation of earth pressure	Coulomb's earth pressure theory
↓ Setting of virtual riverbed level	The point that sum of active earth pressure and residual water pressure equals to passive earth pressure
↓ Calculation of working load and point	Resultant of active earth pressure derived from the virtual riverbed level
↓ Calculation of stress of pile	Based on Chang's formula (as an exposed pile from surface)
↓ Calculation of embedded length of pile	Based on Chang's formula Embedded length = 3/β, where β is a specific coefficient
↓ Computation of deformation at pile head	Allowable deformation is within 10 cm

(2) Distribution Diagram of Working Earth Pressure



(3) Results of Design Analysis

Item	Normal Condition
Calculation Basis	Steel Sheet Pile U-IV (SY295)
Section area A (per meter of wall)	242.5cm²/m
Moment of area (before corrosion)	36,091cm⁴/m
Section modulus Z (per meter of wall)	2,122cm³/m
Virtual riverbed level (m)	-2.54
Pile head level (m)	+2.00
Pile width B(cm)	100
Stiffness against bend EI(kgf/cm²) Before corrosion (for calculation of embedded length)	7.579×10^{10}
Working point of horizontal force (m)	1.88
Pile head condition	Free
Horizontal force (tf)	9.43
Constant of lateral resistance of ground Kh(kgf/cm²)	1.00
Specific coefficient β =(Kh/B/4EI)1/4(cm⁻¹) Before corrosion (for calculation of embedded length)	4.262×10^{-3}
Calculation Results	
Surface deformation (cm)	1.45
Deflection angle of surface (rad)	0.00891
Maximum moment of bend underground (tf·m)	21.37
First zero point of moment of bend lm_1 (m)	6.39

(4) Summary of Calculation Results

Steel Sheet Pile	Type U-IV (SY295)
Stress of steel member $\sigma s = M/Z$	$21.37 \times 10^5/2,122 = 1,007 kgf/cm^2 < \sigma_{sa} = 1,400 kgf/cm^2$
Embedded length of pile $L=3/\beta$ *	3/0.4262 = 7.03m, -2.54m(riverbed level) - 7.03m = -9.57m →-10.0m
Deformation of pile head	δ =6.50cm<10.0cm

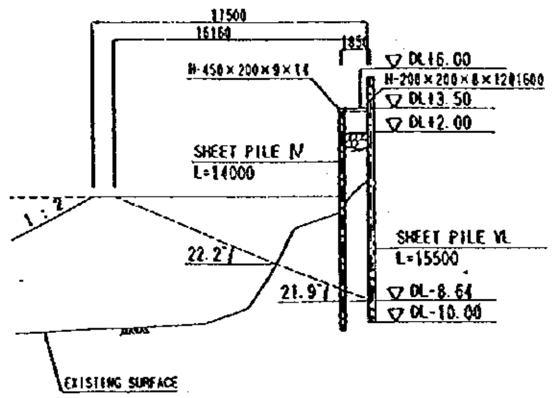
6. Design of Mound

The area of mound embankment should be extended to produce enough passive earth pressure to the existing revetment. The mound width has been determined by the cross point of failure plane of passive earth and the extension of crown level of the mound, or C.D.L -2.0m. The surface of the mound will be covered with armouring stones to protect from wash out of filled sand.

(1) Calculated Section

The typical section of the mound is shown in the following figure. From this results, the mound width of armouring stones has been set at 17.5m.

Calculation of the failure plane of passive earth is based on "Technical Standards for Port and Harbour Facilities in Japan (1991)" and following constants has been used.



Soil constants	Symbol	Bottom Soil (°)	Filled Sand (°)
Internal friction angle	φ	28	27.5
Angle of wall friction	δ	-14	-13.8
Angle of wall surface to vertical	ψ	0	0
Angle of ground surface to horizontal	β	0	0

$$\cot\zeta_p = \tan(\phi-\delta) + \sec(\phi-\delta) \times \left\{\frac{\cos\delta \cdot \sin(\phi-\delta)}{\sin\phi}\right\}^{1/2}$$

In case of bottom soil ($\phi=28°$, $\delta=\phi/2=14°$)

$$\cot\zeta p = \tan42° + \sec42° \times \left\{\frac{\cos14° \cdot \sin42°}{\sin28°}\right\}^{1/2}$$

$$=0.9004 + 1.3456 \times 1.1760 = 2.4828$$

$$\cot\zeta_p = \tan^{-1}(1/2.4828) = 21.94°$$

In case of filled sand ($\phi=27.5°$, $\delta=\phi/2=13.8°$)

$$\cot\zeta p = \tan41.3° + \sec41.3° \times \left\{\frac{\cos13.8° \cdot \sin41.3°}{\sin27.5°}\right\}^{1/2}$$

$$=0.8785 + 1.3311 \times 1.1782 = 2.4468$$

$$\cot\zeta_p = \tan^{-1}(1/2.4468) = 22.23°$$

(2) Examination of Armor Stones

Design conditions :

River velocity: V=1.5m/sec.

Gradient: 1:2

The required weight of the armor stones will be determined by the following formula proposed by Erosion Protection Board, US Coast Guard.

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

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

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

dg = 0.11 - 0.21m/piece or greater→ 25cm or greater (15,625cm³ or greater when converted to a cube)

Thus the stone weight comes to; w = 15,625cm³ x 2.65 = 41.4kg → 50kg/piece or more.

JICA