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GOVERNMENT OF PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANIZATION

FEDERAL REPUBLIC OF GERMANY

KREDITANSTALT FÜR
WIEDERAUFBAU (KfW)

FRENCH REPUBLIC

AGENCE FRANÇAISE DE
DÉVELOPPEMENT (AFD)



7

BANK PROTECTION AND
RIVER TRAINING (AFPM)
PILOT PROJECT
FAP 21/22

PR-710
A-85.7(2)

FINAL PROJECT
EVALUATION REPORT



VOLUME III

Annex 4: The Groyne Test Structure
Design Report

Annex 5: The Groyne Test Structure;
Procurement and
Construction Report

DECEMBER 2001



JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE
CONSULTING CONSORTIUM : AP 21/22

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TECHNOLOGICAL SERVICES LTD. (BETS)
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**BANK PROTECTION AND RIVER TRAINING (AFPM)
PILOT PROJECT FAP 21/22**

FINAL PROJECT EVALUATION REPORT

VOLUME III

- Annex 4: The Groyne Test Structure; Design Report**
**Annex 5: The Groyne Test Structure; Procurement and
Construction Report**

MAY 2001

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BANK PROTECTION PILOT PROJECT
FAP 21

FINAL PROJECT EVALUATION REPORT

ANNEX 4

**THE GROUYNE TEST STRUCTURE;
DESIGN REPORT**

MAY 2001

FAP 21 – BANK PROTECTION PILOT PROJECT

FINAL PROJECT EVALUATION REPORT

ANNEX 4

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- Attachment 4: Selected Construction Drawings as a Base for Future Standard Designs

LIST OF ACRONYMS

ASTM	-	American Society for Testing Materials
BRTS	-	Brahmaputra River Training Structures
BS	-	British Standards
BWDB	-	Bangladesh Water Development Board
CC	-	Cement Concrete
CNR	-	Compagnie Nationale du Rhône
DHW	-	Design High Water Level
DIN	-	Deutsche Industrie Norm (i.e. German Industrial Standard)
EAU	-	Empfehlungen des Arbeitsausschusses "Ufereinfassungen" (Recommendations of the Committee for Waterfront Structures)
FAP	-	Flood Action Plan
FPCO	-	Flood Plan Coordination Organization
HFV	-	High Flood Water Level
IALA	-	International Association of Lighthouse Authorities
JTWC	-	Jamuna Test Works Consultants
LC	-	Loading Class
PIANC	-	Permanent International Association of Navigation Congresses
PWD	-	Public Works Department (datum level)
RRI	-	River Research Institute
SHW	-	Standard High Water
SLW	-	Standard Low Water
SPT	-	Standard Penetration Test
SWMC	-	Surface Water Modelling Centre
WARPO	-	Water Resources Planning Organization



GLOSSARY

TERM	DEFINITION
bed protection	layered systems placed on filters on a horizontal surface as protection against hydraulic forces and scouring
char	island, sand bank and floodable area adjacent to the banks
cover layer	outer protective layer of an embankment revetment or a bed protection
falling apron	multi-layer system of granular material placed directly on the existent subsoil or riverbed
filter	one-layer or multi layer system of well graded granular material or a geotextile or a combination of both
gabions	mattresses and rectangular baskets made from protected steel wire mesh and filled with loose material such as boulders, bricks etc.
geotextile	synthetic fabric (woven, non-woven, needle pinched) applied as a filter or used in tailored geotextile systems (mattresses, etc.)
hydraulic loads	forces due to action of water (hydrostatic or hydrodynamic)
launching apron	integrated and articulating mattress system placed on prepared slopes above and below water or in horizontal excavation well above SLW
revetment	layered systems placed on filters on a sloping surface as protection against hydraulic forces and scouring
rip-rap	layer of loose stones acting as cover layer in an embankment revetment, a bed protection or a falling apron
rock	any hard natural or artificial material requiring the use of blasting or mechanical tools for its removal
scour	removal of soil particles by current or wave induced shear forces
seepage	movement of water into or out of the river bank
subsoil	naturally deposited or filled and compacted soil material on which an embankment revetment, a bed protection or a falling apron is constructed

TERM	DEFINITION
suitable material	all material obtained from excavations within the site or from borrow pits and which is approved by the employer as acceptable for use in the works
toe protection	systems to protect the toe of an embankment against instability due to erosion/scouring
unsuitable material	any other than "suitable material"

LIST OF SYMBOLS

CC	-	cement concrete	(-)
\bar{C}_D	-	drag coefficient taking into account the resistance of the pile against the flow pressure	(-)
C_M	-	inertia coefficient taking into account the resistance of the pile against the acceleration of water particles	(-)
C_u	-	coefficient of uniformity	(-)
c	-	cohesion	(kN/m ²)
c'	-	effective cohesion	(kN/m ²)
D	-	dimension of cube	(m)
D	-	outer diameter (of piles)	(mm)
D	-	compactness (of non-cohesive soils)	(-)
D	-	specific size or thickness of protection unit	(m)
D_n	-	grain size diameter corresponding to n % by mass of finer particles	(mm)
D_n	-	nominal thickness of protection unit	(m)
D_{n50}	-	nominal diameter of rip-rap	(m)
D_{10}	-	effective grain size	(mm)
d	-	thickness of mattress	(m)
d	-	water depth	(m)
d_b	-	thickness of cover layer of open stone asphalt	(m)
d_{sm}	-	design depth of scour hole	(m)
d_{n50}	-	grain diameter	(m)
e	-	distance between pile axes	(m)
e	-	base of the natural logarithm	(1/m)
f_y	-	yield stress (steel)	(N/mm ²)
g	-	acceleration due to gravity	(m/s ²)
H	-	layer thickness	(m)
H	-	wave height	(m)
H	-	height of the slope	(m)
H_s	-	significant wave height	(m)
h	-	water depth	(m)
K_h	-	depth factor	(-)
K_s	-	slope factor	(-)
K_r	-	turbulence factor	(-)
k	-	reduction factor	(-)
k	-	wave number	(1/m)
k'	-	slope reduction factor	(-)
k_g	-	permeability of geotextile	(m/s)
k_s	-	permeability of soil	(m/s)
k_s	-	coefficient of roughness for the river bed	(-)
L	-	length of pile	(m)
L	-	wave length	(m)
L_o	-	wave length in deep water	(m)
l_E	-	embedment length (of piles)	(m)
p_D	-	pressure due to the water particle velocity caused by the flow resistance per unit length of pile	(kN/m)

p_M	- inertial pressure due to instationary wave movement per unit length of pile	(kN/m)
T	- wave period	(s)
t	- wall thickness (of steel piles)	(mm)
t	- time duration	(s)
t_g	- geotextile thickness	(m)
t_m	- thickness of mattress	(m)
U	- circumference (of piles)	(cm)
u	- mean velocity	(m/s)
u	- horizontal component of the orbital velocity of the water particles	(m/s)
u_b	- bottom velocity	(m/s)
\bar{u}	- depth averaged flow velocity	(m/s)
u_{cr}	- critical velocity	(m/s)
u_r	- maximum current velocity	(m/s)
u_s	- surface velocity	(m/s)
V	- permeability of groynes	(-)
v	- flow velocity	(m/s)
W	- weight	(kN/m)
W	- driving resistance of pile	(kN)
W	- weight of armour unit	(tons)
W_e	- estimated driving resistance of piles	(kN)
α	- slope angle of revetment	(degree)
β	- strength coefficient	(-)
β	- stability coefficient	(-)
β	- angle of potential failure plane	(degree)
γ	- specific weight of solid	(kN/m ³)
γ_w	- specific weight of water	(kN/m ³)
γ'	- specific weight of solid in submerged condition	(kN/m ³)
δ	- shape coefficient for round piles	(m)
ΔH	- head loss	(m)
Δm	- relative density	(-)
Δp	- increase in pressure	(kN/m ²)
ϵ_s	- angle of internal friction of material	(degree)
ϕ	- stability factor	(-)
ϕ	- angle of slope	(degree)
ϕ'	- effective angle of internal friction	(degree)
λ	- coefficient	(-)
ξ_z	- wave breaking parameter	(-)
ρ_c	- specific gravity of concrete	(kg/m ³)
ρ_s	- density of protection material	(kg/m ³)
ρ_w	- density of water	(kg/m ³)
σ	- total normal stress	(kN/m ²)
σ_p	- prestress (in concrete piles)	(MN/m ²)
σ_{sf}	- pile tip resistance	(MN/m ²)
σ'	- effective normal stress	(kN/m ²)
τ	- shear strength	(kN/m ²)
τ_{mf}	- skin friction of piles	(kN/m ²)
θ	- angle of repose	(degree)

Ψ_{cr}	-	critical shear stress parameter	(-)
Ψ_{cr}	-	shields parameter	(-)
ψ	-	stability upgrading factor	(-)

Other symbols are explained in the text at their utilisation.

SUMMARY

The practical effects achieved with a submerged groyne versus standard high-water groynes are evaluated in Annex 7 to the Final Planning Study Report (Consulting Consortium FAP 21/22, 2000 h). For any future standard design the Guidelines (Consulting Consortium FAP 21/22, 2000 n) and the Design Manual (Consulting Consortium FAP 21/22, 2000 o) should be consulted.

As with any simplification design, assumptions tend to become somewhat conservative, which would offend the principle aim of the Project, namely to design economical structures with the least safety level. However, one should realise that structures to be implemented at a river of the magnitude and high mobility of the Jamuna have to match too many variables.

It should be noted again that for any future design of permeable groynes the Guidelines (Consulting Consortium FAP 21/22, 2000 n) and the Design Manual (Consulting Consortium FAP 21/22, 2000 o) must be consulted.

1 INTRODUCTION

1.1 AIM OF THE DESIGN REPORT

The aim of the Bank Protection Pilot Project is to develop and to optimise design criteria and cost-effective construction and maintenance methods, which could ultimately serve as standard bank protection works along the Jamuna and other main rivers of Bangladesh. It was a condition that any such methods should take into consideration locally customary methods and materials and that the improved construction methods and strategies were to be employed and verified for their effectiveness and future applicability through the Pilot Project.

The Project comprised the construction of test structures at two selected test sites, namely at Kamarjani (1994/95) and Bahadurabad (1995/97). The implemented test structures included permeable and impermeable groynes, embankment revetments, bed protections, launching aprons and falling aprons. The functioning and efficiency of these test structures were monitored after completion for several years and suitable modifications were implemented as appropriate.

The present Design Report deals with the Groyne Test Structures at Kamarjani Test Site. Summarising and supplementing earlier reports, such as the Final Planning Study Report (Consulting Consortium FAP 21/22, 1993 a) and the Procurement and Construction Report, Test Site I, Kamarjani (Consulting Consortium FAP 21/22, 1994 a), it shall highlight the

- selection of test site location;
- layout configuration of the test site;
- structural design assumptions and criteria;
- configuration of the groyne structures;
- associated embankment and bed protection measures;
- proposed and applied adaptations to the original structures, and
- basic recommendations for future standard designs.

The following subjects are to be seen in conjunction with this Design Report and are covered elsewhere in the Final Project Evaluation Report, namely

- construction methods [Annex 5], (Consulting Consortium FAP 21/22, 2001 f);
- construction materials [Annex 5], (Consulting Consortium FAP 21/22, 2001 f);
- construction equipment [Annex 5], (Consulting Consortium FAP 21/22, 2001 f);
- construction implementation schedule [Annex 5], (Consulting Consortium FAP 21/22, 2001 f);
- financial and economic evaluation [Annex 5 and Annex 12], (Consulting Consortium FAP 21/22, 2001 f and m);
- evaluation of monitored hydraulic loads on the structures [Annex 7], (Consulting Consortium FAP 21/22, 2001 h), and
- response of the river [Annex 7], (Consulting Consortium FAP 21/22, 2001 h);

1.2 HISTORICAL EXPERIENCE

Groynes have been used in one or the other way and style since generations. They are also called spurs, spur dykes, transverse dykes, etc. and are commonly built from the bank right into the river. Commonly, groynes are being planned and built as impermeable structures, but there are many examples of permeable groynes as well. Experience has shown that impermeable groynes induce excessively deep local scour holes while permeable groynes initiate only very moderate ones. To be

more effective, groynes are arranged in series as a groyne field to cover larger stretches of a riverbank. Dependent on the configuration and direction of a groyne it may either repel or deflect the current or reduce the velocity of the river's flow. The effects range from protecting a bank line against erosion and initiating accretion to maintaining a navigable channel.

Some design rules have been developed for groyne structures over the decades, which however, cannot necessarily be transferred to the conditions of a braiding river such as the Jamuna. Even with the application of most modern computerised modelling and design tools, there will always remain some trial and error approach in preventing bank erosion at the unpredictable Jamuna river.

It must always be kept in mind, however, that whatever river training measure would be created, it presents a substantial interference with the natural flow of the river. The consequent river response will certainly call for compensatory measures in the following years. Therefore, the planning of adaptation and supplementary measures must always be seen as an inseparable element of the respective project.

1.3 ADVANTAGES OF THE USE OF PERMEABLE GROYNES

It was the focus of the Project to develop measures suitable to halt or at least to restrict bank erosion within selected areas along the Jamuna river. One of the technical options would be the protection of the river bank by revetments suitably designed to withstand the forces of the river, the other option to reduce the speed of the river flow through artificial structures to an extent that the natural and unprotected river banks are not or only marginally eroding.

It is the advantage of a bank protection structure consisting of a series of permeable groynes suitably arranged by direction, spacing and length that the

- flow attack on the natural riverbank within that area is being reduced to a magnitude not causing substantial erosion along the bank;
- development of scour (by depth and extent) is much less as compared to impermeable groynes;
- supplementary strong revetments along the main flood embankment are superfluous;
- materials can be ordered well in advance of implementation and kept in strategic stockyards;
- implementation offers maximum flexibility since the final decision on the most suitable layout configuration can be made in the very last minute, well in consideration of prevalent site condition;
- construction window can be maximised since installation of the groynes' piles can be carried out by floating equipment (using the pile installation equipment provided to WARPO/BWDB through the Project);
- adaptation measures in the subsequent seasons are easy to implement. A groyne field can be supplemented by intermediate and/or additional upstream/downstream groynes, extension of existing groynes towards the river, increasing the embedment length of the piles exposed to unexpected scouring, etc.

2 THE KAMARJANI TEST SITE

2.1 PRE-SELECTION AND VERIFICATION OF SITE LOCATION

Kamarjani area was pre-selected as potential test site area during the Study Phase 1992/93 (Consulting Consortium FAP 21/22, 1993 a) and is situated on the right bank of the Jamuna river at an approximate latitude of 25°20'N. In 1993 four major branches were distinguished here, namely the

- WK Branch (West Kamarjani Branch);
- WMK Branch (Western Middle Kamarjani Branch);
- EMK Branch (Eastern Middle Kamarjani Branch), and
- EK Branch (East Kamarjani Branch).

Upstream from these channels again four major branches existed in 1993, with the following designations:

- WT Branch (West Teesta Confluence Branch);
- WMT Branch (Western Middle Teesta Confluence Branch);
- EMT Branch (Eastern Middle Teesta Confluence Branch), and
- ET Branch (East Teesta Confluence Branch).

All the aforementioned branches are indicated in Fig. 2.1-1. The morphological developments of this area were monitored and based on the morphological predictions and the conclusive high certainty of erosion attack it was finally concluded in November 1993 to construct Test Site I with a series of permeable groynes immediately north of the mouth of the Ghagot River.

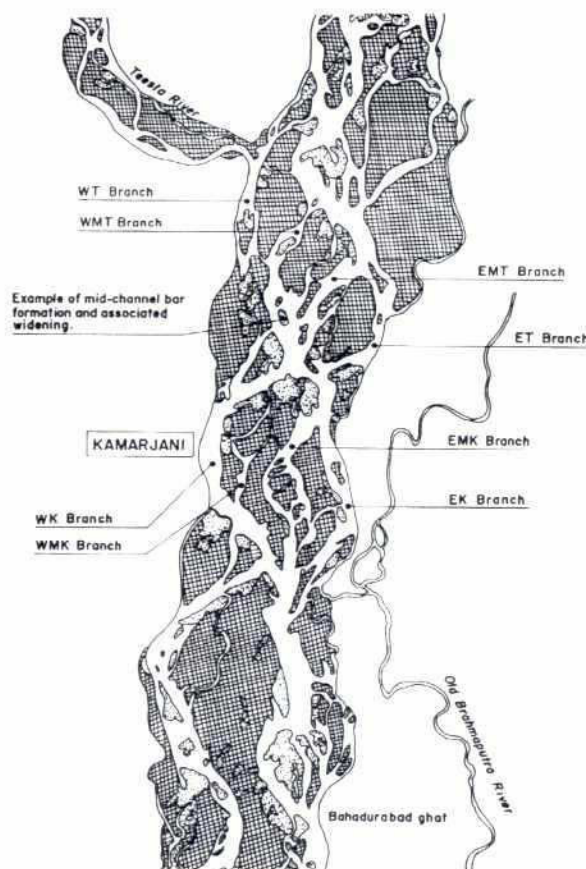


Fig. 2.1-1: Kamarjani Test Area (1993)



When deciding on the final test site location it was clearly pointed out that the erosion rates in the respective area were expected to be high, possibly calling for supplementary measures in the years after construction of the test structures (Consulting Consortium FAP 21/22, 1994 a).

After the monsoon 1994, the morphological development in the test site area was analysed and the suitability for implementation of Kamarjani Test Site confirmed (Consulting Consortium FAP 21/22, 1994 d,e). Subsequently the site for the Groyne Test Structure was finally agreed on between FPCO (now WARPO), the Donors and JTWC in September 1994. The attached drawing No. KA-002/1 presents the general layout of the implementation proposal (see Attachment 4).

2.2 PHYSICAL MODEL TESTS PRIOR TO IMPLEMENTATION

2.2.1 Model Tests at Faridpur

During the Study Phase of the Project physical model tests were carried out at the River Research Institute (RRI) in Faridpur, the results of which are compiled in Annex 14 to the Final Planning Study Report (Consulting Consortium FAP 21/22, 1993 k). The model investigations were conducted in two different models at a length scale of 1 : 75 and 1 : 60 respectively, namely the

- **Kamarjani Model**, and the
- **Bahadurabad Model**.

For the Groyne Test Structure the Kamarjani Model was to determine the following parameters:

- optimisation of the configuration and alignment of test structure;
- shape of the head of impermeable groynes;
- depth and location of scour in relation to the structures, and
- flow pattern in-between as well as along the groynes.

Based on the test results and recommendations issued in conclusion of the first ten model tests six supplementary model tests were carried out at RRI in the second half of 1993 after the start of the Test and Implementation Phase in order to verify the parameters for the layout at the finally selected test site of Kamarjani. The last model (T 16) was considered best suited for implementation and be conclusive for all related design parameters. The respective detailed analyses are presented in the Technical Report No. 1 of Phase II of the Project (Consulting Consortium FAP 21/22, 1994 c), but the resultant recommendations for the original design of the Groyne Test Structure are summarised in the following subsections.

It should be noted that after the first monitoring season, for the purpose of structure adaptation, another series of model tests was carried out at RRI, which are commented under Section 5.1.

2.2.2 Model Tests in France

With the start of the Test and Implementation Phase other supplementary physical model tests were carried out at the laboratories of CNR, Chanaz/France in 1993/94. These tests were aimed at to develop an understanding of the behaviour of falling aprons of concrete blocks placed around groyne structures as well as at the toe of revetment structures in order to restrict the development of scour holes. These models were scaled 1 in 15 and 1 in 25. The results are compiled in Technical Report No. 4 (Consulting Consortium FAP 21/22, 1995 b) and the developed design rules were taken into

consideration in the dimensioning of the bed protection by a falling apron around one of the main groynes (G-2) of the Groyne Test Structure.

2.3 LAYOUT CRITERIA FOR THE ORIGINAL TEST STRUCTURE

2.3.1 Preliminary Remarks

The groyne field test structure implemented at Kamarjani Test Site has been planned originally with three main groynes (identified as G-1, G-2 and G-3), two sub-groynes at the upstream side (identified as G-B/1 and G-B/2) and one sub-groyne at the downstream side (identified as G-A).

The criteria indicated in the following subsections have been applied to the original implementation design of the groynes. It should be noted with caution that the experiences gained with the test structure during the monsoon seasons after its construction, as well as further model test have demanded the adaptation of certain design assumptions. It is, therefore, strongly recommended that for any future design works only the final Guidelines (short for Guidelines for Planning, Design and Implementation of River Training and Bank Protection Works, Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (short for Design Manual for River Training and Bank Protection Works, Consulting Consortium FAP 21/22, 2001 o) should be consulted and applied.

2.3.2 Groyne Spacing

The spacing between the individual groynes was chosen at 250 m to improve the safety level of the design for an extreme outflanking channel approaching the groyne field on a relatively large angle. Thereby it was anticipated that the bank erosion between the individual groynes could be controlled within tenable limits.

2.3.3 Groyne Orientation

Assuming an almost parallel flow along the groyne field the direction of the groyne axes was to point in upstream direction at an angle of 15° with a line perpendicular to the bank line. Based on the model results such layout was considered suitable to reduce the flow velocities along the bankline as well as over the flood plain downstream from the groynes to a magnitude not causing excessive erosion.

2.3.4 Groyne Length

In accordance with the results of the Study Phase it was decided to implement groynes consisting of an impermeable part and a permeable part. Thereby it was considered that the efficiency of the entire groyne field depends also on an appropriate ratio between the groyne spacing and the overall groyne length, which must be chosen in consideration of the direction of flow attack. It was considered advantageous that the head of the most upstream groyne should not reach into the deepest part of the channel (thalweg). Thereby it was assumed that the hydraulic loads on the (possibly) most exposed upstream structure would be less severe, since local scour depths and flow velocities near the somewhat retreated groyne head should be less than compared with a groyne head located near the deepest part of the channel.

For future standard layouts of groyne fields it was considered appropriate that the downstream groynes of a group should have a standard length of the permeable part of 60 m. However, for the

purpose of the test structures it had to be ensured that all groynes would be subjected to current attack during the monitoring period. Thus, it was decided that the downstream groynes should project somewhat longer into the river.

With the above considerations the following lengths were designed for the permeable part of the groynes for the main test field of three groynes:

- (a) most upstream groyne G-1: $L = 50$ m, in order to restrict the design loads for the most attacked structure of the groyne field,
- (b) centre groyne G-2: $L = 60$ m, corresponding to the recommended standard length for the permeable part and to ensure sufficient flow attack on the second groyne of the test field,
- (c) downstream groyne G-3: $L = 70$ m, likewise for sufficient flow attack during the monitoring period of the test structure.

The permeable length of the supplementary groynes upstream as well as downstream from the main groynes was purposely planned behind the imaginary line through the most river-sided end of the main groynes to avoid immediate attraction of the flow by these structures.

2.3.5 Permeability

Supported by the results of the physical model tests it was decided to increase the permeability of groynes from the groynes' root (originally designed 20 m behind the bankline at the time the construction started) to the groynes' head from 50 % to 80 %.

The physical model tests suggested that the efficiency of single-pile-row-groynes of somewhat large pile diameters (1.0 m to 1.4 m) is comparable with twin- or triple-pile-row-groynes with individual pile diameters of 0.5 m to 0.7 m as long as the permeability of the groyne structure is equivalent. Consequently, and in view of fast and economical construction methods the groynes were designed as mono-pile structures with pile diameters duly dimensioned in consideration of stability criteria of an individual pile.

The groyne sections on the existent flood plain were planned impermeable, assuming that this would generally lead to an additional reduction of the flow velocities along the bankline downstream from the respective groyne. The resultant negative effects namely eddy formation downstream at the transition between the impermeable groyne head and the permeable groyne section was given due consideration when planning the scour protection works. However, as proven during the first monsoon season, these effects have been much more severe in practice than anticipated. In this context Annex 7 to the Final Project Evaluation Report (Consulting Consortium FAP 21/22, 2001 h) should be consulted.

2.3.6 Submerged Groynes

As an attempt to restrict the hydraulic loads on a groyne structure at higher water levels, such as high flow velocities, floating debris and excessive waves one of the main groynes (G-3) was designed as a submerged permeable groyne. The top-level raises from about 2 m above Standard Low Water at the groyne head to Design High Water level at the groyne root (see Table 2.4-1).

Though no such consideration was made in the design of the test structure, the reduced hydraulic loads would increase the stability of the structure and require considerably shorter pile lengths in any standard design. As a disadvantage slightly higher flow velocities and slightly deeper scour holes

would have to be expected with such a design. This disadvantage, however, would be compensated by a more economical design for future standard groynes.

2.4 DESIGN CRITERIA

2.4.1 General

When deciding on the design parameters for any bank erosion prevention measure due regard must be given to the fact that the initial planning data, which may be determined long time ahead of the final structure implementation, must in all probability match reasonably with the conditions at the site where the structure is being constructed finally. Therefore, as a matter of principle, design parameters must always be chosen with an appropriate margin to allow final adaptation to the actual site conditions without material deviation.

The Final Planning Study Report (Consulting Consortium FAP 21/22 (1993 a), in particular its Annexes 16 and 21 (Consulting Consortium FAP 21/22, 1993 m and r), recommended the basic design criteria to be considered for dimensioning the groyne structures. These inputs were further elaborated and verified in consideration of the results of model test runs at RRI, Faridpur (see Section 2.2).

It should be noted that the design criteria for the dimensioning of the Groyne Test Structure at Kamarjani site apply only to these test structures, but for any future standard design the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o) shall be consulted.

2.4.2 Water Levels

The decisive water levels were determined for the location of Kamarjani through water level frequency analyses carried out on statistical databases established by FAP 25, subsequently being taken over and available with the Surface Water Modelling Centre (SWMC). The frequency analyses covers a 25-years period and produced the results compiled in Table 2.4-1.

Water Level	Reference to PWD	Reference to SLW	Return Period
Standard Low Water (SLW)	15.2 m+PWD	± 0.00 m+SLW	-
Standard High Water (SHW)	20.7 m+PWD	5.50 m+SLW	-
Design High Water Level (DHW)	22.9 m+PWD	7.70 m+SLW	25
High Flood Water Level (HFW)	23.3 m+PWD	8.00 m+SLW	100

Table 2.4-1: Design Water Levels Kamarjani Test Site

Therein the High Flood Water Level (HFW) represents the statistical highest high water level for a 100-years return period, which is normally considered the design water level for bank protection measures in Bangladesh. For the purpose of the Bank Protection Pilot Project, however, the Design High Water Level (DHW) has been determined corresponding to the probability of reoccurrence within a 25-years period.

2.4.3 Design River Bed Profile

Numerous measured bed profiles of the Jamuna river have been compared within the Study Phase in order to arrive at a Standard River Profile for the purpose of the initial physical model tests at RRI. Based upon this analysis the theoretical riverbed profile within the selected test site area was to be assumed at about 10 m+PWD (- 5.20 m+SLW) to 6 m+PWD (- 9.20 m+SLW) at the heads of the planned main groynes.

Also resulting from the said analyses of Jamuna river bed profiles the slope of the riverbed can be characterised as listed in Table 2.4-2.

From	To	Slope
Flood plain level	4 m below SLW	1 to 3
4 m below SLW	10 m below SLW	1 to 5
10 m below SLW	Beyond	1 to 10

Table 2.4-2: Standard Jamuna river bed slopes

To ensure the required flexibility (see Subsection 2.4.1) in adjusting the structures to the finally determined site location, the Design River Bed Level was fixed at 5.2 m+PWD (- 10.0 m+SLW) at 65 m distance from the river-sided head of the impermeable part of the groynes.

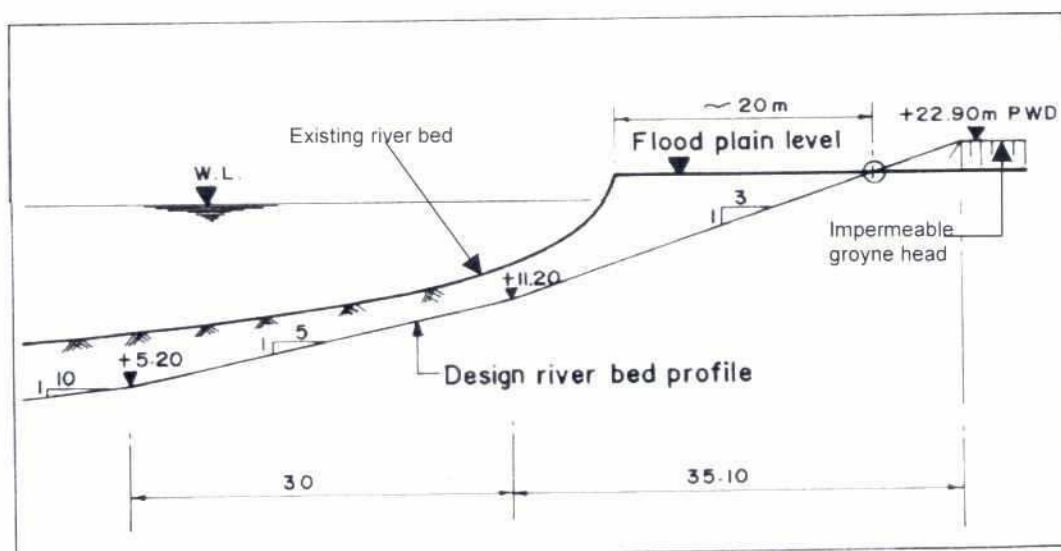


Fig. 2.4-1: Design River Bed profile

2.4.4 Hydraulic Loads

(a) General

The hydraulic loads on a groyne structure are characterised by the flow velocity and wave climate at the respective site location. Floating debris, which is eventually piling-up upstream from a groyne structure will influence the flow velocities and subsequently the scour development downstream from the groyne head.

(b) Design Flow Velocity

As design flow velocity the depth averaged flow velocity (\bar{u}) had been selected, which definition has universally been used in all design formulas and computations of the Project. Fig. 2.4-2 presents the general understanding of definitions used, but a more elaborate presentation is contained in Annex 7 to the Final Project Evaluation Report (Consulting Consortium FAP 21/22, 2001 h).

The influence of the secondary flow and of deviating turbulence characteristics of the flow along the test site had been compensated by increasing the design flow velocity.

With due regard to the test character of the groyne structures the piles of the permeable groynes had been designed in consideration of a calculated flow velocity for water levels with a 25-years return period [\bar{u}_{25}]. Thereby it was anticipated that, for the purpose of monitoring the effects of the groyne structures on the behaviour of the river in the area, the pile structure should remain in place also under more severe circumstances.

Contrary to this, however, revetments, bed protections and falling aprons for Kamarjani Test Site were dimensioned in consideration of a calculated flow velocity for water levels with a 2-years return period [\bar{u}_2], to avoid over-designing and thus to allow for a certain damage to those parts of the structures, in accordance with the design philosophy of the Project.

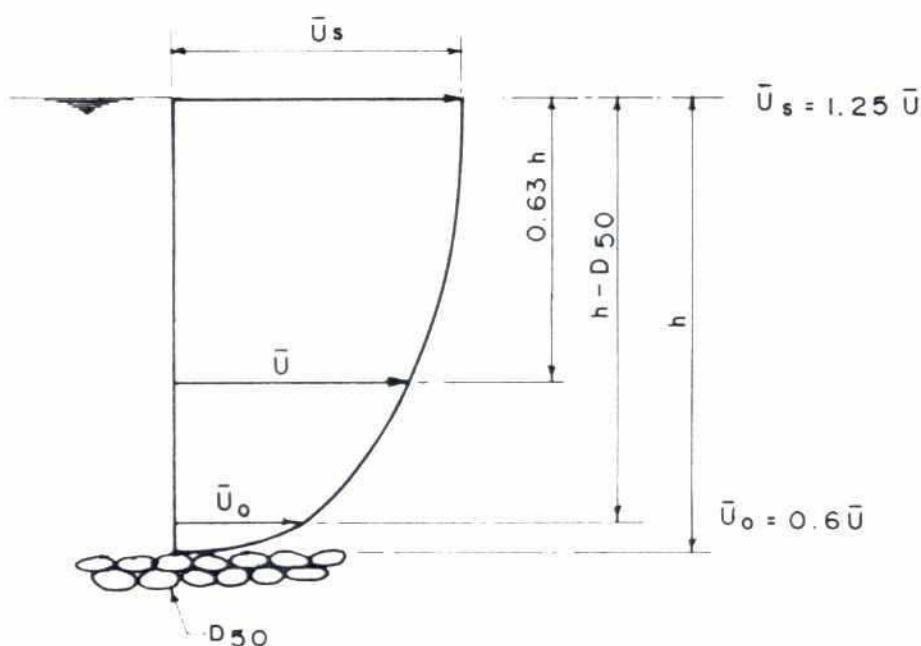


Fig. 2.4-2: Logarithmic flow velocity profile

The depth averaged flow velocity \bar{u} as determined for Kamarjani Test Site is presented in Table 2.4-3. Again, it should be noted that these assumptions can not be transferred to any other site, but the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o) should be consulted.

Distance from Impermeable Groyne Head	Depth Averaged Flow Velocity \bar{u} [m/s]	
	2-years Return Period [\bar{u}_2]	25-years Return Period [\bar{u}_{25}]
00 - 15 m	1.3	1.3
15 - 30 m	2.1	2.1
30 - 45 m	2.3	2.9
45 - 60 m	2.3	3.2

Table 2.4-3: Design Flow Velocities (\bar{u})

(c) Wave Loads

Waves are generated on the Jamuna river by wind during the tropical Norwesters, by cyclones and by passing of ships. Since inland waterway transport does not yet play an important role for the upper reach of the Jamuna river, ship induced waves are infrequent and have therefore not been given consideration in the design assumptions for the test structures.

The intensity of tropical cyclones is comparably low in the interior location. Tropical cyclones are expected to occur in this region once in 30 years only. Due to this rare recurrence, cyclonic storms have not been considered as a design criteria for the test structures.

Important for the Jamuna river are the squalls during pre-monsoon (Norwesters) and post-monsoon periods, which generate considerable waves on the river. Squalls are local disturbances causing substantial wind flows with thunderstorms, mainly occurring in the months of March to May, but also at other times of the year.

Detailed studies on the wind and wave conditions at the Jamuna were carried out under FAP 1-project as well as in connection with the Jamuna Multipurpose Bridge Project, the results of which were given due consideration in the Final Planning Study Report (Consulting Consortium FAP 21/22, 1993 a and j).

For the test site at Kamarjani a brief analysis was carried out for wind generated shallow water waves, which is presented in Attachment 1 to this Design Report. It should be noted that this paper represents only a rough analysis of the wave conditions, however, the results are considered appropriate for the present purpose.

For the conditions at Kamarjani Test Site the design wave parameters are compiled in Table 2.4-4.

Parameters of Design Wave	Return Period	
	2 years	100 years
Significant wave height H_s	0.65 m	1.0 m
Wave period T	2.5 s	3.0 s

Table 2.4-4: Design Wave Parameters

(d) Floating Debris

In the absence of measurable data or observations, it was assumed in the design of the permeable Groyne Test Structure that at the most upstream groyne floating debris may pile up to about 1 m depth below any water level, but at the downstream groynes up to 0.5 m depth only.

The resultant loads on the piles are of major significance, wherefore the piling-up of floating debris has been observed, measured and recorded during the monitoring period of the Project. The respective results and conclusions are contained in Annex 7 to the Final Project Evaluation Report (Consulting Consortium FAP 21/22, 2001 h) and should be consulted for any future reference. In this context the advantages of submerged groyne structures should be considered, as referred to under Subsection 2.3.6.

For the test structures at Kamarjani it was considered justifiable that excessive debris had to be removed to retain the stability of the structures, which in fact has been done during the first monitoring season quite frequently.

The test structures were designed for an unfavourable load combination of waves and a limited thickness of floating debris. During the monitoring of the structures no evidence could be produced to justify this assumption.

2.4.5 Design Scour Depth

The results of the physical model tests carried out at RRI suggested that the scour depth at the head of a permeable groyne would be only about 6 m compared to about 20 m to 30 m at the head of an impermeable groyne with a slope of 1 in 3 and under average flood conditions. It was further ascertained that for a permeable groyne structure the deepest scour would be downstream from its head at a considerable distance from the structure. Besides, the depth and position of the scour was obviously influenced by the type of bed protection provided around the structure. Consequently, the deepest scour was not considered necessarily to be the design scour depth for dimensioning the embedded length of the piles of a groyne structure.

Table 2.4-5 shows the assumed scour parameters for the different types of scour protection around the piles of the test structure, while Fig. 2.4-3 to 2.4-5 illustrate the definitions and assumptions made.

Type of Scour Protection	Scour Profile Figure	Scour Depths at the Groyne Head (as per physical model tests 1992-93)			
		Around Piles d_{sp} [m]	Maximum depth		Design Scour Depth d_{sd} [m]
			d_{sm} [m]	distance L [m]	
Bed protection (rip-rap on filter)	2.4-3	0	7.0	50-60	0
Falling apron	2.4-4	1.0	7.0	30-40	4.0
No scour protection	2.4-5	7.0	6.0	40-50	4.0

Table 2.4-5: Design Scour Depth for Kamarjani Test Site

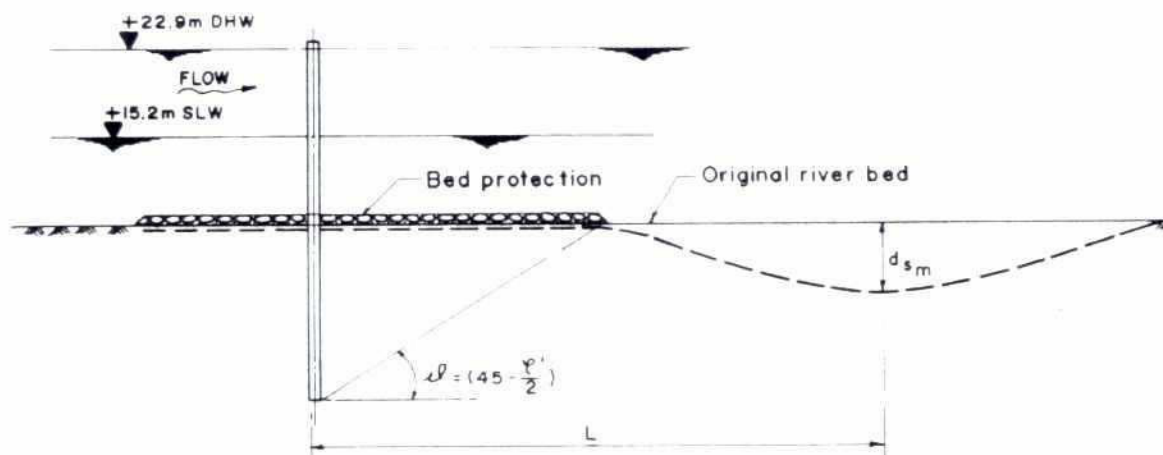


Fig. 2.4-3: Design Scour Profile – bed protection

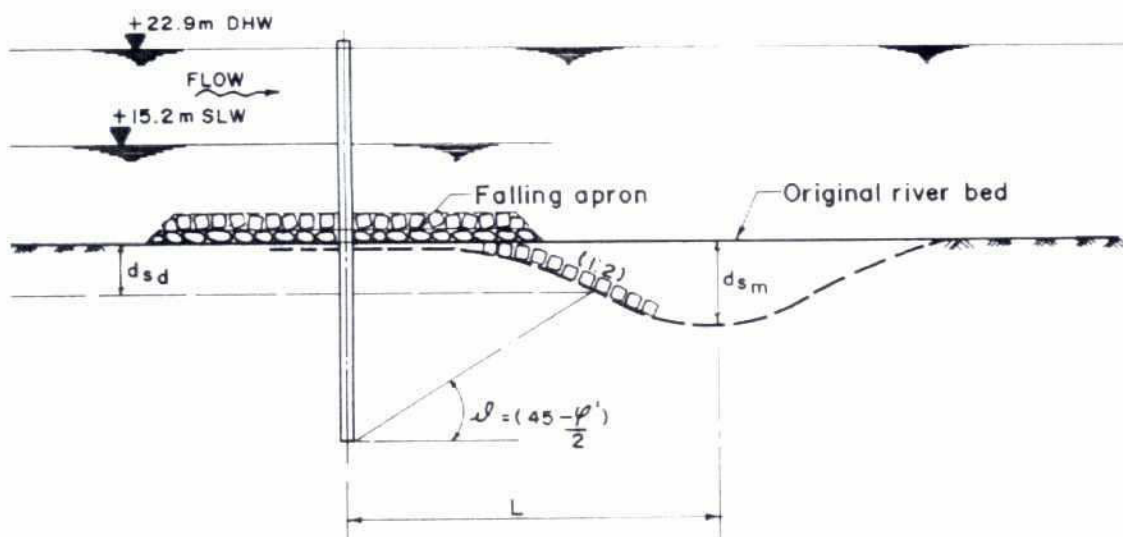


Fig. 2.4-4: Design Scour Profile – falling apron

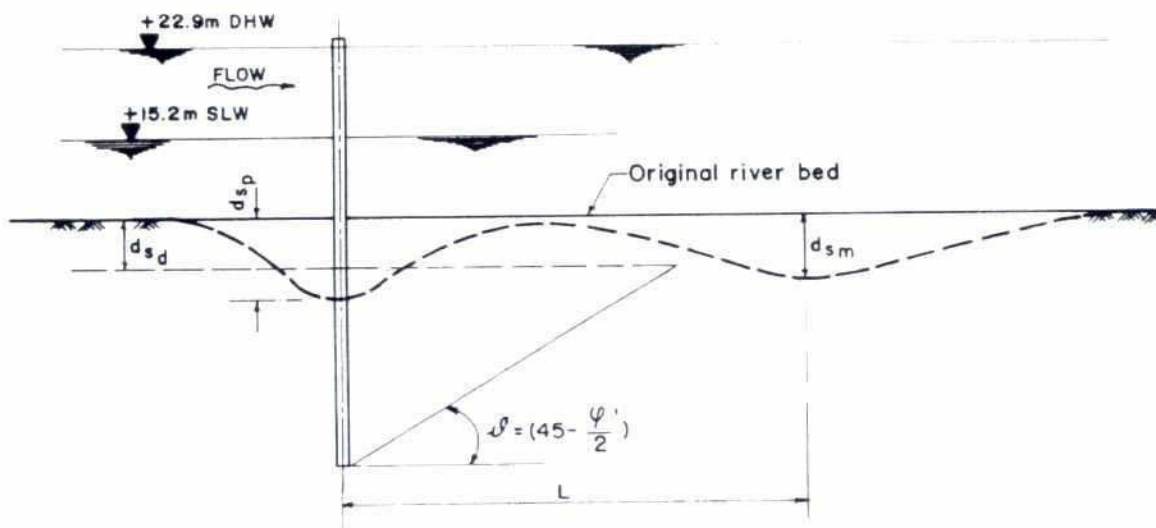


Fig. 2.4-5: Design Scour Profile – without scour protection on river bed

Based on the initial model tests it could be assumed that the scour depths would gradually reduce from the groyne head towards the bankline, which was given due consideration in the design. During the first year of monitoring, however, it became obvious that these developments follow different rules, which could not be reproduced in the physical model. Annex 7 to the Final Project Evaluation Report (Consulting Consortium FAP 21/22, 2001 h) should be consulted in this regard.

2.4.6 Subsoil Conditions

(a) General Information

During the Study Phase of the Project detailed subsoil investigations were carried out at several potential test site locations along both sides of the Jamuna river. The results are compiled in Annex 13 to the Final Planning Study Report (Consulting Consortium FAP 21/22 (1993 j) and have been compared with the data elaborated within other FAP-projects, namely FAP 1 and FAP 3.1, as well as generally available geological data. It was the conclusion that all results match reasonable well to each other and that the following tendencies would be valid for sites along the Jamuna river:

- (i) close to the flood plain surface slightly cohesive soil stratum prevails up to a depth of about 5 m on an average;
- (ii) below the upper layer loosely to medium dense deposited micaceous silty fine sand exists until a depth of about 20 m, followed by
- (iii) medium dense to dense silty fine sand, which can be assumed to prevail up to a depth of interest for bank erosion prevention measures.

Following the rapidly progressing bank erosion at the potential test site area of Kamarjani during 1992-93 it became necessary to carry out supplementary subsoil investigations end 1993, about 500 m to 700 m west of the originally investigated area. The evaluation of field samplings and laboratory tests presented finally the soil mechanical parameters, which have been used in the design of the test structures at Kamarjani and which are summarised below. The detailed results are presented in a separate Subsoil Investigation Report (Consulting Consortium FAP 21/22, 1994 b).

Despite the fact that the soil conditions along the investigated reaches of the Jamuna river may be considered somewhat uniform within the context of structures in question, it must be a standard requirement for any structure to be built that for its final design the soil properties are being determined on the basis of detailed subsoil investigations.

(b) Soil Characteristics for Pile Design of Permeable Groynes

- **Relative Density** (based on SPT n-values obtained during the soil investigations)
 - down to 5 m+PWD: loose to medium dense
 - from 5 m to ± 0.0 m+PWD: medium dense to dense
 - below ± 0.0 m+PWD: dense to very dense
- **Soil Properties**
 - unit weight/submerged unit weight: $\gamma/\gamma' = 18/8 \text{ kN/m}^3$
 - angle of internal friction: $\phi = 27.5^\circ$
 - cohesion
 - from 20.2/20.7 m
 - to 15.0 m+PWD: $c' = 7 \text{ kN/m}^2$
 - below 15.0 m+PWD: $c' = 0$

The above soil characteristics may be considered somewhat conservative. However, the values have been chosen to cover safely variations of the subsoil conditions. Subsection 3.2.6 should be referred to in this connection.

(c) **Soil Characteristics for Design of Filter Materials and Revetments**

- flood plain level: 20.2 m to 20.7 m+PWD
- lowest level for installing filter in dry condition: 15.2 m+PWD (= SLW)
- lowest level for installing filter for bed protection below water (assumed): 7.0 m+PWD
- soil classification:
 - from 20.2 /20.7 m to 18.0 m+PWD: sandy silt
 - from 18.0 m to 7.0 m+PWD: fine to medium sand

	Flood Plain Level to 18.0 m+PWD	18.0 m+PWD to 7.0 m+PWD
Grain Size Distribution (mm)		
d_{60}	0.25	0.25
d_{50}	0.18	0.20
d_{10}	0.006	0.06
Coefficient of Uniformity $U = d_{60} / d_{10}$	3 to 8	3 to 5
Coefficient of Permeability [m/s]	$\sim 3 \times 10^{-5}$	-

Table 2.4-6: Soil characteristics**2.4.7 Earthquake Loads**

Due to the character of the test structures earthquake loads were not considered in the design of groyne structures.

3 DETAILED DESIGN OF THE TEST STRUCTURE

3.1 PRELIMINARY REMARKS

Based on the planning and design criteria presented in Sections 2.3 and 2.4 and with due regard to the respective subsoil characteristics, detailed design computations were carried out during the initial stage of the Test and Implementation Phase of the Project. At that stage some tentative assumptions had to be made with regard to loading combinations. Subsequent monitoring results and studies of the structure's behaviour permit a more comprehensive understanding of the actual hydraulic and static loads on the pile structures. These results are presented in Annex 7 (Consulting Consortium FAP 21/22, 2001 h). Therefore, with exception of the transition between impermeable and permeable groyne section, the structural design of the permeable groynes at Kamarjani Test Site may be considered somewhat conservative.

3.2 PERMEABLE PART OF THE GROYNES

3.2.1 Basic Design Considerations

In a first approach, the permeable sections of the groynes were designed for two situations:

- using the statically least possible but still reasonable pile diameter, which requires twin- and triple-pile rows, and
- using large diameter tubular piles, of a size and a wall thickness, which would be produced in Bangladesh.

The comparison of such different designs carried out in 1993 showed, that, giving due regard to

- material procurement aspects;
- time limitation by construction windows calling for fast and easy construction;
- equivalent structure efficiency, and
- economical considerations

twin- or triple-pile row groynes should be ruled out. Table 3.2-1 compares the number of steel piles and associated steel weight required for a twin-pile groyne structure and a mono-pile structure in support of this statement.

Type of Permeable Groyne with Equivalent Permeability	Total Number of Piles	Total Steel Weight (t)
Twin-piles ϕ 508 x 12 mm	86	340
Mono-piles ϕ 1016 x 12.5 mm	21	185

Table 3.2-1: Comparison of a 60 m long permeable groyne of mono-pile and twin-pile design

Consequently, the detailed design of the test structures at Kamarjani site followed the mono-pile system, with due regard to the limitations of future steel pile production in Bangladesh, which are (as of the year 1993/94):

- maximum diameter to which steel plates can be transformed mechanically: 1,800 mm
- maximum steel plate thickness that can be formed to the said maximum diameter: 14.2 mm

- steel grade of the plate material rolled by Bangladesh steel mills

equivalent to St 37

3.2.2 Structural Design Computations

The detailed structural design computations relevant to the test structures at Kamarjani are not attached to this Design Report, but only the respective results. A simplified and more practical design approach has been developed meantime, which is briefly presented in Section 6 of this Design Report. For any future design work, however, the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o) should be consulted.

3.2.3 Load Combinations and Permissible Stresses

With due consideration to the anticipated different loading situations and their single or combined occurrence, the following loading classes were introduced for the detailed design of the groyne structures:

<u>Loading Class 1:</u>	-	Design Water Level:	22.9 m+PWD
	-	Flow velocity:	as per 25 years reoccurrence (\bar{u}_{25})
	-	Scour depth:	as per Table 2.4-5

The resultant loads must be covered within regular permissible stresses.

<u>Loading Class 2:</u>	-	Design Water Level:	22.9 m+PWD
	-	Flow velocity:	for 25 years reoccurrence (\bar{u}_{25})
	-	Scour depth:	as per Table 2.4-5
	-	Floating debris:	as per Subsection 2.4.4

The permissible stresses may be increased by 15 %.

<u>Loading Class 3:</u>	-	Design Water Level (100 years return period):	+ 23.3 m+PWD
	-	Flow velocity:	for 100 years reoccurrence
	-	Scour depth:	assumed 2 m for groyne G-1
	-	Floating debris:	1 m thickness
	-	Wave loads:	$H_s = 1.0$ m

The permissible stresses may be increased by 30 %.

According to the experience gained during the test and monitoring period it appears uneconomical to combine all excessive loads in Loading Case 3 since the probability of its simultaneous occurrences is rather low. It is reasonable to consider either a wave load of $H_s = 1.0$ m or floating debris of 1 m thickness, but not both at the same time. With a consideration of one meter floating debris all reasonably to be expected wave loads are covered, say up to about 50 cm height.

3.2.4 Tubular Steel Pile Structures

(a) Selection of Steel Grade

There are various aspects to be considered for deciding on the appropriate steel grade to be used for the tubular steel piles of a groyne structure.

- The standard steel grade that would be available from Bangladesh steel mills corresponds to Grade St 37-3 with a tensile strength of 340-470 N/mm² and a minimum yield point of 235 N/mm² ($t \leq 14.2$ mm);

- (ii) Permeable groyne structures consist of a row of free-standing piles subjected to horizontal loads. Resultant bending moments can more economically be carried with higher steel grades than St 37-3 or equivalent (the steel weight and supply cost would reduce, on the other hand such steel may have to be imported);
- (iii) Consideration must be given to the stability/integrity of large diameter pile sections that may require additional plate thickness to avoid buckling. In such case the use of high-grade steel may not produce the advantage as per para (ii);
- (iv) For executional reasons, but also for reasons of para (iii) above it must be avoided that the plate thickness of a tubular steel pile falls below a minimum, which corresponds to 1/100 of the pile diameter (e.g. for diameter 1,000 mm the minimum plate thickness must be 10 mm);
- (v) Turbulence around the piles caused by the river's flow and small but short wind waves may initiate a considerable vibration/oscillation of the respective pile. If a particular configuration exceeds a stage defined by the pile's diameter, its exposed length and the load, the initiated vibration may exceed a critical limit. This could in the long-term result in a failure of the pile due to fatigueness of the steel. Material fatigue is, however, more critical with high-grade steels;
- (vi) Steel piles are delivered normally in standard lengths of maximum 12 m, whereby road and rail transport are the limiting criteria. Site welding to full-length piles has, therefore, to be carried out at site. High-grade steel requires special skill and attention for welding site butt joints, which may be difficult to meet at remote site locations.

With the above considerations it was decided (and is also recommended for future) to use only standard steel grades, such as St 37-3 or equivalent.

(b) Initial Tubular Steel Pile Standard Sizes

With the load assumption and otherwise relevant design criteria and subsoil parameters described in this Design Report the tubular steel pile diameters listed in Table 3.2-2 were finally chosen to be suitable for the different situations at Kamarjani Test Site. The maximum pile diameter recommended for the conditions in Bangladesh is 1,420 mm, corresponding to the maximum available steel plate thickness 14.2 mm multiplied by 100 in accordance with para (a) above.

	Permeability of Groyne			
	50 %	60 %	70 %	80 %
Pile Size [mm]	φ 711.2 x 12.5	φ 1016 x 14.2	φ 1220 x 14.2	

Table 3.2-2: Selected Tubular Steel Pile standard sizes at Kamarjani Test Site (steel grade St 37-3 or equivalent)

For the purpose of the test structures only, it was considered justified to cover also for some unforeseeable circumstances, such as adjustment to final site conditions, late start of construction works, early start of monsoon season, etc. These events could have effected the proper implementation of the test structures within the given construction window. Therefore, and for other executional considerations, such as expected driving difficulties, the wall thickness of the selected pile diameters was increased to 14.2 mm (dia. 711.2 mm) and 20 mm (dia. 1016/1220 mm) respectively. The pile lengths were increased to maximum 36 m, in consideration of customary standard delivery lengths of 12 m.

With the aforesaid adjustments the most exposed piles of the groynes could have been subjected to unexpected scouring beyond the assumed design values. Within 90 % of yield stress of the steel

material scour depths near the groyne heads of up to about the level - 2.50 m+PWD, corresponding to a design scour hole depths of ~ 6.5 m (Groyne G-3) to ~ 9.5 m (Groyne G-1) below design river bed level and of 12 m to 10 m below expected river bed level respectively could have been accommodated without taking a risk of failure or deformation of the piles.

As said elsewhere in this report the reach of the Jamuna river under consideration is not yet utilised for intensive inland waterway transport. Nevertheless, to cover for accidental impact loads to the groyne structures, such as ship impact (e.g. caused by the oil barge plying on the river) the groyne heads were designed with twin piles. Dependent on the location of future groyne structures, this aspect should be given special consideration, see also Subsection 3.2.7 in this regard.

3.2.5 Reinforced Concrete Piles

(a) Basic Considerations

An attractive alternative to the use of tubular steel piles is the use of reinforced concrete piles. In view of the required length of such piles only

- bored cast in-situ reinforced concrete piles;
- coupled prestressed spun concrete piles, or
- coupled pre-cast reinforced concrete piles

would be a feasible option. These types of piles can be produced in Bangladesh, making best use of locally available materials, plant and construction methods.

However, for the application of reinforced concrete piles certain constraints must be given well consideration, which are highlighted in the following subsections.

(b) In-Situ Cast Reinforced Concrete Piles

The construction of bored in-situ cast reinforced concrete piles is a well-established method in the country, at least for the deep foundation of bridges, drainage structures/weirs, high-rise buildings, etc. The available equipment capacity restricted the execution of pile diameters in excess of 1.5 m. The applied methods are still kept highly labour intensive. One advantage of these piles is the ease of equipment mobilisation, in terms of logistics and time. The not too heavy equipment can reach even remote locations and is mostly set-up by labour.

For designing horizontally loaded piles it is to be considered that reinforced concrete piles have (within the limitations of feasible diameters) only limited bending moment capacity. Besides, the execution should always be carried out on-shore, e.g. on the flood plain reaches. Floating execution would be feasible, but requires more sophisticated floating equipment, which is not readily available in the country.

The production of the in-situ concrete piles requires strict adherence to concrete quality, which might be a constraint when proper natural aggregates (river gravel or crushed rock) are not available.

Unlike steel piles which can be extended by welding and re-driven into the ground if needed to cope with unexpected scouring, it is a disadvantage of cast in-situ concrete piles that their embedment length into the subsoil can not be increased once constructed.

The in-situ cast bored pile groyne structures at Kamarjani (G-A, G-B/1 and G-B/2) were designed to suit a maximum water depth of 16 m at the groyne head, and constructed entirely on the flood plain. During the first season the permeable part of Groyne G-A (downstream from the main groyne field) became fully exposed to the river's attack. The scour depth around the piles exceeded by far the design scour depth but the piles remained in position. For completeness it is noted here that during the structure

adaptation works in 1996 the most exposed concrete piles were substituted by additional steel piles installed just upstream from the said concrete piles. The most river-sided concrete piles were finally lost during the subsequent monsoon season due to excessive scouring.

Nevertheless, in-situ cast concrete piles represent a suitable alternative for conditions of moderate scouring, e.g. in the near-bank zone of a groyne, but should always be combined with a suitably dimensioned scour protection around the piles, e.g. by falling apron.

Table 3.2-3 compiles the diameters of bored in-situ cast reinforced concrete piles chosen to be suitable for the given situations at Kamarjani test site.

Permeability	50 %	60 %	70 %	80 %
Diameter [inch]	ϕ 20"	ϕ 36"		
Diameter [mm]	ϕ 500 mm	ϕ 914 mm		
Limiting Water Depth (test structure condition)	7 m	16 m		

Table 3.2-3: Selected diameters of in-situ concrete piles for permeable groynes with scour protection

Groynes with concrete piles are intended only for construction on the flood plain so that bed protection can not be provided at the time of its construction. However, as experienced during the monitoring period of the Project it is essential for any groyne planned with concrete piles that a proper and sufficiently large scour protection is being provided. Therefore, by the time that progressing bank erosion has exposed the groyne's piles the scouring must be monitored and appropriate measures must be implemented in time to install the bed protection (falling apron) around the concrete piles, otherwise the risk of total loss of the groyne's exposed piles is taken.

The in-situ cast concrete piles for the test structure at Kamarjani were designed only with a nominal reinforcement, corresponding to 0.8 % of the pile cross-section and under the assumption of a concrete quality corresponding to Grade 20 only. Based on the practical experience gained during the Project implementation this assumption has been too conservative. The concrete quality achieved under the remote site conditions was more than satisfactory (at least equivalent to Grade 25), which justifies that for future standard designs the reinforcement could be increased to 2 %. With such condition it is feasible to plan the use of reinforced in-situ cast concrete piles for a water depth (including anticipated scour depth) of 20 m to 25 m and an expected flow velocity of 2.5 to 3.0 m/s. Section 6.2 of this Design Report presents a suitable design diagram, but for any future design works only the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o) should be referred to.

(c) Prestressed Spun Concrete Piles

At the time of designing the Groyne Test Structures for the Kamarjani site one Bangladesh company accepted an offer of the Project to convert their production facilities for power transmission line poles to produce prestressed spun concrete piles according to a design of the Project. The production facilities required limiting the pile dimensions to diameter 500 mm and an overall length per pile section of 10 m. For the purpose of testing the production, transportation and installation it was decided to utilise these piles in the near-bank zone up to an anticipated water depth of 12 m. The required total pile length was 20 m, wherefore the pile sections were provided with a purpose-made coupling.

Production, installation and behaviour of these piles were without complaint, but transportation from the factory at Panchagarh to the site had created problems and incurred unexpected high cost. Annex 5 (Consulting Consortium FAP 21/22, 2001 f) provides more details in this regard.

The use of prestressed spun concrete piles for future standard solutions should, however, not be ruled out. With strategic material stocks (refer to Section 1.4) and improved transport logistics on a large scale it would allow a fast implementation at least of the near-bank section of groynes.

Design Drawing No. KA-403 presents the design details of a prestressed spun concrete pile and is attached to this report (see Attachment 4).

3.2.6 Determination of Pile Embedment Length

The term “pile embedment length” used within this Design Report means the length of the respective pile (whether of steel or reinforced concrete) statically required below the level of the design scour depth, as presented in Fig. 3.2-1.

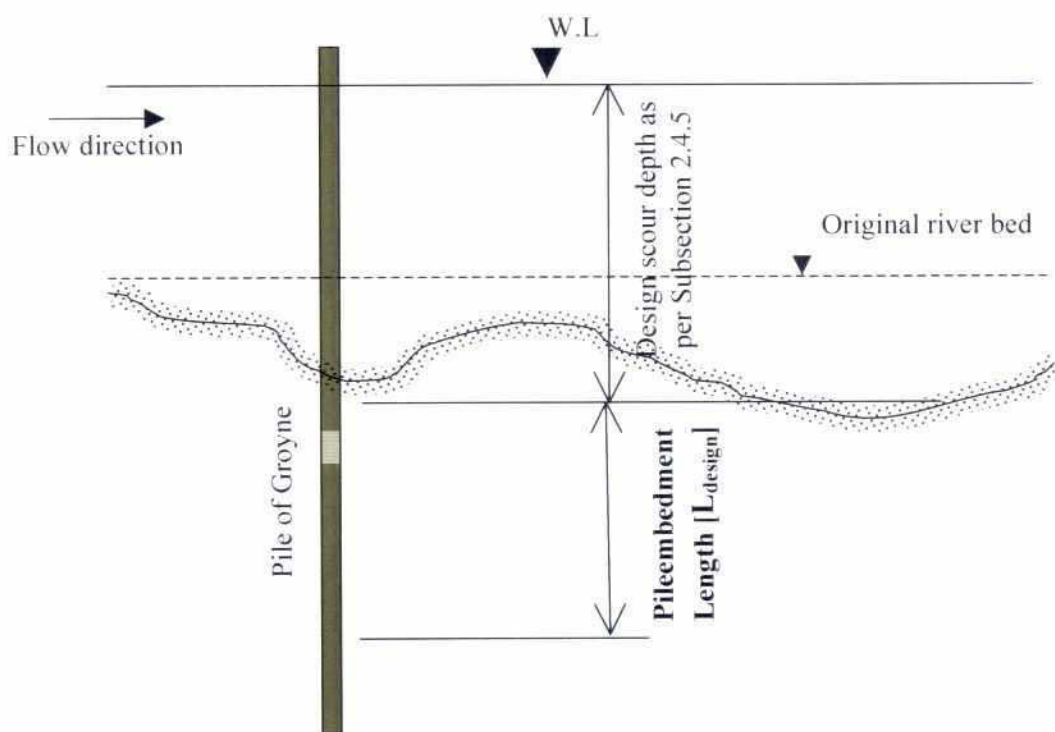


Fig. 3.2-1: Definition of pile embedment length

The embedment length of piles has been determined in consideration of the soil characteristics given under Subsection 2.4.6. The computations followed the so-called BLUM-method in conjunction with R 69 of EAU 1990, a method originally developed for the design of single pile and multi-pile dolphin structure.

Comparative computations have indicated that varying parameters of non-cohesive soils/slightly cohesive soils, i.e. internal angle of friction ranging from $\phi' = 27.5^\circ$ to $\phi' = 32.5^\circ$, unit weight (under uplift) $\gamma' = 8 \text{ kN/m}^3$ to $\gamma' = 10 \text{ kN/m}^3$ or cohesion $c' = 0$ to $c' = 7 \text{ kN/m}^2$, have only a marginal effect on the embedment length of the piles, ranging from only 3 % to about 10 % of the computed length

for the more unfavourable soil conditions. It is, therefore, considered justified to apply the more unfavourable soil characteristics also for future standard designs.

For future designs the final determination of the pile embedment length should follow the steps hereunder:

- (1) compute the theoretical pile embedment length [L_{theo}] using the design tools presented in Section 6.2;
- (2) apply a multiplier of 1.25 for covering the unforeseeable circumstances that must be expected with a river like the Jamuna, to arrive at the design pile embedment length [L_{design}], and
- (3) irrespective of the result of computation as per para (2) above the minimum pile embedment length for any situation should not be less than 5 (five) meters.

$$L_{design} = 1.25 \cdot L_{theo} \geq 5.0 \text{ m} \quad (3.2-1)$$

3.2.7 Navigation Marking of Groyne Structures

Permeable groynes project into the river and present a danger for country boats, inland waterway vessels and bamboo/timber rafts navigating on the river. This holds in particular during high flood water levels.

In line with the international waterway marking recommendations SIGNI of the International Association of Lighthouse Authorities (IALA) the heads of the groynes at Kamarjani were provided with the so-called "East Cardinal Mark", which indicate that the safe side on which the danger is to pass lays to the East of that mark.

It is recommended that for future permeable groyne structures the same navigation mark system should be applied since it corresponds also with the IALA Maritime Buoyage System.

The following rules apply:

(a) Definition of Cardinal Quadrants and Marks:

- the four quadrants (North, East, South and West) are bounded by the true bearings NW-NE, NE-SE, SE-SW and SW-NW taken from the point of interest;
- a Cardinal Mark is named after the quadrant in which it is placed, and
- the name of the Cardinal Mark indicates that the mark should be passed on the side of the quadrant named.

(b) Use of Cardinal Marks

Cardinal Marks may be used

- to indicate that the deepest water in that area is on the named side of the mark;
- to indicate the safe side on which to pass the danger, and
- to draw attention to a particular feature in a channel, such as a bend, a junction, a bifurcation or the extremity of a shoal.

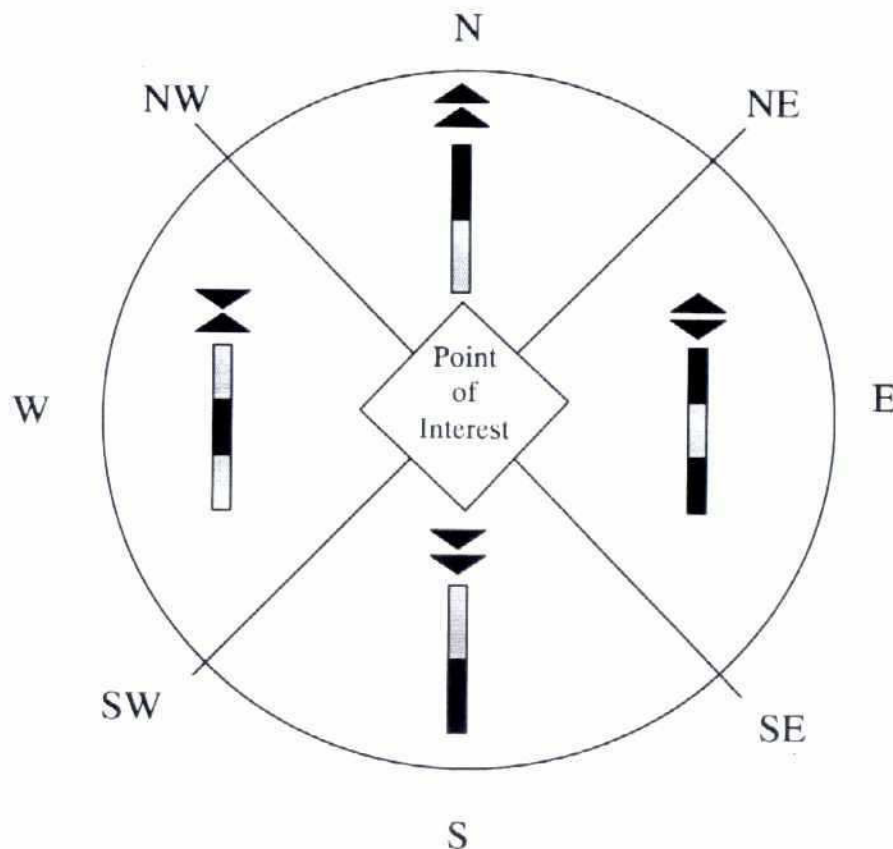


Fig 3.2-2: Designation of cardinal marks

A detailed description of the Cardinal Marks including its night marking may be taken from SIGNI. A typical design of daylight East Cardinal Mark is presented on Drawing No. KA-113 (see Attachment 4).

3.3 IMPERMEABLE PART OF THE GROYNES

3.3.1 Basic Considerations

The concept of the groyne structures at Kamarjani Test Site is based on a configuration with a bank-sided impermeable part corresponding to about half of the total groyne length and a permeable part extending into the river.

Within the test structures of Kamarjani the impermeable part was realised by two principle solutions

- sheet pile cofferdam, and
- earth dam with slope protection,

the design bases of which are presented in the following subsections. It was considered at the time of planning the test structures that the most upstream groyne of the groyne field would be exposed to substantially higher attack wherefore it was decided to construct the impermeable groyne as a cofferdam. The design scour depth as well as the design flow velocities at the head of the impermeable groyne heads were decided on the basis of the physical model tests carried out at the initial phase of the project.

The monsoon seasons following the construction of the groynes at Kamarjani have produced results, which required revising the initial design assumptions. One of the main lessons learned during the first year of testing the groyne field is the fact that the transition between the impermeable and permeable section of the groyne is the most critical point of the entire groyne structure. Subsequent model tests resulted in the recommendation that the transition must be designed extremely gentle or if other constraints (land availability) exists, impermeable groyne sections should be avoided. A detailed analysis of the observations is presented in Annex 7 (Consulting Consortium FAP 21/22, 2001 h), while Chapter 4 presents an overview of the most important results that would be decisive for future standard designs.

3.3.2 Sheet Pile Cofferdams

(a) Preliminary Remarks

Sheet-pile cofferdams can be constructed of steel sheet-piles as well as of reinforced concrete sheet-piles. The first mentioned have to be imported also in future, since local manufacture is not feasible. Reinforced concrete sheet-piles can be manufactured locally. The production must be subject to strict quality control to ensure straightness and well fitting of the groove and tongue interlocks. As a disadvantage it is to note that reinforced concrete sheet-piles have a practical limitation in size and length (max. 10 m due to transport conditions) and are, therefore, not suitable for situations where severe scouring must be expected. For accommodating severe scouring at the head of a cofferdam the use of long steel sheet-piles can not be avoided.

For the test structures at Kamarjani both solutions have been implemented in order to experience the applicability of both under the given remote conditions.

(b) Design Parameters

The design parameters were chosen as follows:

- construction fully within the flood plain area;
- top level of the cofferdam according to Design High Water Level DHW = 22.9 m+PWD (refer to Subsection 2.4.2);
- width of the cofferdam about 8 m, with the intention to utilise this area as a strategic stockpile for boulders and concrete blocks to be available for any emergency action during a season;
- groyne head gently sloped from floodplain level up to the crest level at 1 in 5;
- surface load on the entire cofferdam area (except the sloped part) corresponding to a 1.5 m high stockpile of boulders;
- scour depth around the cofferdam head 4.5 m;
- subsoil characteristics as per Subsection 2.4.6;
- design of the groyne head by compartments to avoid total failure of the cofferdam in case of some damage to its head section, and
- design of a bed protection along the parallel sides of the cofferdam and around the cofferdam head to meet with the anticipated flow conditions.

Assumed flow velocity:	parallel sides of the cofferdam	1.5 m/s
	cofferdam head	2.5 m/s.

Fig. 3.3-1 displays the design profile of the cofferdams as applied to the test structures. It should be noted, however, that the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design

Manual (Consulting Consortium FAP 21/22, 2001 o) contain the information that should be applied to future Standard Designs.

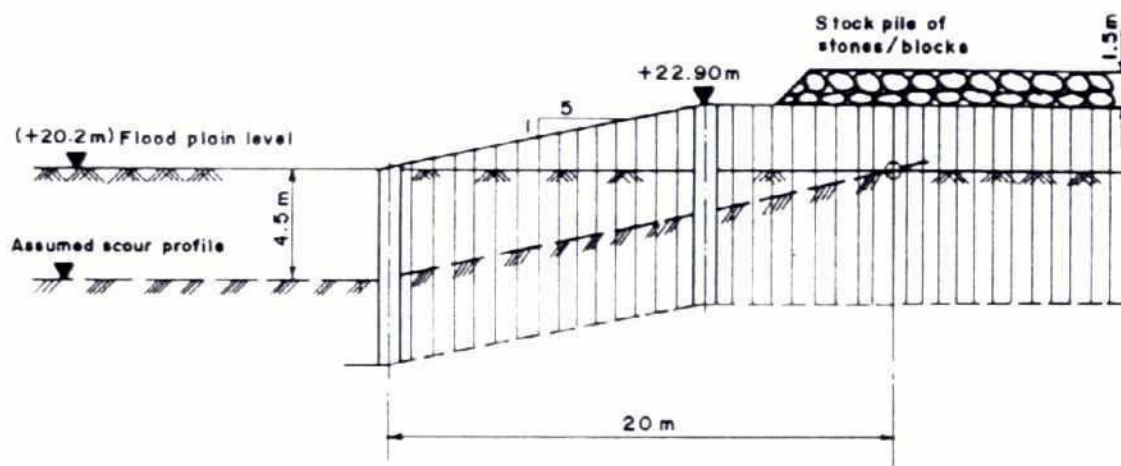


Fig. 3.3-1: Design Profile Cofferdam as impermeable groyne section

(c) Steel Sheet-Pile Cofferdam

With the design parameters of para (b) above, a steel sheet-pile profile Larssen 20 or ARBED PU-6 of steel Grade StSp 37 was chosen. The length of individual sheet-piles was determined at 8 m according to the structural design computations.

The parallel walls of the cofferdam as well as the compartments at the head were tied by steel anchor sets of diameter 50 mm, steel Grade St. 52-3.

The space between the cofferdam walls was filled with suitable material in layers and compacted. The top surface of the sloped groyne head was given pavement by concrete blocks size 20x20x20 cm, laid interlocked on a 30 cm thick mixed granular filter, to withstand the expected flow velocities. The crest area was given a cover by Durba grass sods.

The respective Design Drawings No. KA-106 up to and including No. KA-108/II are attached to this Design Report (see Attachment 4).

(d) Reinforced Concrete Sheet-Pile Cofferdam

In order to withstand the driving forces during its installation a concrete of Grade 45, corresponding to a compressive strength (after 28 days) of 450 N/mm² must be used for the production of reinforced concrete sheet-piles.

With the locally available reinforcing steel Grade 60 /ASTM A-615 and the design parameters of para (b) above, a reinforced concrete sheet-pile profile of 490 mm width and 250 mm thickness has been designed. The length of individual sheet-piles was determined at 8 m according to the structural design computations. The detailed design of a sheet-pile unit is presented on Drawing No. KA-405 in Attachment 4.

The parallel walls of the cofferdam as well as the compartments at the head are tied by steel anchor sets of diameter 50 mm, steel Grade St. 52-3.

A general weakness of reinforced concrete sheet-piles is the poor tightness of the interlocking joints. If not appropriately designed and executed, the interlock joints are likely to permit seepage of backfill material, which in the worst could initiate instability and ultimate failure of the entire cofferdam. For this reason a well-graded granular filter was designed and finally provided in the test structure just behind the driven sheet-piles. Simultaneously with the placing of the filter material the remaining space was filled with suitable material in layers and compacted.

The top surface of the sloped groyne head was given pavement by concrete blocks size 20x20x8 cm, laid interlocked on a 30 cm thick mixed granular filter, to withstand the expected flow velocities. The crest area was given a cover by Durba grass sods.

The respective Design Drawings No. KA-405 up to and including No. KA-407/II are attached to this Design Report (see Attachment 4).

The production and installation of the reinforced concrete sheet-piles has not caused any problem. Within the limitations of production lengths up to say 10 m, these sheet-piles are considered a feasible option for situations where only moderate scouring is to be expected around the head of the cofferdam.

3.3.3 Earth Dam with Revetment

(a) Design Parameters

The design parameters were chosen as follows:

- top level of the earth dam according to Design High Water Level DHW = 22.9 m+PWD (refer to Subsection 2.4.2);
- crest width 8 m, with the intention to utilise this area as a strategic stockpile for boulders and concrete blocks, to be available for any emergency action during a season;
- groyne head and sides sloped at 1 in 3;
- surface load on the entire crest area corresponding to a 1.5 m high stockpile of boulders within 5 m width;
- scour depth near the downstream side of the earth dam head of 4.5 m with an assumed slope of the scour hole about 1 in 2;
- subsoil characteristics as per Subsection 2.4.6;
- fill for the earth dam to be suitable material from borrow pits with maximum 5 % silt content for material with steep grain size distribution curve, or of 10 % for material with flat curves;
- layer-wise compaction of earth dam fill to achieve a degree of density corresponding to $D = 0.60$ to 0.75 (DIN 18126);
- design of revetments and toe protection along the parallel sides of the dam and around its head to meet with the anticipated flow conditions.

Assumed flow velocity:	parallel sides of the earth dam	1.5 m/s
	dam head	2.5 m/s.

(b) Filter Design

In advance of the Revetment Test Structure at Test Site II implemented in the season following the construction of test groynes at Kamarjani several filter alternatives were already designed for the impermeable groyne sections as well as the main embankment. Different types of geotextile filters as well as granular filters were designed, well in consideration of the prevalent subsoil conditions.

The aspects of filter design are exploited in detail in Annex 8 (Consulting Consortium FAP 21/22, 2001 i) and are, therefore, not discussed further within this Design Report on the Groyne Test Structure.

As a speciality, however, reference is made to Subsection 3.5.2 in connection with a geo-jute sand filter mattress for utilisation within bed protections to be installed below water. The practical side of handling and placing such a mattress is dealt with in Annex 5 (Consulting Consortium FAP 21/22, 2001 f).

(c) Slope Stability

With the design assumptions listed under para (a) above, slope stability computations were carried out whereby different loading and scouring conditions were investigated within the limits of the said parameters. An overall slope of 1V in 2H was found to be safe within the assumptions made, but a slope of 1V in 3H was finally chosen for the design.

Two of the main groynes were constructed with earth dams within the impermeable section of the groynes, namely G-2 and G-3. During the first monsoon season after construction these earth dams failed, for which unexpected high return currents and excessive scouring near the head of the impermeable groyne section are the main causes. A detailed analysis of the failure mechanism is contained in Annex 7 (Consulting Consortium FAP 21/22, 2001 h). Reference is also made to Subsection 3.3.1.

(d) Revetment Design

The revetments and bed/toe protections were designed for flow velocities and wave conditions, which with all probability could have been expected to occur within the monitoring period. The respective assumptions were derived from the data compiled in Subsection 2.4.4.

Slopes reaching down to about 17.2 m+PWD, i.e. 2 m below SLW, were considered to be subjected to the effects of flow and wave loads, while for slopes below this level only flow attack was considered for dimensioning the protective layers.

At the initial start of the Test and Implementation Phase of the Project a comparison of different design formulas for revetments was carried out showing that the results are generally within a reasonable margin. For information the respective formulas are listed below, while the respective annotations are contained in Attachment 2 to this Design Report.

(i) Stability of Embankment Revetments under Current Attack

(1) Pilarczyk-Formula (1990)

$$D_n \geq 0.85 D_{50} = \frac{0.035 \cdot u_b^2}{\Delta_m \cdot 2g} \cdot \frac{\phi \cdot K_r \cdot K_h}{K_s \cdot \Psi_{cr}}$$

(2) PIANC-Formula (1987a) (for rip-rap)

$$D_{n50} \geq \frac{0.7 \cdot u_s^2}{\Delta_m \cdot g \cdot k}$$

(3) Modified Isbash-Formula (for rip-rap)

$$D_{n50} \geq \frac{C \cdot u_b^2}{\Delta_m \cdot g \cdot k}$$

- (4) **FAP 1-Formula**
(for concrete blocks)

$$D \geq 0.026 \cdot u^2 \cdot \frac{\pi}{6} \cdot \frac{\rho_s - 1}{\rho_c - 1}$$

- (5) **BRTS-Formula** (5/1993, FAP 1)
(for concrete blocks)

$$D \geq \frac{0.7 v^2}{2(S_s - 1)g} \cdot \frac{2}{\left[\log \left(6 \frac{h}{D} \right) \right]^2} \cdot \frac{1}{\left[1 - \left(\frac{\sin \phi}{\sin \theta} \right)^2 \right]^{\frac{1}{2}}}$$

(ii) Stability of Embankment Revetments under Wave Attack

- (1) **Pilarczyk-Formula** (1990)

$$D_n \geq 0.85 D_{50} \geq \frac{H_s \cdot \xi_z^b}{\Delta_m \cdot \psi_u \cdot \phi \cdot \cos \alpha}$$

- (2) **HUDSON-Formula**

$$W_{50} \geq \frac{\rho_c \cdot H_{des}^3}{K_D \cdot \left(\frac{\rho_s}{\rho_w} - 1 \right) \cdot \cot \alpha}$$

The various formulas presented slightly different results, which differences from calculations may be of interest from the scientific point of view. However, in a practical approach these are marginal and would be adjusted in the final designs for ease of construction works by eliminating such theoretical margins.

Within the context of formula comparison the Pilarczyk-formulas were found to present a suitable tool for giving due consideration to the various aspects of hydraulic loads and conditions. The respective comparative sample computations are attached hereto as Attachment 2.

Considering that the revetment design for the test structures was carried out for current and wave loads corresponding to a 2-years reoccurrence of events, the more unfavourable result of the two loading conditions was taken.

For practical reasons it was considered justified to limit the minimal stone/block size for any protection work within the river (i.e. except flood plain area) to 20 cm (D_{50}), even though bed protection around the head of Groyne G-1, for example, a stone size of $D_{50} = 10$ to 15 cm (depended of selected stability/turbulence factors) would have been adequate for a velocity \bar{u}_2 , as per Pilarczyk-formula.

Based on the aforementioned computations and assumptions the following materials were chosen for the slope revetment of the earth dams presenting the impermeable section of the main groynes:

- rip-rap with various gradation ranges between $D_{50} = 10$ cm and $D_{50} = 30$ cm;
- concrete blocks, cube size 20 cm, 25 cm and 30 cm, and
- brick mattresses of 15 cm and 20 cm thickness as per local customary design, depending on the location of area to be protected.

The revetment design details chosen for the test structures are presented on Drawings No. KA-206 and KA-207 in Attachment 4.

3.4 MAIN EMBANKMENT

3.4.1 Earth Dam

The design parameters for the main embankment along the test site were chosen as follows:

- top level of the earth dam according to Design High Water Level DHW = 22.9 m+PWD (refer to Subsection 2.4.2);
- crest width ≥ 4.25 m;
- sides sloped at 1 in 3, at river-side and land-side;
- subsoil characteristics as per Subsection 2.4.6;
- fill for the earth dam to be suitable material from borrow pits with maximum 5 % silt content for material with steep grain size distribution curve, or of 10 % for material with flat curves;
- layer-wise compaction of earth dam fill to achieve a degree of density corresponding to $D = 0.60$ to 0.75 (DIN 18126);

3.4.2 River-Sided Slope Protection

The design of the river-sided revetment and toe protection along the main embankment was carried out in consideration of a nominal flow velocity of 1.0 m/s near the flanks to the impermeable groynes.

Within the test structure area several revetments were implemented, such as

- brick mattresses of 15 cm and 20 cm thickness, bound together by hot-dip galvanised wiremesh;
- concrete blocks, size $20 \times 20 \times 20$ cm, placed in interlocking pattern;
- rip-rap with a nominal size $D_{50} = 20$ cm and a layer thickness of 40 cm, and
- Durba grass sods with a footing of Vetiver plantation (only in areas where minor flow velocities were anticipated).

The protective materials were laid on synthetic geotextile filters as well as granular filters of different composition. The grass sods were laid on sandy loam and nursed initially to encourage growth. Experience has shown that in its first year after planting the Durba grass coverage is too vulnerable, particularly to wave attack.

The general design details are shown on Drawing No. KA-014 (see Attachment 4).

It is recommended to consult the Design Report of the Revetment Test Structure, Annex 8 (Consulting Consortium FAP 21/22, 2001 i) as well as the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o), which contain more elaborate information.

3.4.3 Land-Sided Slope Protection

The main embankment was constructed by BWDB and a protection of the land-sided slopes could not be materialised.

Experience has shown that for maintaining the integrity of the main embankment it is a necessity to also protect its landside. Rain-cuts can be avoided with certainty, if the slopes are protected by grass such as Durba grass sods.

3.4.4 Borrow Pits

Fill material is normally being obtained from borrow pits opened along a main embankment to be built. This was also followed for the test structures at Kamarjani. The respective borrow pits were arranged to maintain at least a 5 m wide berm between the planned toe of the main embankment and the upper edge of the pit.

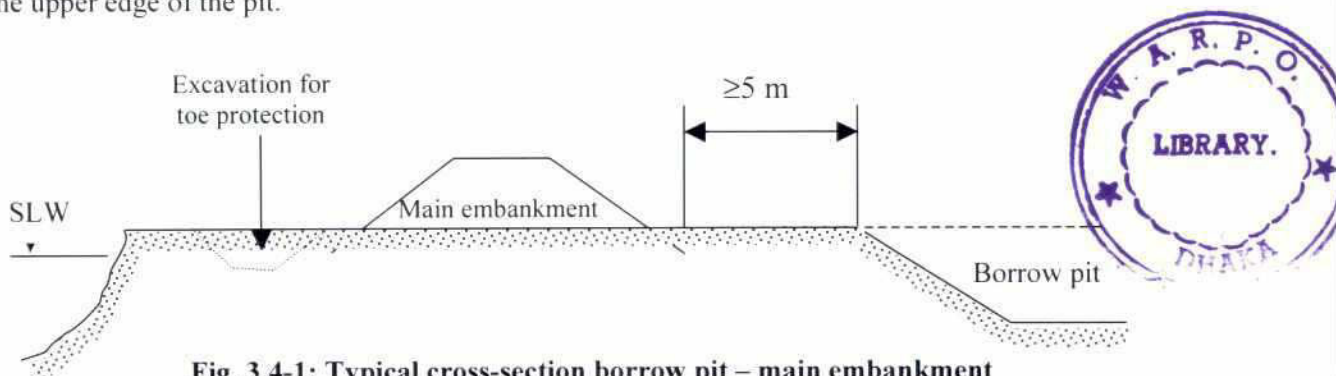


Fig. 3.4-1: Typical cross-section borrow pit – main embankment

The arrangement of borrow pits at the river side of a new main embankment should be avoided, unless its excavation level is restricted to the design level of any planned toe protection along-side.

3.5 BED PROTECTIONS / FALLING APRONS

3.5.1 Definitions

- “Bed Protection” is suitably designed (by single weight, volume and covered area) granular material, such as boulders, concrete blocks, etc., placed on a filter bed; the filter may be well-graded granular material placed in-situ, a geotextile filter mat or sandwich-type composite filter mats of synthetic filter materials combined with geo-jute and an integrated ballast sand-fill.
- “Falling Apron” is a suitably designed (by single weight, volume and covered area) layer of granular material, such as concrete blocks or boulders, placed directly on the existent subsoil and riverbed respectively.
- “Toe Protection” is, according to its true wording, intended to protect the toe of an embankment against instability due to erosion/scouring along its toe. Toe protections within the meaning of this report are falling aprons of suitable size and weight.

3.5.2 Falling Aprons / Toe Protections

The basic principles compiled under para (d) of Subsection 3.3.3 were applied analogously also for the design of falling aprons and toe protections respectively.

For the design of falling aprons only, very limited design rules were available at the time of preparing the design of test structures. For comparison known design formulas were analysed, but showed that the results are generally within a reasonable margin. In particular the following formulas were compared (the respective annotations are contained in Attachment 3 to this Design Report):

(i) Toe Protections / Falling Aprons under Current Attack

- (1) **Preliminary DELFT-Formula**
(concrete blocks)

$$D_{50} \geq \sqrt{\frac{19 \cdot R \cdot D_{50 \text{ soil}}}{c_0}}$$

- (2) **PIANC-Formula**
(1992)

$$D_n \geq 0.85 \cdot D_{50} \geq \frac{0.7 u_b^2}{g \cdot \Delta m \cdot \cos \alpha}$$

- (3) **Jamuna-Formula**

$$D \geq 0.85 \cdot D_{50} \geq \frac{0.7 u_{cr}^2}{\Delta m \cdot 2 \cdot g}$$

- (4) **Modified DELFT-Formula**
(Rip-rap)

$$D_{50 \text{ fa}} \geq \sqrt{\frac{19 \cdot R \cdot D_{50 \text{ soil}}}{c_0 \cdot 0.8}}$$

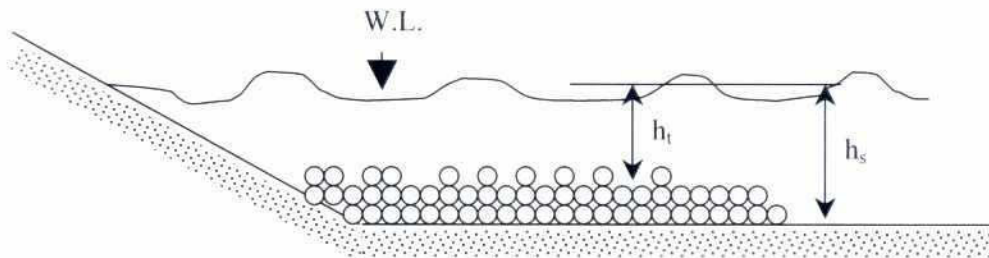
(ii) Toe Protections / Falling Aprons under Wave Attack

Only one formula was available at the time of preparing the designs for the test structures, namely the

PIANC-Formula (1992)

$$D_{50} \geq \frac{H_s}{\Delta m \left[1.14 + 0.03 \frac{h_t}{h_s} + 8.38 \left(\frac{h_t}{h_s} \right)^2 \right]}$$

With the limitations $0.4 < h_t/h_s < 0.8$ as per definition in the following sketch:



For the reason explained under para (d) of Subsection 3.3.3, the Preliminary Delft Formula was applied for flow conditions to design the falling aprons of concrete blocks.

For locations where, dependent on the prevailing water level, falling aprons were expected to be subjected to wave loads the PIANC - Formula (1992) was applied. For those areas subjected to wave loads and flow attack of $u \geq 1$ m/s, the cube size was selected by multiplying the larger block size obtained by the flow and wave formulas respectively by factor 1.3, in line with PIANC-recommendation.

Sample computations are presented in Attachment 3 to this Design Report.

Besides the dimensioning of individual block sizes to be used in a toe protection/falling apron also the area size and thickness of the protection has to be decided. At the time of designing the test structures some tentative approach was developed, which is based on the assumption that a developing scour slope must be stabilised with certain coverage of protection material. The assumptions and interrelationship are presented in Fig. 3.5-1. However, attention should be given to the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 0), where extended design rules are being discussed and presented for future standard designs.

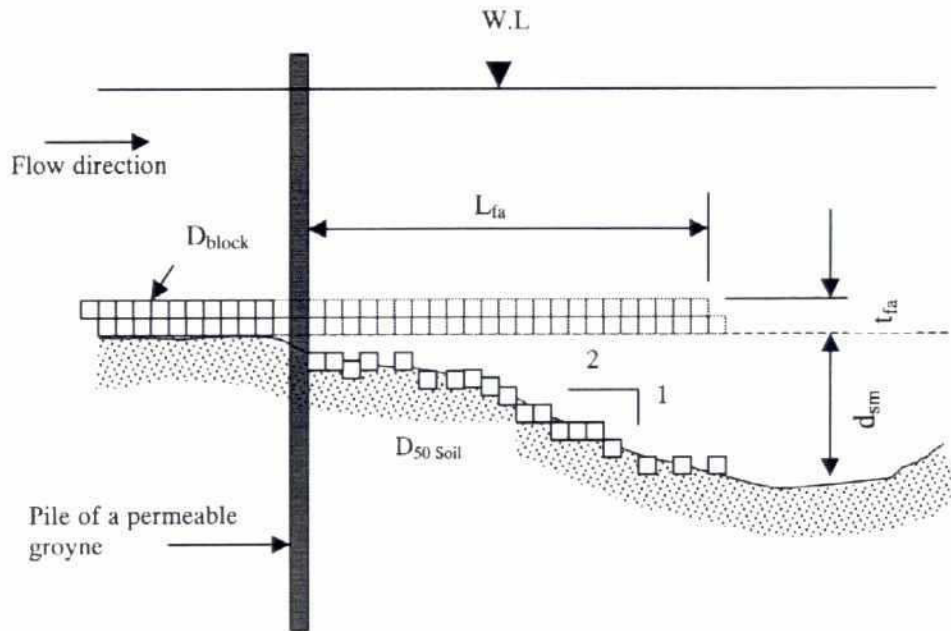


Fig. 3.5-1: Definition of toe protection / falling apron dimensioning

In Fig. 3.5-1 the following annotations are used:

- d_{sm} = design depth of scour hole;
- t_{fa} = thickness of falling apron
 - = $3.5 \cdot D$ (randomly placed cubes)
 - = $4.5 \cdot D$ (randomly placed, where turbulence are expected, e.g. at upstream/downstream terminations, transitions, etc.)
- D = design size of cube/stone in the apron (D_{50})
- $D_{50 \text{ soil}}$ = size of subsoil grain (since settlement i.e. falling of the blocks is permitted, the grain size of subsoil can be assumed at minimal 2 mm, i.e. blocking grain is not desired)
- L_{fa} = length of falling apron as per following formula:

$$L_{fa} \geq 2.0 \cdot d_{sm} \quad 3.5-1$$

3.5.3 Bed Protections

Bed protections were designed analogously to falling aprons, at least with regard to the determination of a single weight of a protection unit. Contrary to falling apron behaviour, with a bed protection it is anticipated that the under-laying filter material prevents substantial seepage of subsoil, which would

ensure a relatively stable position of the bed protection, with only the exposed sides launching-out to follow the scouring of the unprotected river bed.

For the application of bed protections in rivers such the Jamuna enormous construction constraints exist for placing filters below water level at relatively high flow velocities, at least when employment of specially designed and expensive construction equipment shall be avoided (as in the case of this Project). Annex 5 (Consulting Consortium FAP 21/22, 2001 f) reports on these issues.

4 LESSONS LEARNED SINCE CONSTRUCTION

During the monsoon flood of 1995 damage was caused at the transitions between the impermeable and permeable parts of the groynes G-2 and G-3, leading to partial collapse of the earthen dams and a few piles. The detailed analyses of the morphological as well as planning and design aspects related to the damage are presented in the Report on Monitoring and Adaptation at Kamarjani Test Site, Monsoon 1995 (Consulting Consortium FAP 21/22, 1996 c) and Annexes 6 and 7 (Consulting Consortium FAP 21/22, 2001 g and h).

This Section only highlights the engineering design aspects related to the observations made and lessons learned during the first test season after construction of the groynes in 1994/95, since these experiences have the most important influence regarding development of future standard designs. For reference Table 4-1 shows, compares and comments the engineering parameters.

	Unit	Design Value	Observed Maximum Value	Comments
Flood level	m + PWD	22.90	22.38	0.5m below design value
River bed level at groyne head	m + PWD	5.20		-
Critical bed level at groyne head	m + PWD	- 5.0	≈ - 5.0	well considered in the pile design
Flow velocity near groyne head	m/s	3.2 and 2.9	≈ 2.9	well predicted
Flow direction	degrees	0 (parallel)	20 (oblique)	unpredicted
Return current along river bank	m/s	0.5	0.8 to 1.0	higher, was not evident from modelling
Wave height [$H_{1/3}$]	m	1.0	0.55	obviously conservative assumption
Wave period [T]	s	3.0	-	not determined by monitoring
Floating debris (layer thickness)	m	1.0	> 1.5 (assumed)	effects to be further studied
Scour depth related to pile design (below design river bed)	m	4	≈ 6	under estimated
Maximum scour downstream from groyne head:				
Scour depth:	m	6 to 7	8 to 9	well predicted
Distance downstream of groyne:	m	30 to 60	20 to 50	well predicted

Table 4-1: Comparison of design parameters versus observed values

Apart from the engineering design parameters the following conclusions are drawn from the monitoring observations, which should suitably be considered in any future standard design:

- (a) Eddy formation just downstream from the impermeable groyne heads has been much more severe than anticipated as per physical modelling (it is to note that these phenomena could not be reproduced in the physical model of scale 1 : 75 and 1: 60 respectively at RRI).

Appropriate measures are to be designed

- to reduce, rather to avoid these eddies by modification of the transition between the impermeable and permeable groyne section, and/or

- to extend the launching and falling aprons just downstream from the impermeable groyne head to cope with the consequent scouring.
- (b) Return currents between the groynes have been almost twice the predicted value and have caused excessive erosion up to/into the main embankment.
- to prevent failure of the main flood embankment the distance between the location of the expected deep scour downstream from the permeable groyne head and the toe protection along the main flood embankment must be increased.
 - as a safe slope between the deepest expected scour and the toe of the embankment an angle corresponding to 1V : 7H may be considered, with at least a 5 m wide berm along the embankment toe.
- (c) Configuration of the groyne field, as are determined by length, spacing and orientation of the groynes, must suit also variable approach directions of current flow.
- (d) Floating debris (water hyacinths) had contributed to the development of additional scour depth, however, the presence of such floating debris can not be prevented.

Submerged groynes could avoid detrimental effects at least during high flood situations.

- (e) Sheet-pile cofferdams instead of earth dams within the impermeable part of a groyne are vulnerable to excessive scouring. The cofferdam compartment facing the river should be designed for twice the predicted scour depth.
- (f) For steel pile groynes the use of scour protection by bed protections or falling aprons (refer to definitions under Subsection 3.5.1) is considered superfluous. For economical reasons and the aspects of easy and short construction works it is more advantageous to increase the pile length appropriately rather than to dump substantial quantities of heavy protection materials.

Comparison between behaviour of groyne G-3 (without any bed protection) and groyne G-2 (falling apron) as well as groyne G-1 (bed protection) has proven that the increase of the pile embedment length presents a higher reliability for the structure stability. The economical aspect thereof is discussed in Annex 12 (Consulting Consortium FAP 21/22, 2001 m). A comparison of monitored scour depths downstream from the permeable groyne heads is presented in Fig. 4-1.

- (g) In groyne sections with permeability of less than 50 %, the clearance between the individual piles becomes marginal, which ultimately could lead to problems during pile installation. For the very reason, the piles of the original test structures were staggered in depth within the 50 % - portion. Thereby it was assumed that those piles, which had been connected by steel girders to the adjacent pile would carry-off the hydraulic loads even with a minimum embedment length of about 3 m. Experience in the first monitoring season has shown that due to the unexpected high scouring at the head of the impermeable groyne heads, the full embedment length of all piles is essential to retain the integrity of the structure also under unfavourable circumstances. Experience during the construction has also proven that the installation of piles to full penetration depth does not cause a constraint as long as the so-called pilgrim-method is applied during installation (refer to Annex 5, Consulting Consortium FAP 21/22, 2001 f).

- (h) The slope protection of the main flood embankment by Durba grass is a suitable measure to prevent surface erosion by rain, wherefore the land-sided slopes should always be provided with properly laid and nursed grass sods.

For the river-side of the main flood embankment the application of Durba grass sods need to be improved to ensure sufficient growth and rooting before the high flood season. Additional protection should be provided by Vetiver plantation along the embankment toe as well as along two or three parallel lines between toe and crest. The roots of Vetiver plants grow very fast and penetrate deep into the soil within short time provided they are pampered thoroughly after planting. The Vetiver plants will serve as a kind of wave breaker and protect also the grass from wave erosion.

To achieve the aim it is important to lay the grass sods and to plant the Vetiver as early as possible and to ensure proper nursing until the rainy season has started.

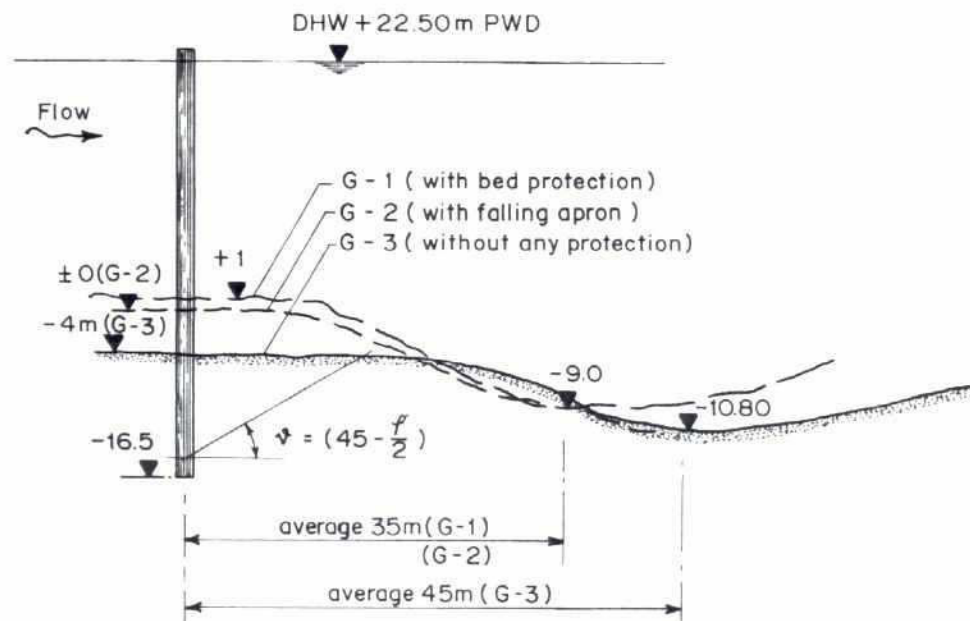


Fig. 4-1: Comparison of monitored scouring downstream from permeable groyne heads

There are two important matters to be realised for any further consideration:

- firstly, the instabilities of the subsoil exposed to the hydraulic forces of the river have contributed to the failures to a very considerable extent. However, the subsoil conditions are an unavoidable fact, wherefore all measures must be carefully adopted to suit the particular subsoil characteristics, which includes optimising of the
 - configuration of the groyne field determined by groyne length, spacing and orientation;
 - transition between impermeable and permeable groyne section;
 - blockage and permeability respectively between groyne head and root;
 - distance between river-most groyne head and main flood embankment, and
 - scour protection measures at exposed locations.
- secondly, the response of the river in the seasons following the construction of the test groyne field is a typical example that any artificial interference with the natural flow of a river demands supplementary measures to compensate for the river's response. Such measures must be seen as an integral element of the project and should not be separated from it.



5 DESIGN OF ADAPTATION WORKS

5.1 PHYSICAL MODEL TESTS 1995/96

After the monsoon flood of 1995 additional physical model tests were carried out at RRI to support the design of adaptation measures for the groyne field at Kamarjani. The model test results are compiled in Technical Report No. 5 (Consulting Consortium FAP 21/22, 1996 a). The investigated improvements and adaptation measures had to fulfil the following conditions:

- to substantially reduce the magnitude of return currents and vortices within the groyne field in particular along the main embankment, and
- to improve the transition between the impermeable and the permeable sections of the groynes with the aim to further reduce the development of severe return currents, turbulence and vortices.

Based on the results of the physical model tests it was considered that the above aims could be best achieved with the following adaptation measures to the existing structures:

- reducing the slope of the impermeable groyne head to 1V : 5H;
- introducing a step along the centre crest of the slope line of about 1 m height;
- adjusting the permeability of the pile structure towards the head of the impermeable groyne at a rate of 50-60-65-75-70-40 %;
- extending the distance between the deepest scour downstream from the permeable groyne head and the toe of the main flood embankment, and
- increasing the roughness of the flood plain between the groynes by providing stone dumps, stone-filled gabion sacks or natural plantation, such as Vetiver.

Supplementary to the above, it was considered essential to supplement the groyne field by a secondary groyne at the downstream end of the groyne field, since it was anticipated that the flow attack would move further downstream during the forthcoming season, with a threat to the main flood embankment near the Ghagot River mouth.

The adaptation works for Kamarjani Test Site were planned accordingly, whereby the applied design parameters for dimensioning structural elements correspond to the originally assumed ones. The following drawings present the suggested adaptations to the groyne structures and are attached to this report (see Attachment 4).

Drawing No. AD-KA-020 Rev.2:	Groyne G-2; General Layout Plan of Modified
Drawing No. AD-KA-021 Rev.4:	Groyne G-2; Pile Layout Plan (Modified Permeability)
Drawing No. AD-KA-023/1 Rev.1:	Groyne G-2; Detailed Layout of Modified Groyne
Drawing No. AD-KA-024 Rev. 0:	Groyne G-2; Cross-Sections of Modified Groyne Head
Drawing No. AD-KA-050 Rev. 3:	Groyne G-A/2; General Layout Plan
Drawing No. AD-KA-051 Rev. 2:	Groyne G-A/2; Pile Layout Plan

It is to be noted that the adaptation works could not be carried out and completed in 1996 as scheduled, due to late delivery of construction materials. In particular the modification of the impermeable groyne head could not be implemented, but the groynes G-2 and G-3 were finally converted to fully permeable groynes in 1996 and 1997.



5.2 PHYSICAL MODEL TESTS 1997/98

Supplementary to Section 5.1 ten physical model tests were carried out at RRI at a scale of 1:55 in an open air model, representing a stretch of 1.2 km of the right bank of Jamuna river. The objectives of these additional tests were to investigate into the causes of damages observed in 1995, and to gain more knowledge about the configuration optimisation of groyne fields towards formulation of design rules.

The model tests resulted in the following recommendations to be considered in future designs:

- the combination of impermeable and permeable groyne sections within one structure should be avoided, since the resultant return currents and vortices are not manageable;
- the orientation of groynes should be at an angle of 15° pointing upstream;
- the ration between spacing and total length of the groynes is recommended at 2 to 2.5;
- the slope revetment of the main embankment along the groyne field should be designed to withstand a flow velocity of 1 m/s;
- the permeability for a proposed 105 m long groyne should be (from embankment towards the river) 40 % (37.5m), 50 % (24m), 60 % (14,5m), 70 % (14.5m) and 80 % (14.5m);
- the crest elevation should be submerged or gradually submerging (from the bank towards the river) to limit the risk of blockage by floating debris, and
- the model results also suggested the use of falling aprons around the piles, extending up to the top of the main embankment.

The above recommendations will be further elaborated in the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o) should be consulted for any future planning and design.

6 RECOMMENDATIONS FOR STANDARD GROYNE DESIGNS

6.1 INTRODUCTION

As already mentioned elsewhere in this Design Report the Guidelines (Consulting Consortium FAP 21/22, 2001 n) and the Design Manual (Consulting Consortium FAP 21/22, 2001 o) present the essential recommendations and tools for the planning and design of permeable groyne structures. Therefore, the respective documents should be consulted for any future work.

The following Sections and Subsections present, however, a simplified graphic method for dimensioning the individual piles of a permeable groyne structure.

6.2 DESIGN GRAPHS FOR A SINGLE GROYNE

6.2.1 Basic Assumptions

The design parameters used for dimensioning the groyne pile structures at Kamarjani Test Site are detailed under Sections 2 and 3. Numerous computations carried out subsequently have indicated that for standardisation certain simplifications can be introduced to limit the variable parameters to be introduced in the designs.

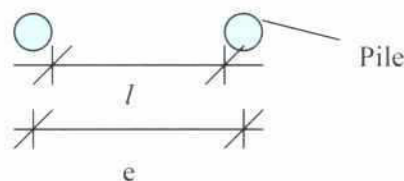
With a view to the above, the following design parameters have been streamlined for preparing the pile design graphs:

- subsoil:

angle of internal friction:	ϕ'	=	27.5°
cohesion:	c'	=	0
submerged unit weight :	γ	=	10 kN/m ³
- permeability of groyne:

hydraulic load considered for 50 %, irrespective of actually defined permeability V

$$V = l / e \geq 50 \%$$



- design water depth: the result of Design High Water level minus design river bed level plus design scour depth.
- flow velocity: $v = 1.5$ m/s up to 4.0 m/s (for the purpose of defining the pile design graphs).
- floating debris: $h_f = 0.5$ m up to 1.0 m, dependent on loading classes.
- wave height: $H_{\text{wave}} = 0.6$ m to 1.0 m dependent on loading classes.
- Loading Classes: Loading Classes LC1, LC2 and LC3 are considered as per Subsection 3.2.3 were analysed in various scenarios.

It turned out that for LC1 and LC2 the necessary embedment length decreases with increasing permeability of the groyne.

The difference between the maximum and minimum embedment length is merely ~ 50 cm so that the results for $V = 50\%$ are used for the pile design diagrams. As for LC3, the necessary embedment length has its minimum around $V = 70\%$ and its maximum at $V = 90\%$. The reason is the increased horizontal load per pile initiated by floating debris for this case. Therefore the results for $V = 90\%$ are relevant in case of LC3.

A comparison of the results of the 3 load cases showed that LC2 with $H_{\text{wave}} = 1.0$ m and the same flow velocities as in the other load cases leads to the highest loads. Therefore, the results of LC2 were taken into account for the preparation of the simplified design diagrams.

The results of $\text{LC2} = v + (H_{\text{wave}} = 1.0\text{m})$ are comparable to a combined loading of

$\text{LC4} = v + (H_{\text{wave}} = 0.5\text{m}) + (h_t = 1.0\text{m})$, and

$\text{LC5} = v + (H_{\text{wave}} = 0.7\text{m}) + (h_t = 0.5\text{m})$ respectively.

- safety factors: in line with the strategy of the Project the safety factors for the loads are reduced to 1.0.
- pile embedment length: the calculation of the embedment length l_E was done according to EAU 1990, restricting the minimum embedment length to $l_E = 5.0$ m.

6.2.2 Tubular Steel Pile Structure

The tubular steel pile design diagram presented in Fig. 6.2-1 has been developed in consideration of the variables stipulated under Subsection 6.2.1 and the following:

- for the dimensioning of tubular steel piles DIN 18800 was applied;
- steel grade corresponding to RSt 37-2 ($f_{y,k} = f_{y,d} = 240$ N/mm²) or equivalent quality, and
- for buckling and stability problems during pile installation the thickness of the tubular steel pile walls has been set to be at least 1/100 of the respective pile diameter.

The use of the diagram is self-explanatory and applies to any groyne permeability between 50 % and 90 %.

6.2.3 Reinforced Concrete Pile Structure

The diagram for reinforced concrete pile design presented in Fig. 6.2-2 has been developed in consideration of the variables stipulated under Subsection 6.2.1 and the following:

- the reinforcement design was done according to Eurocode 2 with safety factor $\psi=1.0$ for the loads;
- Concrete Class B35, DIN 1045, has to be specified for pile construction. However, for design purpose only Class B 25 with a strength of $f_{ck} = 25$ N/mm² has been considered in the calculations to give consideration to executional deficiencies during in-situ pile construction below ground water level, and

- reinforcing steel Grade 60 ($f_{y,k} = 414 \text{ N/mm}^2$) has been considered for the design graphs. The maximum percentage of reinforcement should not exceed about 2 % of the pile's cross-section in order to guarantee sufficient flow of concrete around the steel bars during concreting.

The use of the diagram is self-explanatory and applies to any groyne permeability between 50 % and 90 %.

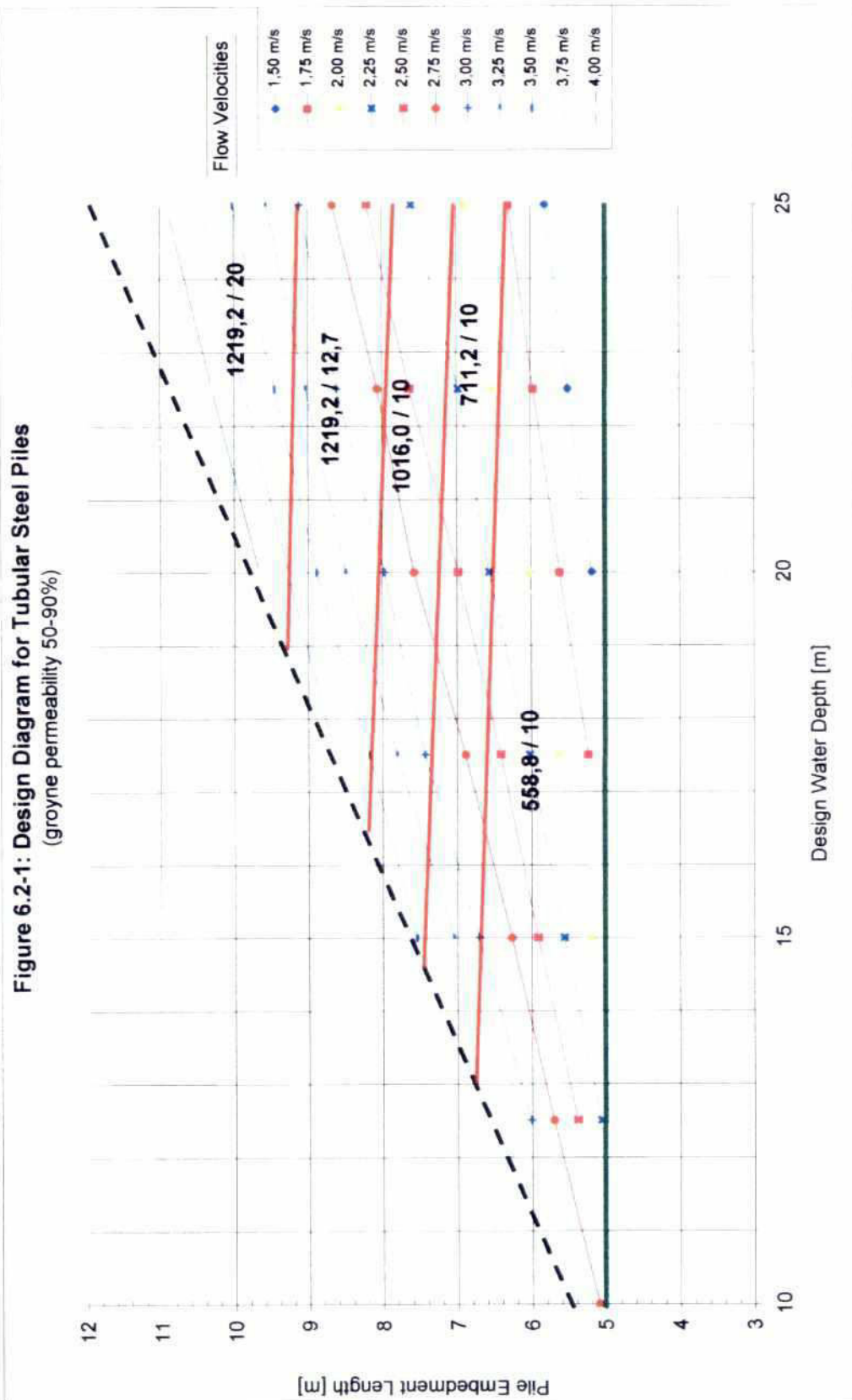


Fig. 6.2-1: Design diagram for tubular steel piles
(Groyne Permeability 50 % to 90 %)

Figure 6.2-2: Design Diagram for Reinforced Concrete Piles

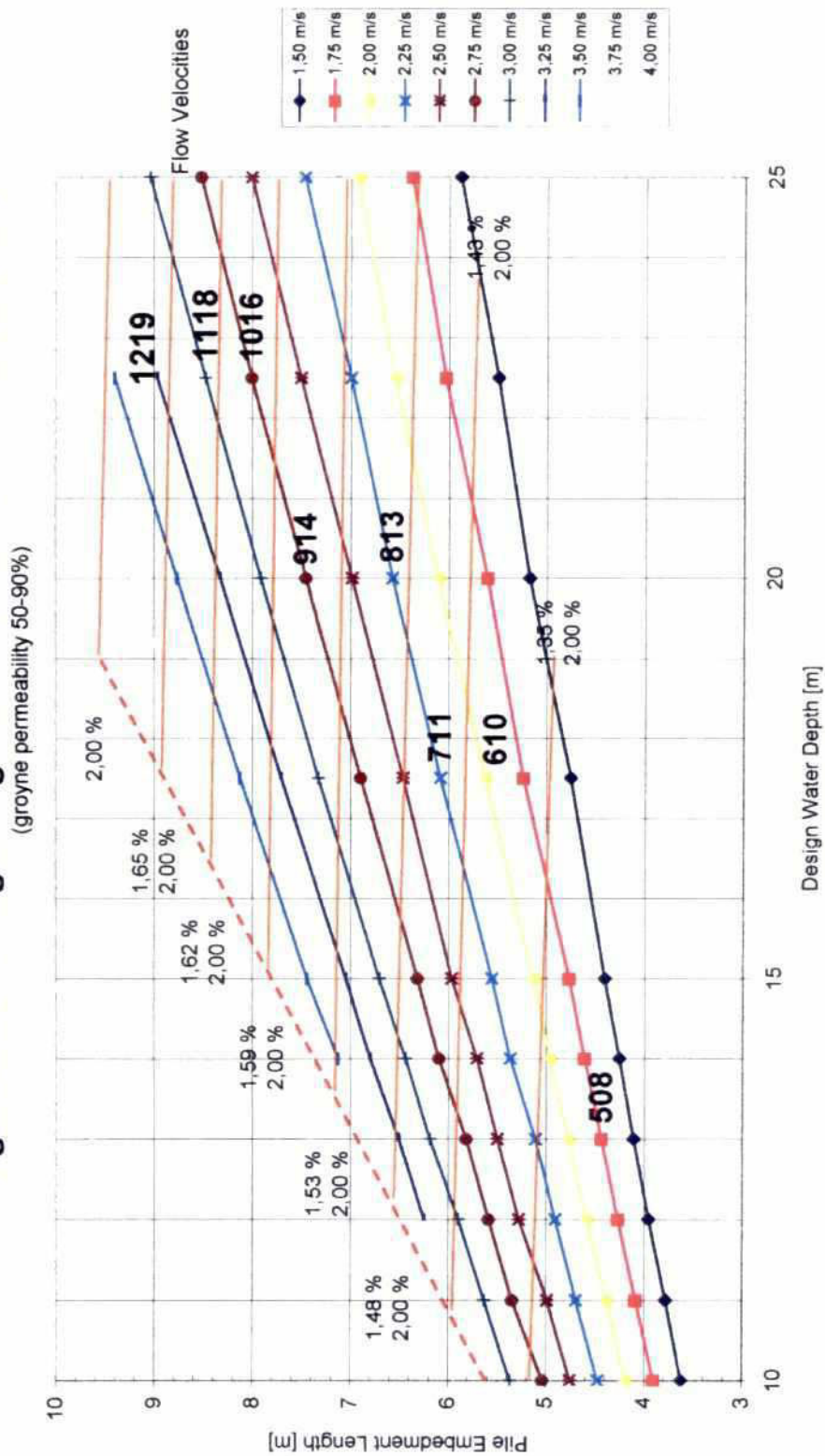


Fig. 6.2-2: Design diagram for reinforced concrete piles (groyne permeability 50 % to 90 %)

6.3 CALCULATION METHOD

6.3.1 Calculation of Loads

(a) Loading Case 1 - Current

With the construction of the groyne the natural flow area of the river is narrowed. This leads to a local lifting of the water level upstream from the piles. With the assumption that there is always a streaming flow of water the increase in the water level can be calculated with the formula by REHBOCK:

$$h_s = \alpha \cdot [\delta - \alpha \cdot (\delta - 1)] \cdot (0.4 + \alpha + 9 \cdot \alpha^3) \cdot \left(1 + \frac{u_2'^2}{g \cdot d}\right) \cdot \frac{u_2'^2}{2 \cdot g} \quad (6.3-1)$$

where

α	=	D / e	(-)
D	=	diameter of pile	(m)
e	=	distance between pile axes	(m)
δ	=	shape coefficient for round piles	
	=	2.10	
d_2	=	design depth i.e. the undisturbed water level downstream	(m)
u_2'	=	average design velocity i.e. the undisturbed velocity downstream	(m/s)

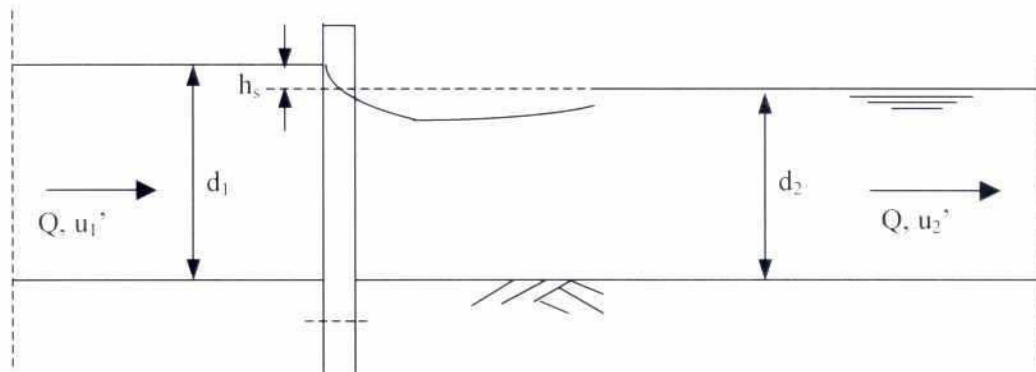


Fig. 6.3-1: Increased water level in front of a pile structure

This leads to:

$$d_1 = d_2 + h_s \quad (6.3-2)$$

With $Q = d_1 \cdot u_1' = d_2 \cdot u_2'$ the reduced average upstream flow velocity can be calculated:

$$u_1' = \frac{u_2' \cdot d_2}{d_1} \quad (6.3-3)$$

The course of the flow velocity with increasing water depth can be calculated with the following formula:

$$u_{l(z)} = u_1' \cdot \frac{\ln\left(30 \cdot \frac{d_1 - z}{k_s}\right)}{\ln\left(11 \cdot \frac{d_1}{k_s}\right)} \quad (6.3-4)$$

where:

$u_{l(z)}$	=	flow velocity in the considered depth	(m/s)
z	=	distance of the considered depth from the water level	(m)
u_1'	=	average (upstream) flow velocity	(m/s)
k_s	=	coefficient of roughness for the river bed	(-)

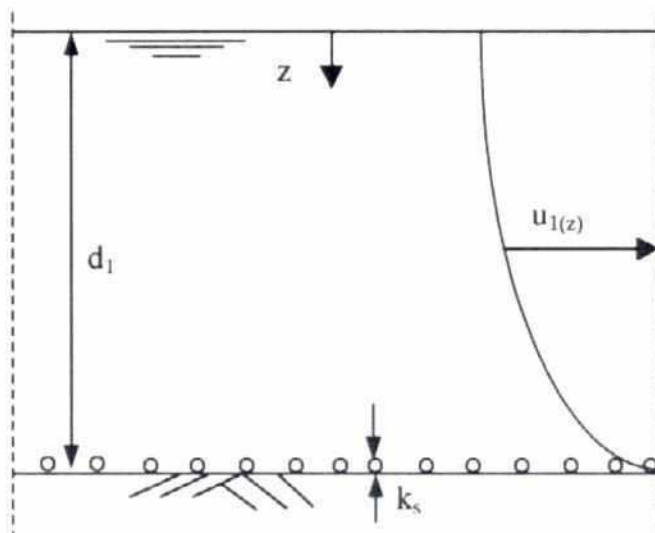


Fig. 6.3-2: Course of flow velocity with the depth

The load on the pile resulting from the flow velocity can be calculated by the following formula (see EAU 1990, R 159):

$$p_{(z)} = C_D \cdot 0.5 \cdot (\gamma_w / g) \cdot D \cdot u_{l(z)}^2 \quad (6.3-5)$$

where

C_D	=	coefficient taking into account the shape and the surface friction of the pile, the viscosity and the turbulence of the fluid	
	=	0.7	(-)
γ_w	=	density of water	(kN/m ³)
g	=	acceleration due to gravity	(m/s ²)
D	=	diameter of pile	(m)

(b) Loading Case 2 - Current and Wave

For deep water conditions ($d / L > 0.5$) as it is the case regarding the groynes, AIRY'S linear wave theory (first order) can be used to calculate the velocities and accelerations of water particles (see EAU 1990, R 159):

$$u = \frac{H}{2} \cdot \omega \cdot e^{k \cdot z} \cdot \cos \vartheta \quad (6.3-6)$$

$$\frac{du}{dt} = \frac{H}{2} \cdot \omega^2 \cdot e^{kz} \cdot \sin \vartheta \quad (6.3-7)$$

where

d	=	water depth	(m)
u	=	horizontal component of the orbital velocity of the water particles at studied pile location	(m/s)
du / dt	=	horizontal component of the orbital acceleration of the water particles at studied pile location	(m/s ²)
H	=	wave height	(m)
t	=	time duration	(s)
T	=	wave period	(s)
ω	=	wave angular frequency	(1/s)
	=	$\frac{2 \cdot \pi}{T}$	
L	=	wave length	(m)
	=	$\frac{g \cdot T^2}{2 \cdot \pi}$	
e	=	base of the natural logarithm	
	=	2.718...	
k	=	wave number	(1/m)
	=	$\frac{2 \cdot \pi}{L}$	
z	=	ordinate of the investigated point	
z	=	0 (still water table)	
ϑ	=	phase angle	
	=	$k \cdot x - \omega \cdot t$	
x	=	abscissa of the investigated point	

The resulting load on the piles is calculated with MORISON'S formula, because the condition $D / L \leq 0.20$ is fulfilled (see CIRIA Report *Dynamics of Marine Structures*):

$$p = p_D + p_M = C_D \cdot \frac{1}{2} \cdot \frac{\gamma_w}{g} \cdot D \cdot u \cdot |u| + C_M \cdot \frac{\gamma_w}{g} \cdot \frac{D^2 \pi}{4} \cdot \frac{\partial u}{\partial t} \quad (6.3-8)$$

where

p_D	=	pressure due to the water particle velocity caused by the flow resistance per unit length of pile	(kN/m)
p_M	=	inertial pressure due to instationary wave movement per unit length of pile	(kN/m)
C_D	=	drag coefficient taking into account the resistance of the pile against the flow pressure	
	=	0.7	(-)
C_M	=	inertia coefficient taking into account the resistance of the pile against the acceleration of water particles	
	=	2.0	(-)

g	=	gravity acceleration	
	=	9.81	(m/s ²)
γ_w	=	density of water	
	=	10	(kN/m ³)

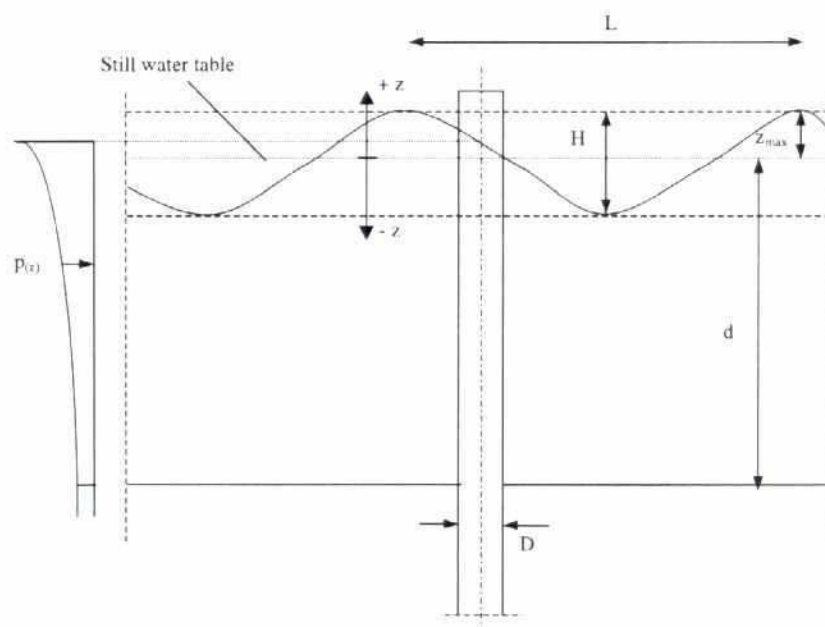


Fig. 6.3-3: Wave load on piles

Correction factors β for wave load on pile clusters:

e / D	2	3	4
β	1.5	1.25	1.0

This increase of the pile loads is comparable to REHBOCK'S formula applying to current loads on pile clusters (refer to Subsection 6.3.1).

(c) Loading Case 3 - Current and Floating Debris

The formulas considering the increase of the water level by means of floating debris resulted in a few millimetres additional water depth. Therefore, they were neglected in the subsequent calculations. Besides the load of the current onto the piles (formulas have already been given in para (a) of Subsection 6.3.1 dealing with LC1) the horizontal load F_h (kN) of a heap of debris has to be considered for this load case. This load is dependent on the distance e between the piles, the height h_d of the heap and the maximum current velocity u_{max} at this place.

$$F_h = 0.5 \cdot (\gamma_w / g) \cdot e \cdot h_d \cdot u_{max} \quad (6.3-9)$$

6.3.2 Calculation of the Pile Embedment Length

Behind a single pile loaded by a single horizontal force a passive earth pressure slip wedge develops. This results in a non-linear increase of the earth pressure. According to BLUM the width of the slip wedge can be assumed as the width of the pile plus its embedment length t_E . If there are several piles

as in the case of a groyne, the slip wedges touch when the distance between the piles is equal to the width of the slip wedges (see Fig. 6.3-4).

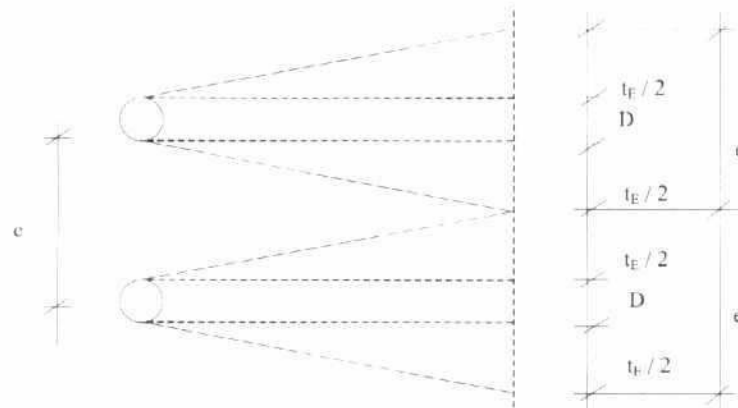


Fig. 6.3-4: Top view, slip wedges behind piles

This case occurs when the embedment length t_E reaches the critical depth t_{crit} : $t_{crit} = e - D$

Beyond this depth the earth pressure cannot increase non-linear any more and the assumption of a spatial passive earth pressure is substituted by the assumption of a planar passive earth pressure acting on the width e (see Fig. 6.3-5).

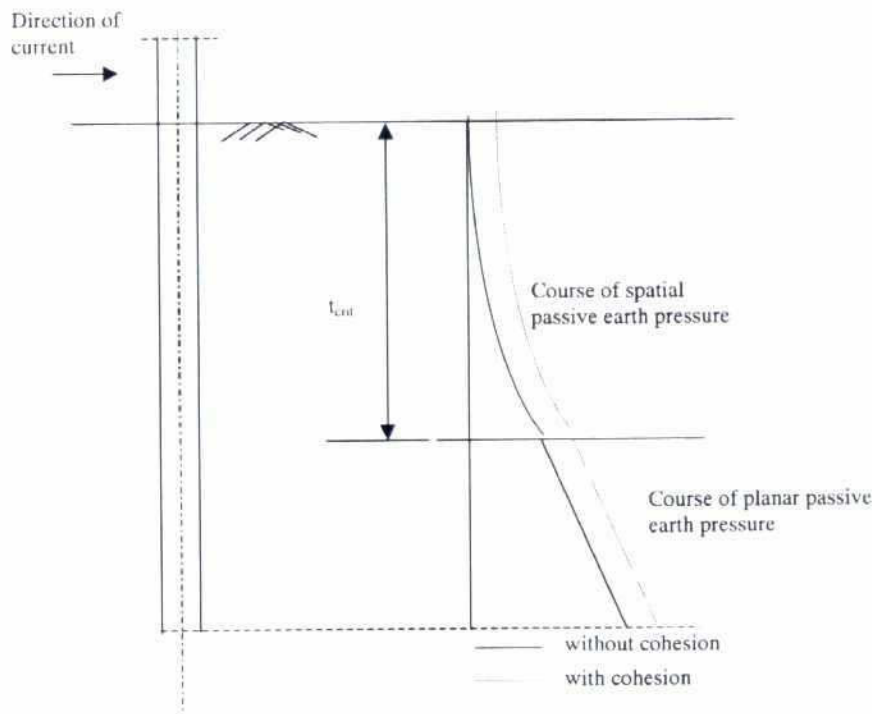


Fig. 6.3-5: Section, course of earth pressure behind piles

The spatial passive earth pressure coordinates are determined according to DIN V 4085 – 100:

$$s_{pa} e_{ph} = \gamma' \cdot t \cdot s_{pgh} \cdot K_{pgh} \cdot D + c' \cdot s_{pch} \cdot K_{pch} \cdot D \quad (6.3-10)$$

where

$$\begin{aligned} \gamma' &= \text{submerged density of soil} & (\text{kN/m}^3) \\ c' &= \text{cohesion in drained state of soil} & (\text{kN/m}^2) \end{aligned}$$

The angle of wall friction of passive earth pressure is assumed to be $\delta_p = 0$ according to EAU 1990. That results in the following horizontal passive earth pressure coefficients:

$$K_{pgh} = \tan^2(45^\circ + \varphi' / 2) \quad (6.3-11)$$

$$K_{pch} = 2 \cdot \sqrt{K_{pgh}} \quad (6.3-12)$$

where

$$\varphi' = \text{effective angle of internal friction} \quad (\text{degree})$$

The shape coefficients s_{pgh} and s_{pch} depend on the ratio t / D :

	$t / D < 3.33$	$t / D \geq 3.33$
$s_{pgh} =$	$1 + 0.45 \cdot \frac{t}{D}$	$1.37 \cdot \sqrt{\frac{t}{D}}$
$s_{pch} =$	$1 + 1.80 \cdot \frac{t}{D}$	$3.29 \cdot \sqrt{\frac{t}{D}}$

When the critical embedment length t_{crit} is reached, the planar passive earth pressure is calculated according to the following formula:

$$\Delta e_{ph} = \varphi' \cdot K_{pgh} \cdot D \cdot (t - t_{crit}) \quad (6.3-13)$$

The calculation of the embedment length is done analogously to the calculation method for dolphins by BLUM.

First the resulting horizontal force Q deriving from current, waves or drifting plants is calculated. Q causes a bending moment in the pile that is reduced with increasing embedment by the resisting passive earth pressure. The theoretical embedment length t_0 is calculated with the condition that the resulting bending moment in the pile reduces to $\Sigma M = 0$. This point is called theoretical point F.

The end restraint of the pile in the subgrade can only be achieved when the resultant of all horizontal forces becomes zero: $\Sigma H = 0$.

This is possible by means of the equivalent force C that results in a necessary extra length Δt of the pile required for the absorption of C . According to EAU, R56, the extra length can be determined with the following formula:

$$\Delta t = \frac{C}{2 \cdot \gamma' \cdot t_0 \cdot K_{ph} \cdot \cos \delta_p} \quad (6.3-14)$$

With $\delta_p = 0$ this formula simplifies to the following equation:

$$\Delta t = \frac{C}{2 \cdot e_{ph,t}} \quad (6.3-15)$$

The necessary embedment length is the sum of the theoretical embedment length and the extra length:

$$t = t_0 + \Delta t \quad (6.3-16)$$

The maximum bending moment of the pile can be found by the condition that the shear force has to be zero at that point.

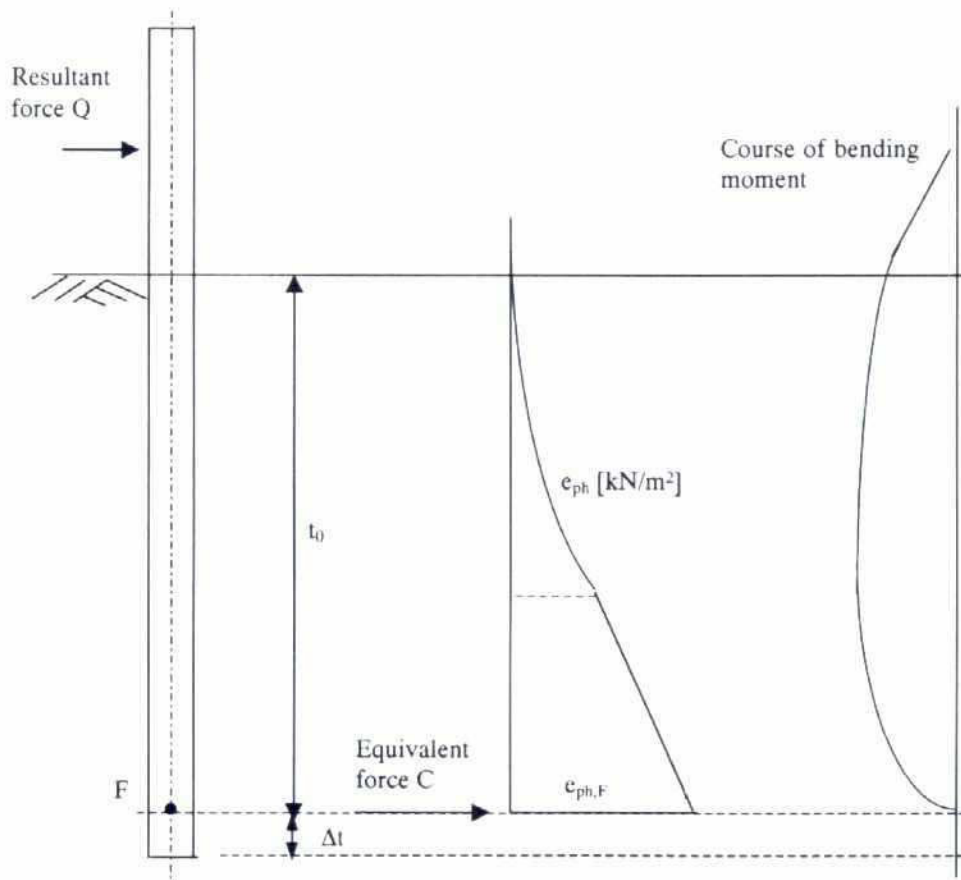


Fig. 6.3-6: Determination of the embedment length

6.3.3 Influence of Varying Design Parameters on the Result

(a) Angle of Internal Friction ϕ' and density γ

The calculation for the diagrams was done with $\phi' = 27,5^\circ$. The assumption of $\phi' = 30^\circ$ leads to a reduction of 3.5 % of the embedment length, whereas $\phi' = 32.5^\circ$ results in 6.5 % reduction of the embedment length. The dimension of the pile remains the same, because higher angles of internal friction mean lower maximum moments.

An increase of the density of the subsoil results in lower embedment lengths of the piles:

From $\gamma = 9 \text{ kN/m}^3$ to $\gamma = 10 \text{ kN/m}^3$: $\Delta t \approx - 3.4 \%$

From $\gamma = 10 \text{ kN/m}^3$ to $\gamma = 11 \text{ kN/m}^3$: $\Delta t \approx - 3.2 \%$

From $\gamma = 11 \text{ kN/m}^3$ to $\gamma = 12 \text{ kN/m}^3$: $\Delta t \approx - 3.0 \%$

The influence of varying ϕ' and γ respectively was considered negligible and therefore not considered in the design diagrams.

(b) Permeability

As it was said elsewhere, the permeability has also influence on the embedment length of the piles. The following two diagrams show the embedment length at constant water depth but changing permeability and current velocity. It can be seen that:

- for LC1 and LC2 the embedment length reduces with growing permeability;
- for LC3 the embedment length has its minimum at $V = 70 \%$ and its maximum at $V = 90 \%$, and
- the difference in embedment length between $V = 50 \%$ and $V = 90 \%$ is higher with higher current velocities and water depths

The design diagrams presented in Fig. 6.2-1 and 6.2-2 were produced with the results for groynes with a permeability of 50 % and LC2. A glance on the diagrams presented in Fig. 6.3-7 and 6.3-8 shows that this assumption is justified and on the safe side because the embedment length of LC2 with $V = 50 \%$ is higher than the embedment length of LC3 with $V = 90 \%$.

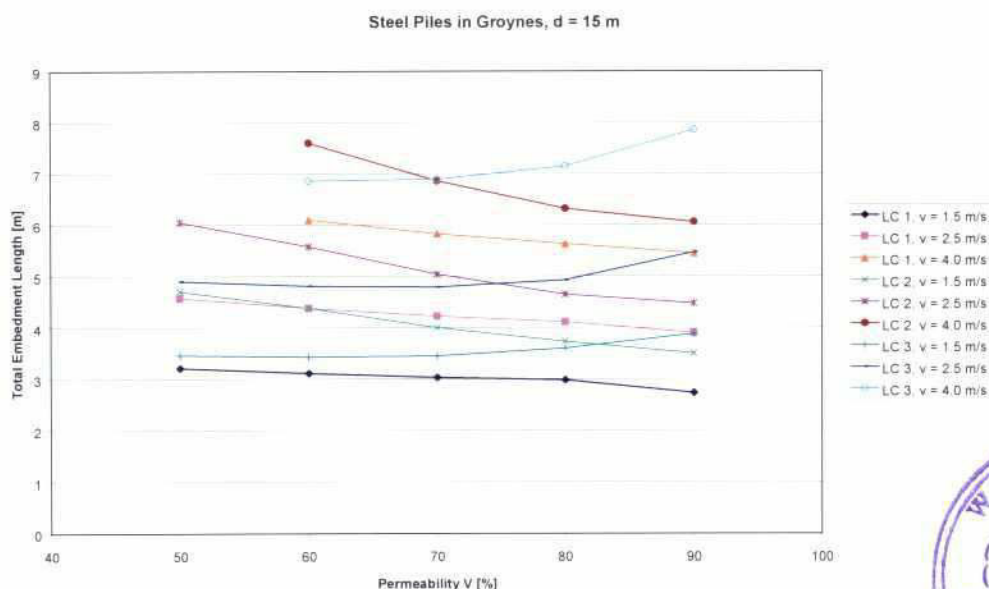


Fig. 6.3-7: Embedment length versus permeability of groyne (15 m water depth)

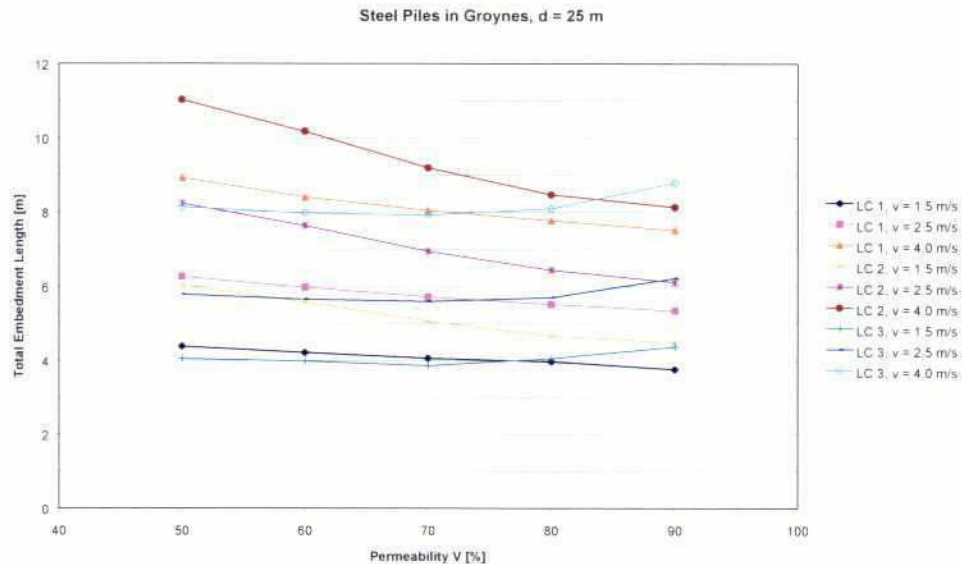


Fig. 6.3-8: Embedment length versus permeability of groyne (25 m water depth)

However, if submerged groynes are designed, LC3 can not occur because floating debris would not be intercepted by the groyne. For this case it is advisable to consider the change of embedment length depending on permeability and current velocity with a correction term.

The following Fig. 6.3-9 is derived from the last two diagrams and gives the reduction of the embedment length for LC2 with increasing permeability.

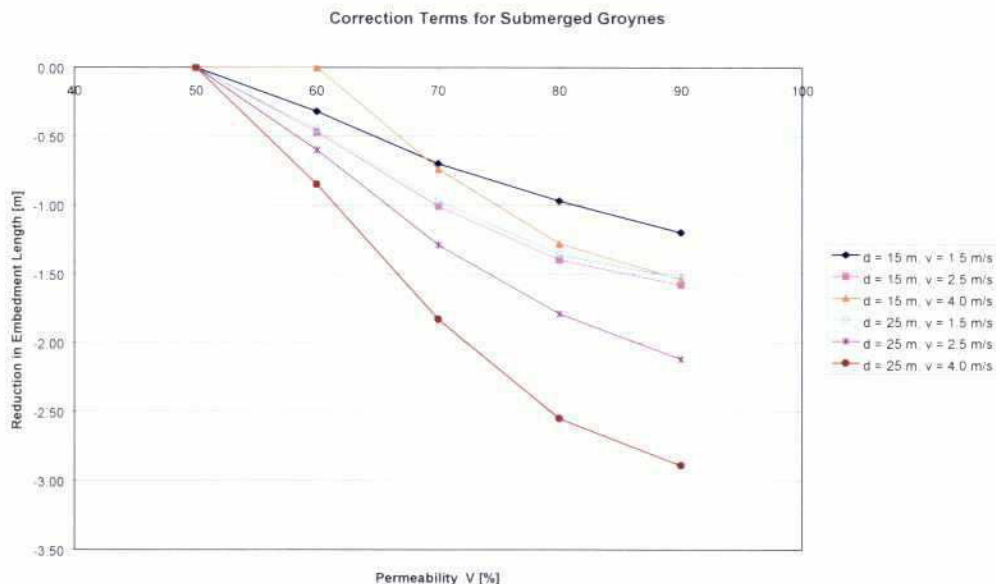


Fig. 6.3-9: Submerged groynes – reduction of pile embedment length

For instance, if the water depth is 25 m and the velocity 4.0 m/s the embedment length according to Fig. 6.2-1 is $l_E \approx 11.0$ m. This length is valid for a permeability of $V = 50$ %.

If $V = 70$ %, then the embedment length reduces by $\Delta l_E = 1.8$ m, so that the permissible embedment length becomes $l_E^* = 11.0 - 1.8 = 9.2$ m for submerged groynes.

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Attachment 1
Analysis of Wind Generated Waves
for the Design of Bank Protection
Structures at the Jamuna River

ANALYSIS OF WIND GENERATED WAVES FOR THE DESIGN OF BANK PROTECTION STRUCTURES AT THE JAMUNA RIVER

1 PRELIMINARY REMARKS

Waves are generated on the Jamuna River by wind during the tropical Norwesters, by cyclones and by passing of ships. Since inland waterway transport does not yet play an important role for the reach of the Jamuna River in question, ship induced waves are infrequent and are therefore not given consideration in the present design assumptions. An approximate calculation of the wind induced waves can be done by theoretical methods. But, due to the very strong current component and the substantial fluctuations in water depth over the fetch length at the Jamuna river, a reliable theoretical prediction of wave parameters is rather complex. Nevertheless, a design wave height of $H_{25} = 1.0$ m is commonly used for the major rivers of Bangladesh and seems to provide a reasonable estimate. In the following only a brief analysis of the wave conditions is given, to verify this assumption and to investigate the band width of results.

2 WIND SITUATION AT THE JAMUNA

Important for the Jamuna situation are the squalls during pre-monsoon (Norwesters) and post monsoon situation, which generate considerable waves on the river. Squalls are local disturbances causing substantial wind speeds with thunderstorms, mainly occurring in the months of March to May, but also at other times of the year.

Cyclones, in the form of typhoons and hurricanes, which are associated with strong wind velocities are rare in the interior part of Bangladesh. According to studies done for the Jamuna Multipurpose Bridge Project severe cyclones with a gust speed of 33 m/s (64 knots) may be expected in this region once in 30 years only. Due to the rare recurrence, cyclonic storms are not being considered as a design criteria for erosion prevention structures of general nature.

3 DESIGN WIND VELOCITY

Wind data of the Bangladesh Meteorological Department, Dhaka were evaluated for the stations Faridpur, Sirajganj, Bogra, Rangpur and Mymensingh by FAP 1 [1]. These results are considered as representative for the FAP 21 - Pilot Project.

In addition reference is made to the Jamuna Bridge Report [2]. The distribution curves for the wind velocity as a function of the return period presented therein are applied for determination of the design wind velocity.

A period of 15 minutes has been chosen deliberately in [2] as an approximate time for the full growth of wind waves for the conditions of the Jamuna areas in question.

-
- [1] BRTS Master Plan Report (May 1993)
Technical Annexes, Annex 4, Appendix H
 - [2] JAMUNA BRIDGE PROJECT, Phase II Study
River Training Works (May 1990)

Using the distribution formula of [2]

$$V_{\text{mean (15 min)}} = 15 + 2.25 \ln(T) \quad [\text{m/s}] \quad (9-1)$$

where T = return period (years), the following data result:

Return Period	[year]	2	5	10	25	50	100
$V_{\text{mean (15 min)}}$	[m/s]	16.56	18.62	20.18	22.24	23.80	25.36

Table 1.1: Design Wind Velocity for Wave Generation $V_{\text{mean (15 min)}}$

The above values are comparable with statistical values determined in [1], which resulted in a wind speed of 35 knots (~ 18 m/s) for a return period of about 5 years.

4 WIND DIRECTION

The wind directions from South-West to North-West (wind sector between 225° and 315°) are of major importance for the wind induced wave impact at the Bahadurabad Revetment.

5 FETCH LENGTH AND ASSOCIATED WATER DEPTH

An important parameter regarding wave generation is the fetch length. During monsoon, when the river flow is at its peak and many chars are inundated, the fetch length is at its maximum. The average water level elevations measured at Phulchari and Bahadurabad have been used as basis for the calculations (Table 1.2).

Location	April	May	June	July	Aug.	Sep.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.
Phulchari	14.3	15.7	17.5	18.8	18.7	18.3	17.1	15.0	14.0	13.5	13.3	13.4
Bahadurabad	15.0	15.8	17.5	18.6	18.9	18.0	17.0	15.0	14.0	13.5	13.2	13.5

Table 1.2: Average High Water Levels [m PWD]

Considering the above compiled water level information and the typical cross-sections established for Bahadurabad in the study phase (Main Report FAP 21, ANNEX 7), an average water depth between 7 m and 10 m during high flood stage in the months of July, August and September, associated with the fetch area, was estimated. The fetch length was approximated from satellite images and is varying between 2 and 5 km.

6 WAVE PARAMETERS

Because of the continuous changes between sand bars, chars, tributaries, etc. it is assumed that for the prediction of wind generated waves at the Jamuna River situation the "shallow water" conditions apply, even this is not following the general interpretation when analyzing the average values of relative water depth.

The significant design wave height H_s [m] and wave period T [s] can be computed by the following formulae (Shore Protection Manual, 1984):

$$H_s = 0.283 \tanh \left[0.530 \left(\frac{g d}{U^2} \right)^{0.75} \right] \tanh \left\{ \frac{0.0125 \left(\frac{g F}{U^2} \right)^{0.42}}{\tanh \left[0.530 \left(\frac{g d}{U^2} \right)^{0.75} \right]} \right\} \frac{U^2}{g} \quad (9-2)$$

$$T = 1.20 \tanh \left[0.833 \left(\frac{g d}{U^2} \right)^{0.375} \right] \tanh \left\{ \frac{0.077 \left(\frac{g F}{U^2} \right)^{0.25}}{\tanh \left[0.833 \left(\frac{g d}{U^2} \right)^{0.375} \right]} \right\} \frac{2 \pi U}{g} \quad (9-3)$$

where:

$$\begin{aligned} U \quad [\text{m/s}] &= V_{\text{mean } (15)} = 15 + 2.25 \ln (T) \\ F \quad [\text{m}] &= \text{fetch length} \\ d \quad [\text{m}] &= \text{average water depth within fetch} \\ g \quad [\text{m/s}^2] &= \text{gravitational acceleration} \end{aligned}$$

The computed wave parameters for different recurrence intervals are presented in the Table 1.3.

Recurrence Interval T	[year]	2	5	10	25	50	100
$V_{\text{mean}(15)} = U$	[m/s]	16.56	18.62	20.18	22.24	23.80	25.36
Recurrence Interval T	[year]	2			5		
Fetch Length F	[m]	2,000	3,500	5,000	2,000	3,500	5,000
Water Depth d	[m]	7.00			7.00		
Wave Height H_s	[m]	0.56	0.69	0.78	0.64	0.78	0.88
Wave Period T	[s]	2.64	2.97	3.19	2.80	3.14	3.37
Recurrence Interval T	[year]	10			25		
Fetch Length F	[m]	2,000	3,500	5,000	2,000	3,500	5,000
Water Depth d	[m]	9.00			9.00		
Wave Height H_s	[m]	0.71	0.87	0.98	0.79	0.97	1.10
Wave Period T	[s]	2.92	3.29	3.53	3.08	3.47	3.73
Recurrence Interval T	[year]	50			100		
Fetch Length F	[m]	2,000	3,500	5,000	2,000	3,500	5,000
Water Depth d	[m]	10.00			10.00		
Wave Height H_s	[m]	0.86	1.06	1.20	0.93	1.15	1.30
Wave Period T	[s]	3.20	3.60	3.88	3.31	3.73	4.02

Table 1.3: Computed Wave Data for Various Recurrence Intervals

From these results a value of $H_s = 1.0\text{m}$ is characterizing a lower estimate of the wave heights to be expected. On the other hand, the recurrence interval concerning the combination of extreme water levels and excessive wind speeds is much larger, hence, when designing for a 25 years return period, this will lead to a rather conservative layout. To prevent from over-dimensioning of the structure components and to allow for an investigation regarding the influence of predominant high flow velocities on the structure stability, a wave height of $H_s = 1.0\text{m}$ seems to be practicable and was used as design basis for this project. For the estimation of wave induced loads at the revetment components, the design wave height H was defined at $1.0 H_s$.

Attachment 2
Design of Revetments
Comparison of Design Formulas

STABILITY OF SOIL DAM REVETMENTS UNDER CURRENT ATTACK Comparison of Design Formulas

Depth Averaged Velocity \bar{u} (m/s) 2,30

Slope Ratio 1 : 3

Pilarczyk - Formula (1990)			
$D_n = 0.85 D_{50} = \frac{0.035 u_b^2}{\Delta m \cdot 2 g} \cdot \frac{\phi K_r K_h}{K_s \psi_{cr}}$			
Factors	Unit	Input	
Density of protection material ρ_s	(kg/m³)	2,600	
Density of water ρ_w	(kg/m³)	1,000	
Bottom velocity $u_b = 0.6 \cdot \bar{u}$	(m/s)	1,38	
Acceleration due to gravity g	(m/s²)	9,81	
Stability factor ϕ	(-)	0,75	
Turbulence factor K_r	(-)	1,50	
Angle of slope α	(degree)	18,43	
Angle of response (on geotextile: 20 on granul. filter: 25) $\epsilon_s = 20^\circ - 25^\circ$	(degree)	25,00	
Water depth h	(m)	10,00	
Depth factor $K_h = 1.0$ if $u=0.6\bar{u}$	(-)	1,000	
Slope factor $K_s = \sqrt{1 - (\sin^2 \alpha / \sin^2 \epsilon_s)}$	(-)	0,663	
Critical shear stress parameter ψ_{cr}	(-)	0,035	
Relative density $\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	1,600	
Nominal thickness of a single unit of the protection layer D_n	(cm)	10	
D_{50}	(cm)	13	

Rip-rap			
PIANC - Formula (1987a)			
$D_{n50} \geq \frac{0.7 u_s^2}{\Delta mgk}$			
Factors	Unit	Input	
Density of protection material ρ_s	(kg/m³)	2,600	
Density of water ρ_w	(kg/m³)	1,000	
Bottom velocity $u_b = 0.6 \cdot \bar{u}$	(m/s)	1,38	
Acceleration due to gravity g	(m/s²)	9,81	
Angle of slope α	(degree)	18,43	
Angle of response (on geotextile: 20 on granul. filter: 25) $\epsilon_s = 20^\circ - 25^\circ$	(degree)	25	
Slope factor $k = \sqrt{1 - (\sin^2 \alpha / \sin^2 \epsilon_s)}$	(-)	0,663	
Relative density $\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	1,600	
Nominal thickness of a single unit of the protection layer D_{n50}	(cm)	13	

STABILITY OF SOIL DAM REVETMENTS UNDER CURRENT ATTACK Comparison of Design Formulas

Rip-rap				Concrete Blocks			
Modified Isbash - Formula				FAP 1 - Formula			
$D_{n50} = \frac{Cu_b^2}{\Delta mgk}$				$D = 0.026 \cdot u^2 \cdot \frac{\pi}{6} \cdot \left(\frac{\rho_s}{\rho_c} - 1 \right)$			
Factors		Unit	Input	Factors		Unit	Input
Density of protection material	ρ_s	(kg/m³)	2.600	Specific gravity of stone	ρ_s	(kg/m³)	2.600
Density of water	ρ_w	(kg/m³)	1.000	Specific gravity of concrete (1:3:6)	ρ_c	(kg/m³)	1.980
Bottom velocity	$u_b = 0.6 \cdot \bar{u}$	(m/s)	1.38	Flow velocity	u	(m/s)	2.30
Acceleration due to gravity	g	(m/s²)	9.81				
Turbulence factor	C	(-)	0.70				
Slope factor	$k = \sqrt{1 - (\sin^2 \alpha / \sin^2 \epsilon_c)}$	(-)	0.663				
Relative density	$\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	1.600				
Nominal thickness of a single unit of the protection layer	D_{n50}	(cm)	13	Diameter of cube	D	(cm)	12

STABILITY OF SOIL DAM REVETMENTS UNDER CURRENT ATTACK
Comparison of Design Formulas

Concrete Blocks			
BRTS - Formula (5/1993) (FAP 1)			
$D = \frac{0.7v^2}{2(S_r - 1)g} \left[\log \left(\frac{6h}{D} \right) \right]^2 \cdot \frac{1}{\left(1 - \left(\frac{\sin \phi}{\sin \theta} \right)^2 \right)^{\frac{1}{2}}}$			
Specific gravity of cube material	S_s	(t/m ³)	1,98
Amplification factor A(f)	(straight = 1,1; upstr. term. = 1,3; groyne head = 1,4	(-)	1,00
Max. velocity close to bank	$v = \bar{u} \cdot 1,25 \cdot A_f$	(m/s)	2,88
Acceleration due to gravity	g	(m/s ²)	9,81
Angle of slope	ϕ	(degree)	18,43
Angle of repose	θ	(degree)	40
Water depth	h	(m)	10,00
Depth factor	h/D=5	(-)	5
Dimension of concrete cube (uniform size)	D	(cm)	20

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STABILITY OF EMBANKMENT REVETMENTS UNDER WAVE ATTACK Comparison of Design Formulas

Significant Wave Height (m)	H_s	0,65	0,65
Mean Wave Period (s)	T_m	2,50	2,50
Slope Ratio 1 :	1 :	3	5

0,65	0,65
2,50	2,50
3	5

Pilarczyk - Formula (1990)				
$D_n = 0.85 D_{50} \geq \frac{H_s \xi^b}{\Delta m \psi_u \phi \cos \alpha}$				
Factors	Unit	Input	Input	Input
Density of protection material	ρ_s (kg/m ³)	2.600	2.600	2.600
Density of water	ρ_w (kg/m ³)	1.000	1.000	1.000
Stability factor	ϕ (-)	2,25	3,00	(--)
Angle of slope	α (degree)	18,43	11,31	11,31
Breaker similarity index	$\xi_s = \tan \alpha \left(\frac{H_s}{L_0} \right)^{-0.5}$ (-)	1,29	0,77	(--)
Wave length	$L_0 = \frac{g \cdot T_m^2}{2 \pi}$ (m)	9,76	9,76	(--)
Interaction exponent	b (-)	0,50	0,50	(--)
Stability upgrading factor	ψ_u (-)	1,00	1,00	2,20
Relative density	$\Delta m = (\rho_s - \rho_w) / \rho_w$ (-)	1,60	1,60	1,60
Nominal thickness of a single unit of the protection layer	D_n (cm)	22	12	16
	D_{50} (cm)	26	15	19
Thickness of Protection Layer (Dn50 * 2.0)		52	44	38

HUDSON - Formula				
$H'_{s0} = \frac{\rho_s \cdot H_{s0}^3}{K_D \cdot \left(\frac{\rho_s}{\rho_w} - 1 \right)^3} \cdot \cot \alpha$				
Density of protection material	(kg/m ³)	2.600	2.600	2.600
Density of water	(kg/m ³)	1.000	1.000	1.000
Angle of slope	(degree)	18,43	11,31	11,31
Shape and stability coefficient	K_D	2,20	2,20	2,20
Relative density	(-)	1,60	1,60	1,60
Weight of average block size (kg)	W_{50}	26	26	16
Thickness of Protection Layer	D_{50}	22	22	19
		44	38	

Attachment 3

Design of Falling Aprons
Comparison of Design Formulas



STABILITY OF FALLING APRONS UNDER CURRENT ATTACK Comparison of Design Formulas

Depth Averaged Velocity	\bar{u}	2,30 (m/s)
Slope Ratio	1 :	10
Water depth		6,00 (m)

Preliminary Delft - Formula (Concrete Blocks)			
$D_{50} = \sqrt{\frac{19 * R * D_{50_{sed}}}{C_0}}$			
Factors	Unit	Input	
Density of protection material	ρ_s	(kg/m ³)	1,980
Density of water	ρ_w	(kg/m ³)	1,000
Water depth	R	(m)	6,00
Chery Coefficient	C (60 to 80)	(-)	60
Subsoil Grain	$D_{50_{sed}} (\geq 2 mm)$	(m)	0,0020
Thickness of top layer of falling apron	d	(m)	0,75
Critical shear stress parameter	ψ_{cr}	(-)	0,037
Design velocity	\bar{u}	(m/s)	2,30
Acceleration due to gravity	g	(m/s ²)	9,81
Angle of slope	α	(degree)	5,71
$C_0 = 1 + 1,8 \cdot \left(\frac{\psi_{ts} \cdot \Delta_m \cdot R \cdot C^2}{H^2} \right)^{0,33} \cdot \left(1 - \frac{d}{R} \right)^{0,45}$			
Relative density	$\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	0,980
Nominal thickness of a single unit of the protection layer	D_{50}	(cm)	15
Uniform concrete blocks : $D_{ts_{fa}} = D_{50_{fa}}$			

PIANC - Formula (1992)			
$D_{ti} = 0,85 \cdot D_{50} \geq \frac{0,7 u_b^2}{g \cdot \Delta m \cdot \cos \alpha}$			
Factors	Unit	Input	
Density of protection material	ρ_s	(kg/m ³)	1,980
Density of water	ρ_w	(kg/m ³)	1,000
Bottom velocity	$u_b = 0,6 \cdot \bar{u}$	(m/s)	1,38
Acceleration due to gravity	g	(m/s ²)	9,81
Angle of slope	α	(degree)	5,71
Relative density	$\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	0,980
Nominal thickness of a single unit of the protection layer	D_{n50}	(cm)	16

STABILITY OF FALLING APRONS UNDER CURRENT ATTACK Comparison of Design Formulas

Jamuna Formula			
$D = 0.85 \cdot D_{50} \geq \frac{0.7 \cdot u_{cr}^2}{\Delta m \cdot 2 \cdot g}$			
Factors		Unit	Input
Density of protection material	ρ_s	(kg/m³)	1,980
Density of water	ρ_w	(kg/m³)	1,000
Critical velocity	$u_{cr} = u_b = 0.6 \cdot \bar{u}$	(m/s)	1,38
Acceleration due to gravity	g	(m/s²)	9,81
Relative density	$\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	0,980
Nominal thickness of a single unit of the protection layer	D_{n50}	(cm)	8

Modified Delft - Formula (Rip-rap)			
$D_{50_{fu}} = \sqrt{\frac{19 \cdot R \cdot D_{50_{sol}}}{c_0 \cdot 0.8}}$			
Factors		Unit	Input
Density of protection material	ρ_s	(kg/m³)	2,600
Density of water	ρ_w	(kg/m³)	1,000
Water depth	R	(m)	6,00
Chery Coefficient	C (60 to 80)	(-)	60
Subsoil Grain	$D_{50_{sol}} (\geq 2 \text{ mm})$	(m)	0,0020
Thickness of top layer of falling apron	d	(m)	0,75
Critical shear stress parameter	ψ_{ter}		0,037
Design velocity	\bar{u}	(m/s)	2,30
Acceleration due to gravity	g	(m/s²)	9,81
Angle of slope	α	(degree)	5,71
$c_0 = 1 + 1.8 \cdot \left(\frac{\psi_{ter} \cdot \Delta m \cdot R \cdot C^2}{\bar{u}^2} \right)^{0.33} \cdot \left(1 - \frac{d}{R} \right)^{0.45}$			11,37
Relative density	$\Delta m = (\rho_s - \rho_w) / \rho_w$	(-)	1,600
Nominal thickness of a single unit of the protection layer	$D_{50} (D_{15_{fa}} \approx 0.8 \cdot D_{50_{fa}})$	(cm)	16

Attachment 4
Selected Construction Drawings
as a Base for Future Standard Designs

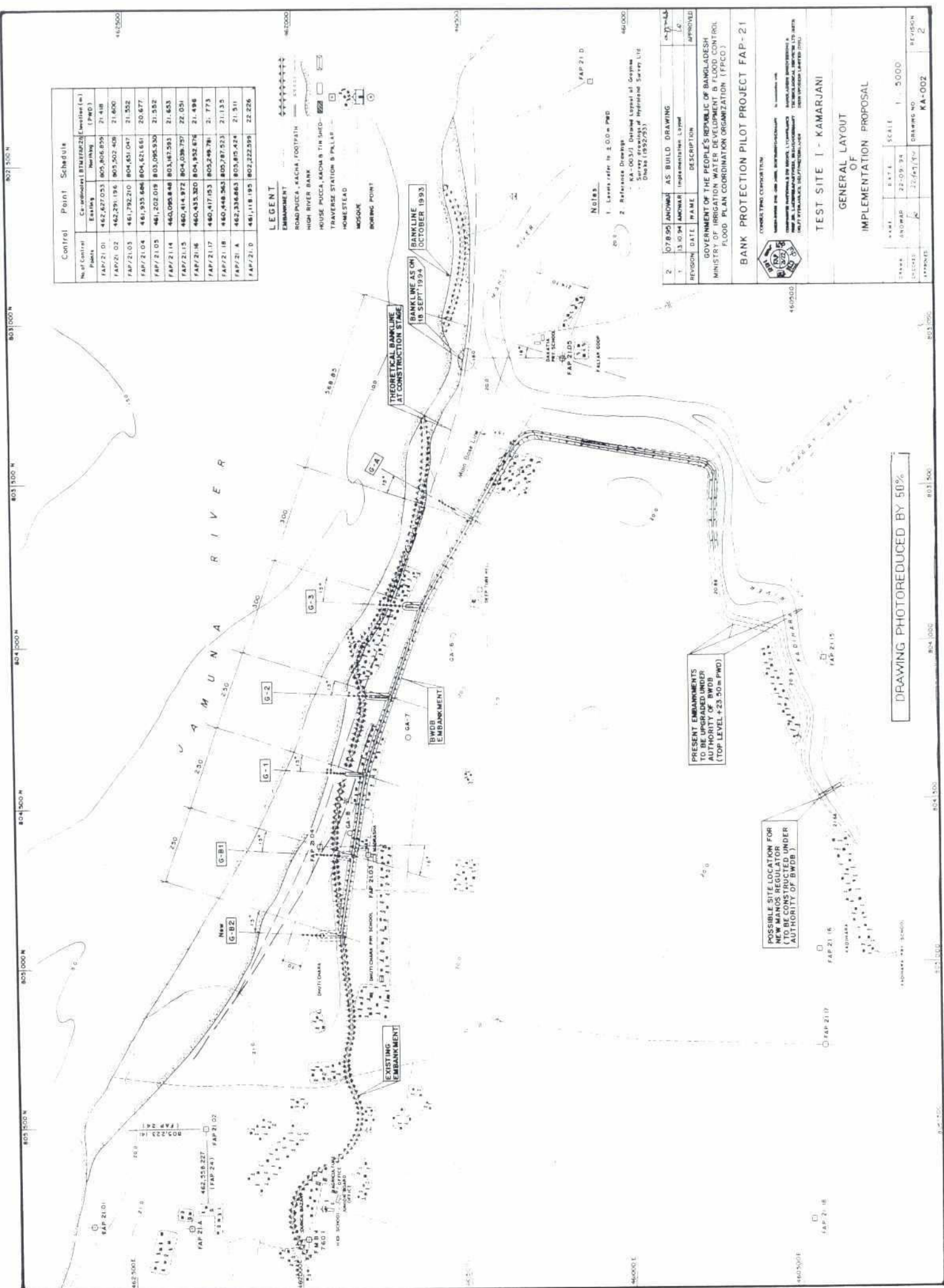
List of Drawings

Original Design

KA-002 Rev. 2	General Layout of Implementation Proposal
KA-014 Rev. 2	BWDB-Embankment – Revetment Details
KA-106 Rev. 1	Groyne G-1: Sheet-Pile Cofferdam (ARBED PU-6) Pile Installation Plan, Pile Schedule
KA-107 Rev. 1	Groyne G-1: Sheet-Pile Cofferdam (ARBED PU-6) Detailed Layout, Sections, Details
KA-108/I Rev. 1	Groyne G-1: Sheet-Pile Cofferdam (ARBED PU-6) Details of Anchoring – Sheet I
KA-108/II Rev. 1	Groyne G-1: Sheet-Pile Cofferdam (ARBED PU-6) Details of Anchoring – Sheet II
KA-113 Rev. 1	Navigation Aids “East Cardinal Mark”
KA-403 Rev. 1	Groyne G-A: Prestressed Spun Concrete Pile dia. 500 x 100 mm
KA-405 Rev. 1	Groyne G-A: Reinforced Concrete Sheet-Piling
KA-406 Rev. 2	Groyne G-A: Concrete Sheet-Pile Cofferdam Pile Installation Plan, Details
KA-407/I Rev. 1	Groyne G-A: Concrete Sheet-Pile Cofferdam; Details of Anchoring – Sheet I
KA-407/II Rev. 1	Groyne G-A: Concrete Sheet-Pile Cofferdam; Details of Anchoring – Sheet II

Adaptation Works

AD-KA-020 Rev.2:	Groyne G-2: General Layout Plan
AD-KA-021 Rev.4:	Pile Layout Plan
AD-KA-023/I Rev.1:	Groyne G-2: Detailed Layout of Modified Groyne
AD-KA-024 Rev. 0:	Groyne G-2: Cross-Sections of Groyne Head
AD-KA-050 Rev. 3:	Groyne G-A/2: General Layout Plan
AD-KA-051 Rev. 2:	Groyne G-A/2:Pile Layout Plan, Pile Installation Schedule, Danger Point Mark



Control Point Schedule		
No. of Control Points	Co-ordinates (UTM/Zone 22N)	Location (m)
1	462,427,033	805,806,859
2	462,427,033	805,806,859
3	462,427,033	805,806,859
4	462,427,033	805,806,859
5	462,427,033	805,806,859
6	462,427,033	805,806,859
7	462,427,033	805,806,859
8	462,427,033	805,806,859
9	462,427,033	805,806,859
10	462,427,033	805,806,859
11	462,427,033	805,806,859
12	462,427,033	805,806,859
13	462,427,033	805,806,859
14	462,427,033	805,806,859
15	462,427,033	805,806,859
16	462,427,033	805,806,859
17	462,427,033	805,806,859
18	462,427,033	805,806,859
19	462,427,033	805,806,859
20	462,427,033	805,806,859
21	462,427,033	805,806,859
22	462,427,033	805,806,859
23	462,427,033	805,806,859
24	462,427,033	805,806,859
25	462,427,033	805,806,859
26	462,427,033	805,806,859
27	462,427,033	805,806,859
28	462,427,033	805,806,859
29	462,427,033	805,806,859
30	462,427,033	805,806,859
31	462,427,033	805,806,859
32	462,427,033	805,806,859
33	462,427,033	805,806,859
34	462,427,033	805,806,859
35	462,427,033	805,806,859
36	462,427,033	805,806,859
37	462,427,033	805,806,859
38	462,427,033	805,806,859
39	462,427,033	805,806,859
40	462,427,033	805,806,859
41	462,427,033	805,806,859
42	462,427,033	805,806,859
43	462,427,033	805,806,859
44	462,427,033	805,806,859
45	462,427,033	805,806,859
46	462,427,033	805,806,859
47	462,427,033	805,806,859
48	462,427,033	805,806,859
49	462,427,033	805,806,859
50	462,427,033	805,806,859

- LEGEND**
- EMBANKMENT
 - ROAD PUCCA, KACHA, FOOTPATH
 - HIGH RIVER BANK
 - HOUSE PUCCA, KACHA & TIN SHED
 - TRAVEL STATION & PILLAR
 - HOMESTEAD
 - MOSQUE
 - BORING POINT

NOTES

1. Levels refer to 100m PWD
2. Reference Drawing: KA-003/J Detailed Layout of Gopine Survey of the Irrigation Survey Ltd Dhaka (1992/93)

REVISION	DATE	BY	DESCRIPTION
1	13.10.94	ANWAR	Implementation Layout
2	07.05.95	ANWAR	AS BUILT DRAWING

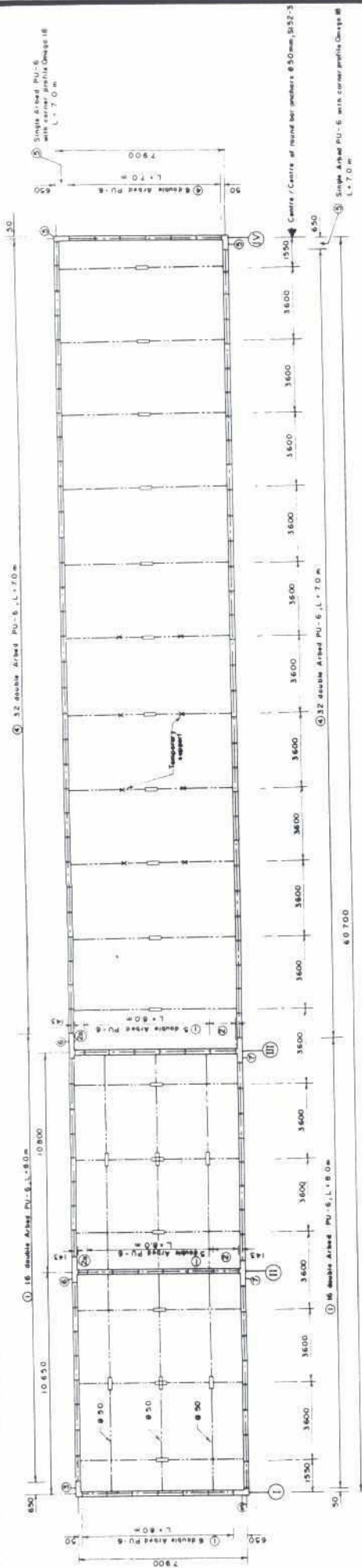
GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (FPCCO)

BANK PROTECTION PILOT PROJECT FAP-21

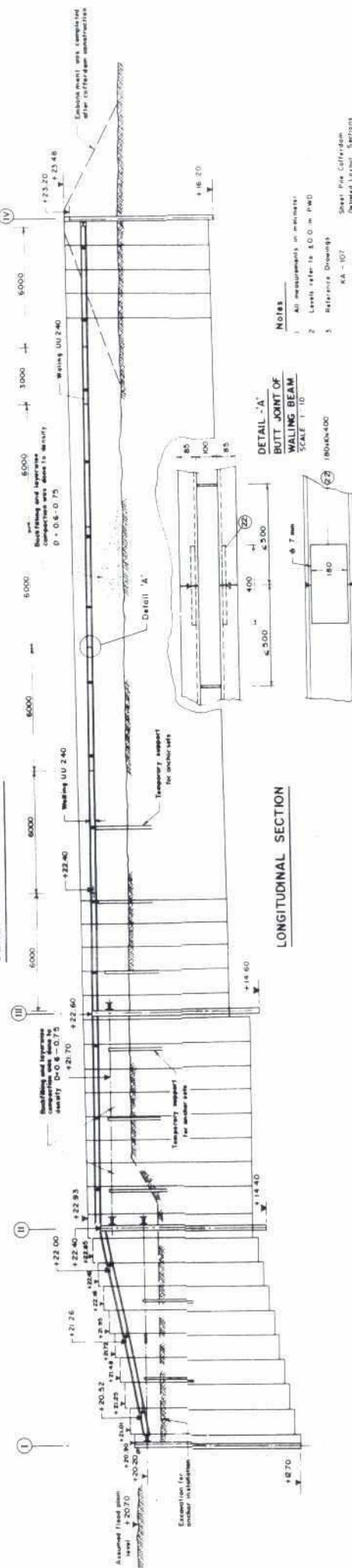


TEST SITE 1 - KAMARJANI			
GENERAL LAYOUT OF IMPLEMENTATION PROPOSAL			
SCALE	DATE	SCALE	REVISION
1:5000	22.09.94	1:5000	2
ANWAR	22.09.94	ANWAR	2
DATE	22.09.94	DATE	2
BY	22.09.94	BY	2
APPROVED	22.09.94	APPROVED	2
DRAWING NO.	KA-002	DRAWING NO.	KA-002
REVISION	2	REVISION	2

DRAWING PHOTOREDUCTION BY 50%



GENERAL LAYOUT PLAN



LONGITUDINAL SECTION

STEEL SHEET PILE ARBED PU-6 SISO 37/ SIE240 SP
SCHEDULE OF MATERIAL SUPPLIED BY EMPLOYER

ITEM	NO.	DESIGNATION	SINGLE WEIGHT (kg)	TOTAL WEIGHT (kg)
1	48	Double profile, L x B = 8.0 m	724.80	37689.60
2	2	Double profile, L x B = 8.0 m (to be used in cofferdam profile No. 107 mm)	724.80	1449.60
2a	2	Single profile, L x B = 8.0 m (to be used in cofferdam profile No. 507 mm)	362.40	724.80
3	2	Single profile with corner profile Omega 161, L x B = 8.0 m	506.40	1012.80
4	70	Double profile, L x B = 7.0 m	634.20	44394.00
5	2	Single profile with corner profile Omega 161, L x B = 7.0 m	443.10	886.20
6	2	Double profile with corner profile No. 22, L x B = 7.0 m	809.60	1619.20
7	2	Single profile with corner profile No. 22, L x B = 7.0 m	809.60	1619.20
	130		92.8300	12068.60

⑥ Double profile Arbed PU-6 with corner profile No. 22, L x B = 8.0 m



⑦ Double profile Arbed PU-6 with corner profile No. 22, L x B = 8.0 m



⑧ Single profile Arbed PU-6 with corner profile Omega 161, L x B = 8.0 m



⑨ Double profile Arbed PU-6, L x B = 7.0 m



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BANK PROTECTION PILOT PROJECT FAP-21



TEST SITE 1 - KAMARJANI

GROYNE G-1
SHEET PILE COFFERDAM (ARBED PU-6)
PILE INSTALLATION PLAN, PILE SCHEDULE

DATE	APPROVED	SCALE	REVISION
16-01-94	KA-106	1:100	1

REVISION



Item No	Notes		Section / Dimensions	Steel Weight Grade	Single Weight (kg)	Total Weight (kg)	Remarks
	Request	Score					
1	2	1	Round bar anchor Ø 30 – 5560	S15.2 – 3	85.62	1718.40	
2	3	4	Round bar anchor Ø 30 – 39.00	S15.2 – 3	60.04	2,202.28	
3	4	3	Turnbuckle Ø 80 – 430	S15.2 – 3	9.80	284.40	
4	2	3	Welding UI 240 – 6 000	S15.2 – 3	282.95	40	
5	4	4	Anchor support plate 160 x 35 x 160	S13.7 – 2	8.06	354.64	
6	3	5	Anchor plates 310 x 40 x 310	S13.7 – 2	3.08	241.44	
7	4	3	Anchor plates 160 x 35 x 160	S13.7 – 2	8.06	278.76	
8	6	4	Post DIN 555	S13.7 – 2	110	125.40	
9	12	14	Welding bar plate 160 x 25 – 360	S6.25 – 3	1.75	241.20	
10	12	15	Welding bar plate 310 x 25 x 310	S13.7 – 2	18.83	2,820.15	
11	12	15	Welding bar plate 120 x 75 x 160	S13.7 – 2	3.77	538.03	
21	4	—	CD 160 x 8 x 400	S8.37 – 2	2.51	160.04	
22	4	4	CD 180 x 10 x 400	S13.7 – 2	5.45	2,998.00	
23	4	2	CD 100 x 10 x 800	S13.7 – 2	62.80	378.80	
						Total	378.33

[illegible]

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MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD & AN COORDINATION ORGANIZATION (FPCO)

BANK PROTECTION PILOT PROJECT FAP-21



Source: U.S. Census Bureau, *Current Population Reports*, 1990.

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TEST SITE 1 - KAMARJANI

GROYNE G-1

SHEET PILE COFFERDAM (ARBED PU-6)
DETAILS OF ANCHORING - SHEET

NAME	DATE	SCALE	1 2
ANWAR	19-12-83		
CHECKED	18-01-84	DRAWING NO.	KA-108/1
		REVISION	1

Steel Pile Catalogue
 Pile Installation Plan, Pile Schedule
 Steel Pile Catalogue
 Detailed Layout, Sections
 Steel Pile Catalogue (Ained PU-6)

Notes

- Notes
- All measurements in millimeter
- Level ratio to 1.00 = PWD
- Reference Drawings

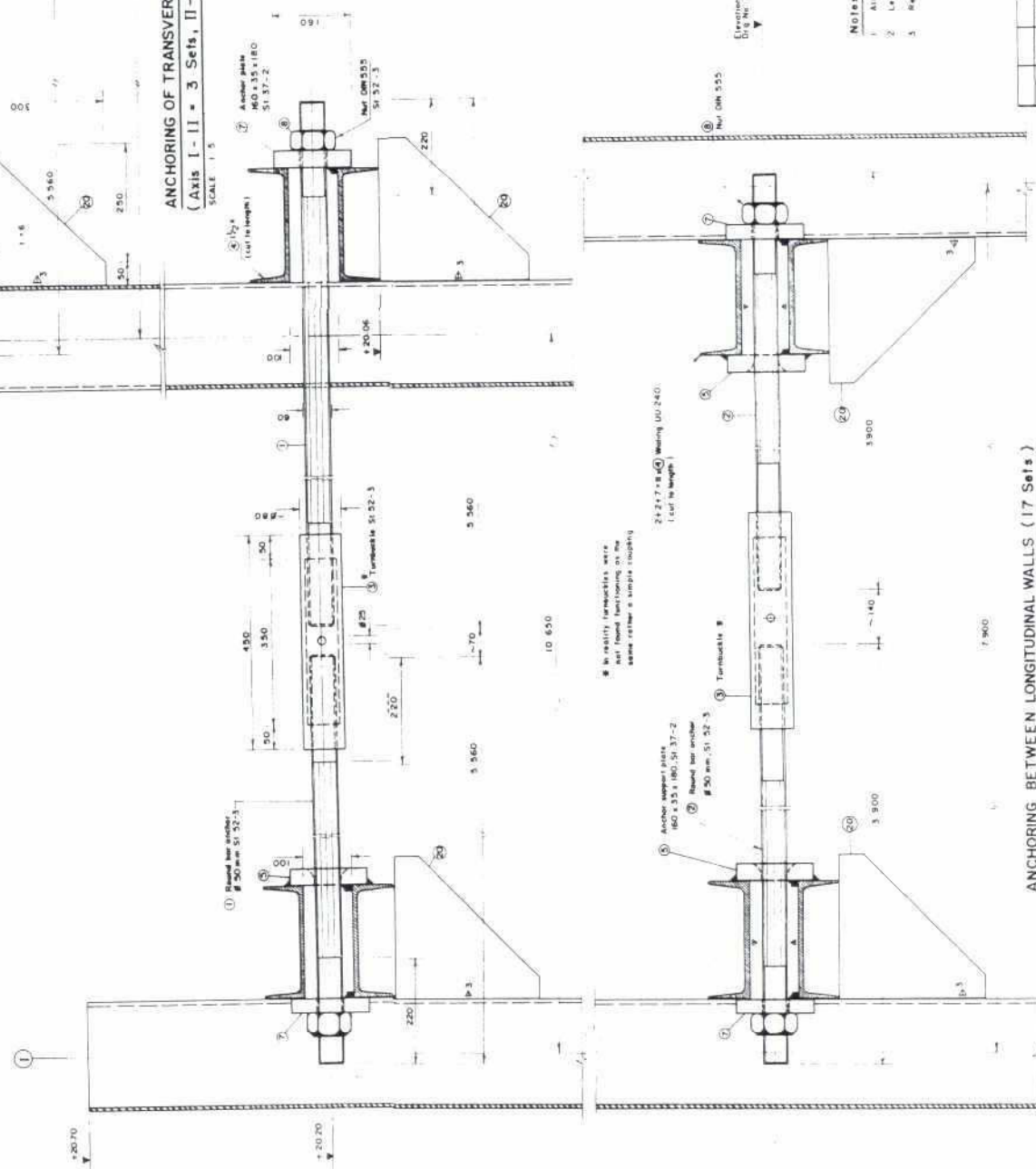
KA-105

404-107

100

ANCHORING BETWEEN LONGITUDINAL WALLS (17 Sets)

TABLE 1

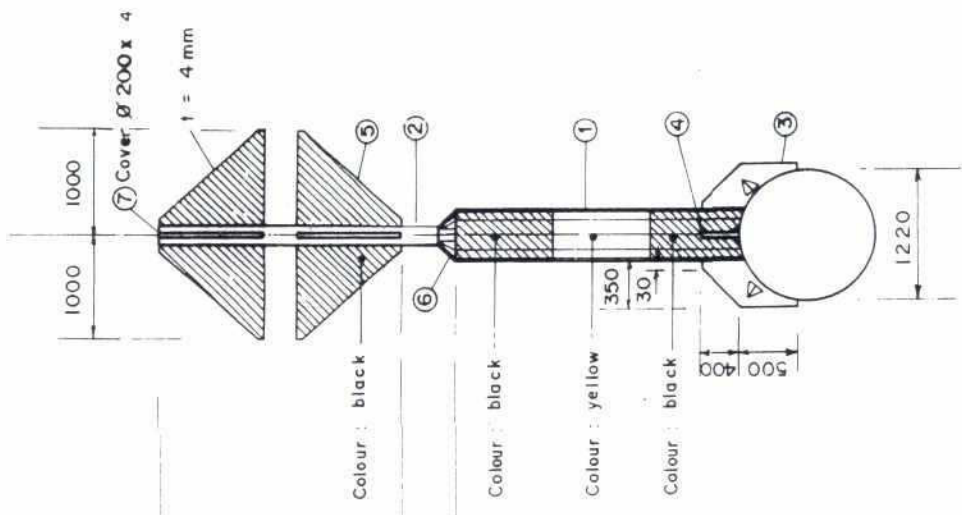


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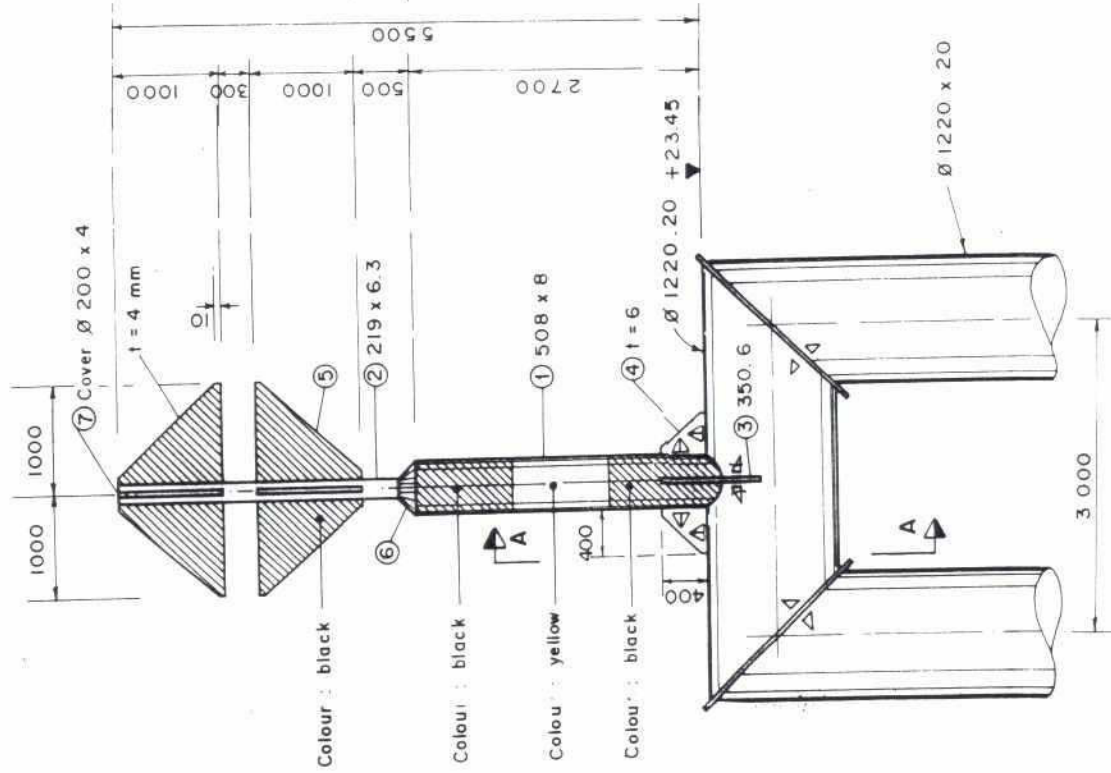
MATERIAL SCHEDULE				
Item No.	Nos.	Section / Size	Weight per No. kg	Weight kg
1	1	Ø 508 x 8 ... 2900	—	285.93
2	1	Ø 219 x 6.3 ... 3000	—	99.10
3	2	Ø 350 x 6 ... 900	14.84	29.68
4	2	Ø 400 x 6 ... 400	5.02	10.04
5	8	Ø 890 x 4 ... 1000	21.67	173.36
6	1	Ø 508/219 ... 200	—	15.00
7	1	Ø 220 x 4	—	1.18
Total (1 No.)			—	614.80

NOTES :

1. All measurements in millimeters
2. Levels refer to ± 0.0 m PWD
3. Steel grade St 37, DIN 17100 or equivalent
4. Welding seam thickness $a = 0.7$ min or as shown on the drawing
5. All exposed steel edges were chamfered
6. Preservation coating Type II, Subsection 1452 of Specification, colour : yellow and black as shown on the drawing
7. Reference Drawings
KA - 101 Groyne G-1, General Arrangement
KA - 201 Groyne G-2, General Arrangement
KA - 301 Groyne G-3, General Arrangement



SECTION A-A
Scale 1:50



ELEVATION
Scale 1:50

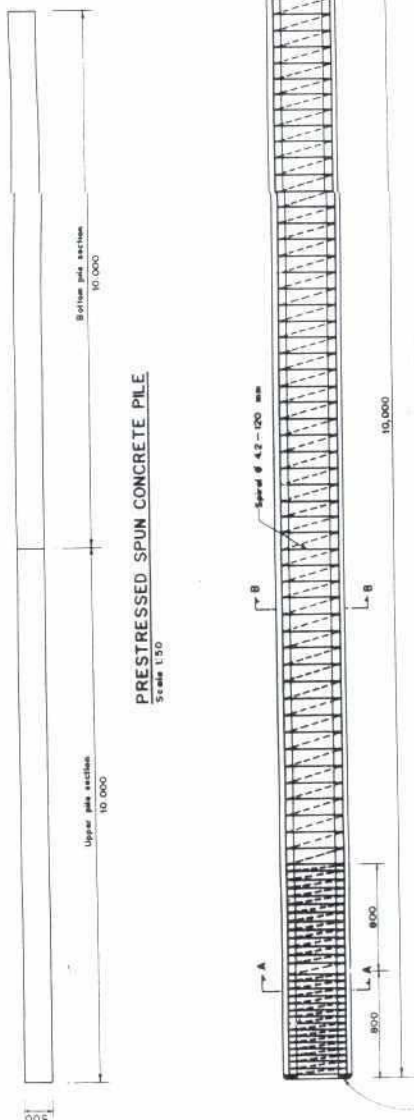


GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF IRRIGATION, WATER DEVELOPMENT & FLOOD CONTROL FLOOD PLAN COORDINATION ORGANIZATION (FPCO)	
BANK PROTECTION PILOT PROJECT FAP - 21	
	DRAWN: ANWAR CHECKED: W APPROVED: A
NAME: ANWAR DATE: 14-12-93 SCALE: 1:50	DRAWING NO: KA-113 REVISION: 1
TEST SITE I - KAMARJANI	
NAVIGATION AIDS	
"EAST - CARDINAL MARK"	

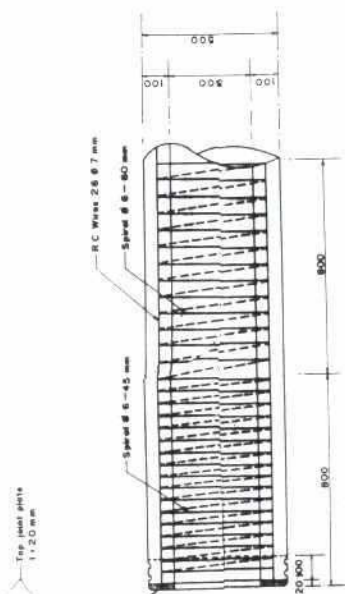
DRAWING PHOTOREDUCED BY 50 %

REV	DATE	NAME	DESCRIPTION
1	28.5.95	ANWAR	AS BUILT DRAWING
			APPROVED

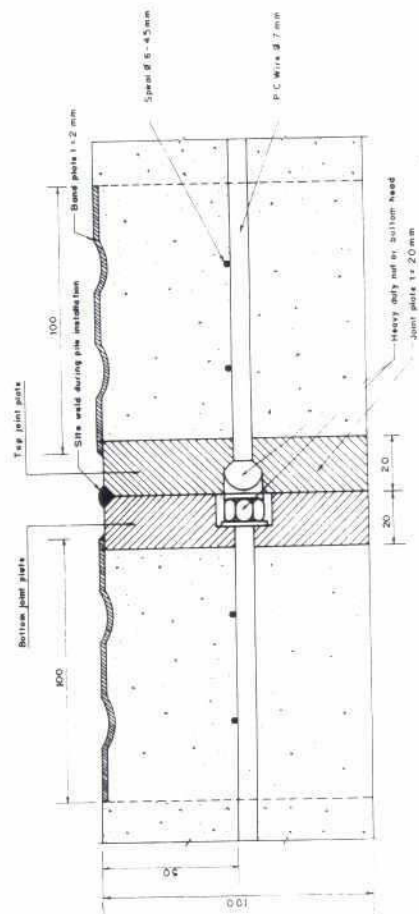
Scale 150



Scale 1: 20

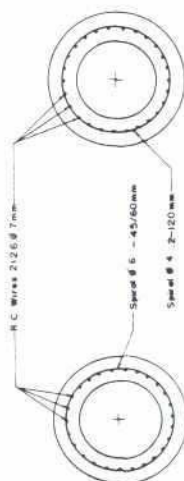


Score 110

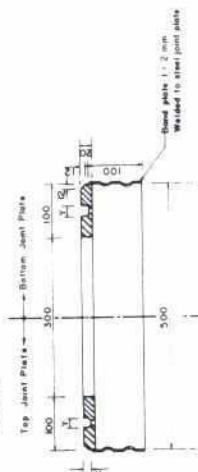


Scale 1/1

Scale 110

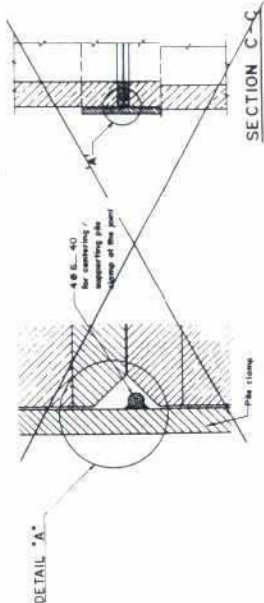
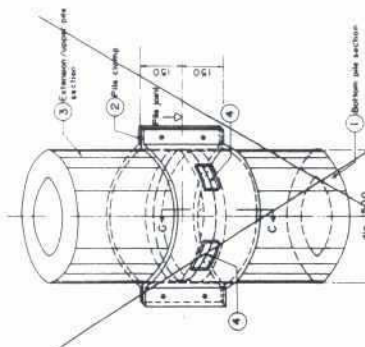


Scale 1:10



Scale 1 90

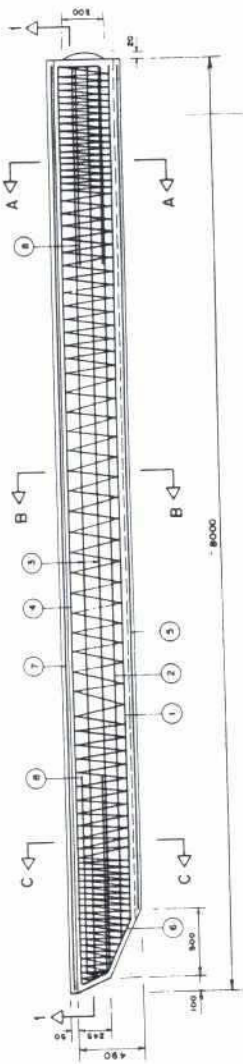
- 1 Drive first pile section to a lap level connector, four welding joints.
- 2 Fit pile clamp.
- 3 Place, align and secure pile extension.
- 4 Tighten the pile joint at the four corners of the pile clamp.
- 5 Remove pile clamp and weld the complete pile joint (weld 12 mm).



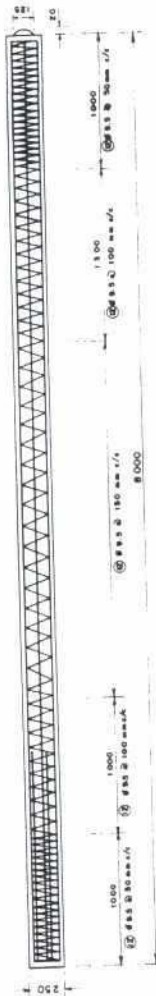
1. Prestressed shear concrete joists are produced by GEMCON Ltd., Perthshire.
2. Concrete quality grade B 45, DIN 1045.
3. Prestressing steel $> 1600 \text{ N/mm}^2$ tensile strength.
4. Joints were not bonded/split till the concrete has reached compression strength of about 3.5 N/mm^2 .
5. Reference Drawing No. KA-404. Pile Installation Plan.

1	25-4-80	REV	DATE	NAME	AS BUILT DRAWING	APPROVED
<p>GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF IRRIGATION, WATER DEVELOPMENT & FLOOD CONTROL FLOOD PLAN COORDINATION ORGANIZATION (FPCCO)</p> <p>BANK PROTECTION PILOT PROJECT FAP-21</p>						
<p>GENERAL NOTES CONCERNING</p> <p>1. ALL WORKS SHALL BE DONE IN ACCORDANCE WITH THE SPECIFICATIONS AND STANDARDS OF THE GOVERNMENT OF BANGLADESH.</p> <p>2. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROTECTION OF ALL EXISTING UTILITIES AND STRUCTURES.</p> <p>3. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROTECTION OF ALL EXISTING UTILITIES AND STRUCTURES.</p> <p>4. THE CONTRACTOR SHALL BE RESPONSIBLE FOR THE PROTECTION OF ALL EXISTING UTILITIES AND STRUCTURES.</p>						
<p>TEST SITE 1 - KAMARJANI</p> <p>GROYNE G - A</p> <p>PRESTRESSED SPUN CONCRETE PILE</p> <p>DIA. 500 X 100 mm</p>						
DRAWN	DATE	NAME	DATE	NAME	SCALE	REVISION
CHECKED	24-11-93	M.A. HALIM	10-02-94	10-02-94	1:50, 1:20, 1:10, 1:1	KA-403
APPROVED						

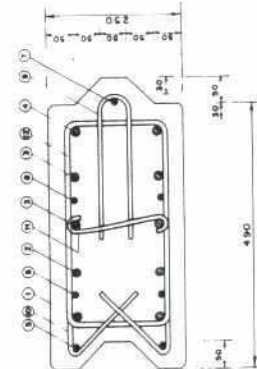
DRAWING PHOTOREDUCED BY 50 %



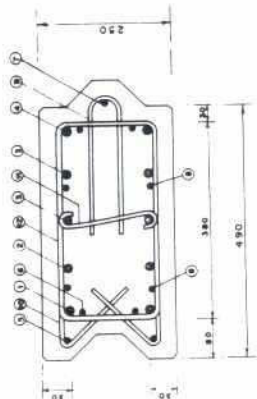
FRONT SIDE
SCALE 1:20



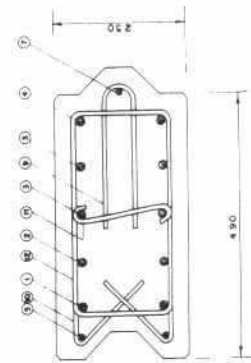
LONGITUDINAL SECTION 1-1
SCALE 1:20



SECTION A-A
SCALE 1:5



SECTION C-C
SCALE 1:5



SECTION B-B
SCALE 1:5



SECTION OF STARTER PILE
SCALE 1:5

BAR SCHEDULE FOR ONE SHEET PILE

ITEM No.	BAR DIA.	BAR SHAPE (reference in millimeter)	No.	BAR LENGTH (mm)	WEIGHT (kg)
1	Ø 15.9 (Ø 5/8")	1900	2	7.70	24.33
2	Ø 15.9 (Ø 5/8")	7450	2	7.85	24.17
3	Ø 15.9 (Ø 5/8")	7450	4	7.85	49.61
4	Ø 15.9 (Ø 5/8")	7400	2	8.10	25.80
5	Ø 9.5 (Ø 3/8")	6350	2	6.35	7.84
6	Ø 9.5 (Ø 3/8")	1500	2	3.00	3.70
7	Ø 9.5 (Ø 3/8")	7400	1	7.90	4.87
8	Ø 9.5 (Ø 3/8")	1500	4	3.20	7.90
9	Ø 9.5 (Ø 3/8")	1500	52	0.70	22.46
10	Ø 9.5 (Ø 3/8")	1500	98	0.50	29.62
11	Ø 9.5 (Ø 3/8")	1500	52	0.40	12.83
12	Ø 9.5 (Ø 3/8")	1500	105	0.478	8.78
Total weight for one Standard Pile					277.72
Total weight for one Starter Pile (1 and 6 added)					250.59

SHEET PILE SUPPLY SCHEDULE

Type	Supplied Nos	Total weight of Reinforcing steel (kg)	Remark
① Starter Pile	6	1,502.34	6
② Standard Pile	310	86,093.20	299
TOTAL		87,595.54	1

- Notes:
- All measurements in millimeter.
 - Reinforcing steel, welded mesh, Grade 60, ASTM A-615.
 - Concrete class 945, DIN 1045.
 - Lifting and handling of precast sheet piles were done after concrete compressive strength had reached 35 N/mm².
 - Reference Drawing: KA-406 Concrete Sheet Pile Cofferdam Pile Installation Form, Device

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FLOOD PLAN COORDINATION ORGANIZATION (FPCCO)

BANK PROTECTION PILOT PROJECT FAP-21



TEST SITE 1 - KAMARJANI

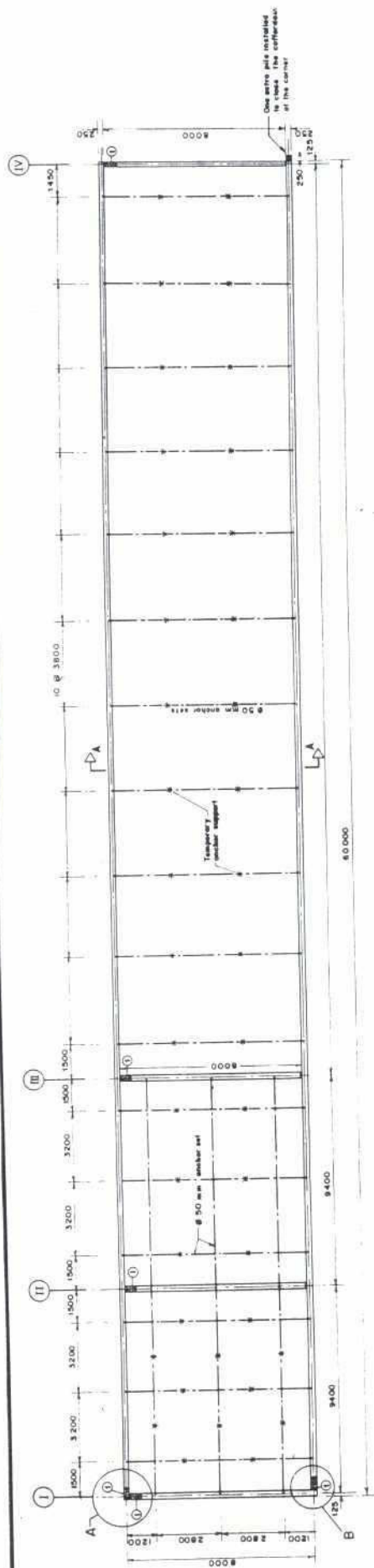
GROYNE G-A
REINFORCED CONCRETE SHEET PILING

DRAWING PHOTO REDUCED BY 50 %

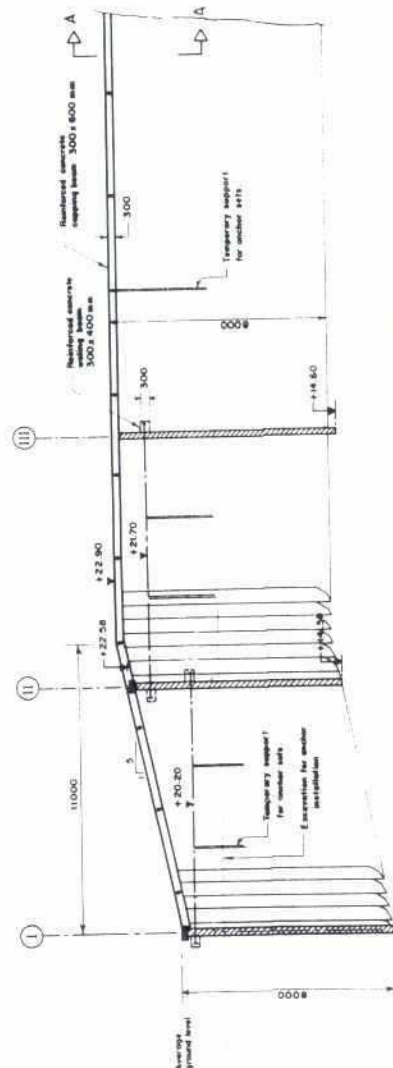
REV.	DATE	NAME	DESCRIPTION
1	NO-4-95	AKHAR	As built drawing

DATE	CHECKED	APPROVED	NAME	DATE	SCALE	REVISION
28-11-93	AKHAR	AKHAR	AKHAR	28-11-93	1:5, 1:20	1
26-01-94	AKHAR	AKHAR	AKHAR	26-01-94	1:5, 1:20	2
10-02-94	AKHAR	AKHAR	AKHAR	10-02-94	1:5, 1:20	3

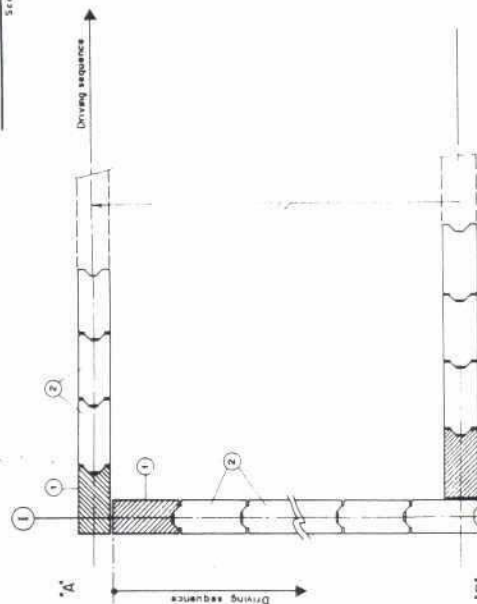
KA-405



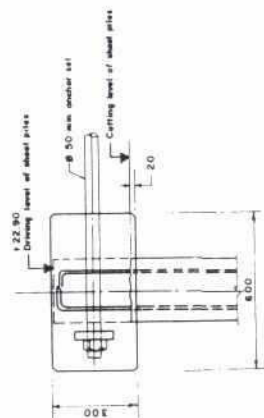
GENERAL LAYOUT PLAN



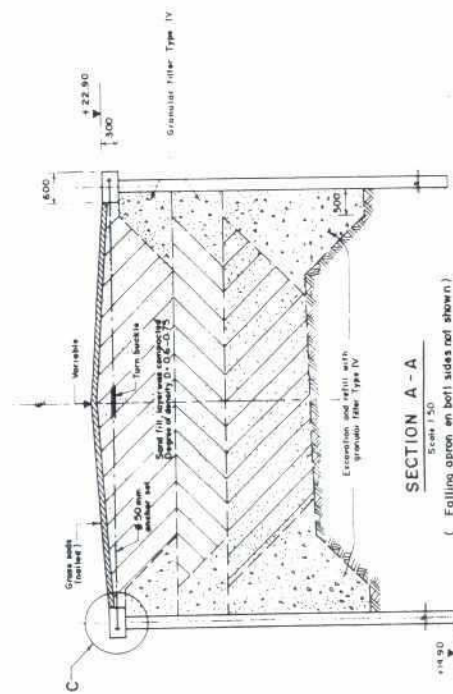
PART LONGITUDINAL SECTION



DETAIL - A & B



DETAIL - C



SECTION A - A

(Falling apron on bot! sides not shown)

Notes

- | | |
|---|---|
| 1 | All measurements in millimeters. |
| 2 | Steel plate No. 2 0 0 0 PWD. |
| 3 | <p>Sheet pile supporting beam and walking beams:</p> <ul style="list-style-type: none"> - Reinforcing steel grade 60, ASTM A-615 - Concrete, class B25, DIN 1045 - Cement Type 1 (Rohr brand, Bangladesh) - Concrete cover to reinforcing steel 30 mm |
| 4 | <p>① - Starter pile
② - Standard pile</p> |
| 5 | <p>Reference Drawings:</p> <ul style="list-style-type: none"> KA - 003 Layout of Guywires KA - 004 Construction of Sheet Pile KA - 007 Details of Anchoring KA - 008 Details of Bracing KA - 009 Details of Beam, Walking Beam KA - 014 Details of Sower Protection |

DRAWING PHOTOREDUCED BY 50 %

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (FPCO)

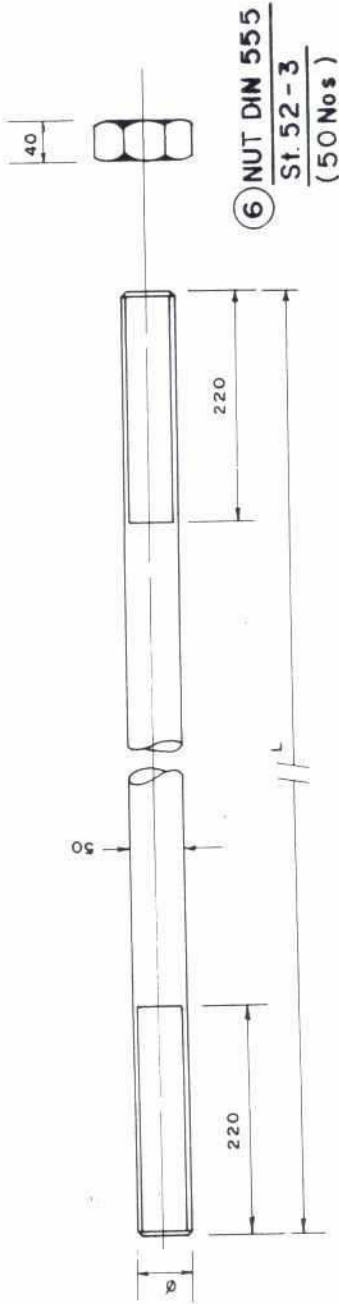
BANK PROTECTION PILOT PROJECT FAP-21



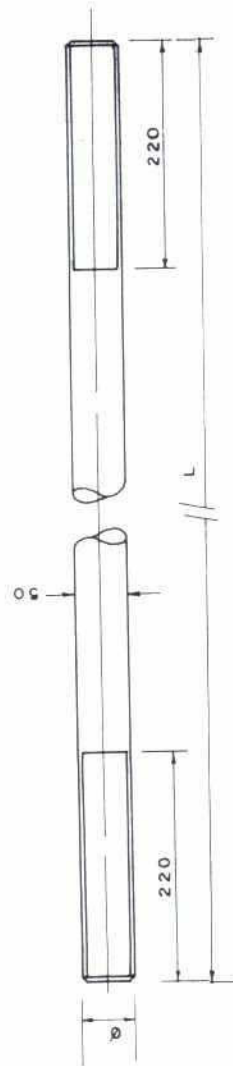
TEST SITE 1 - KAMARJANI

GROYNE G - A
CONCRETE SHEET PILE COFFERDAM
PILE INSTALLATION PLAN DETAILS

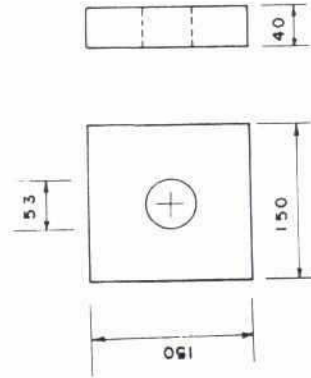
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① ROUND BAR ANCHOR DIA 50mm, St. 52-3
(14 Nos.) L = 5,100 mm
Scale 1:5



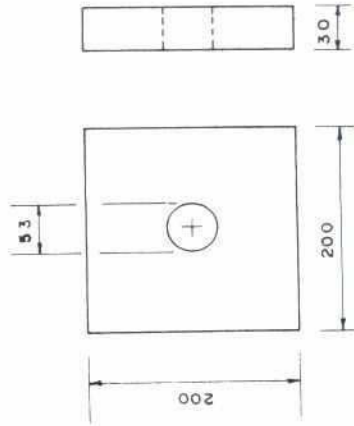
② ROUND BAR ANCHOR DIA 50mm, St. 52-3
(36 Nos.) L = 4,200 mm
Scale 1:5



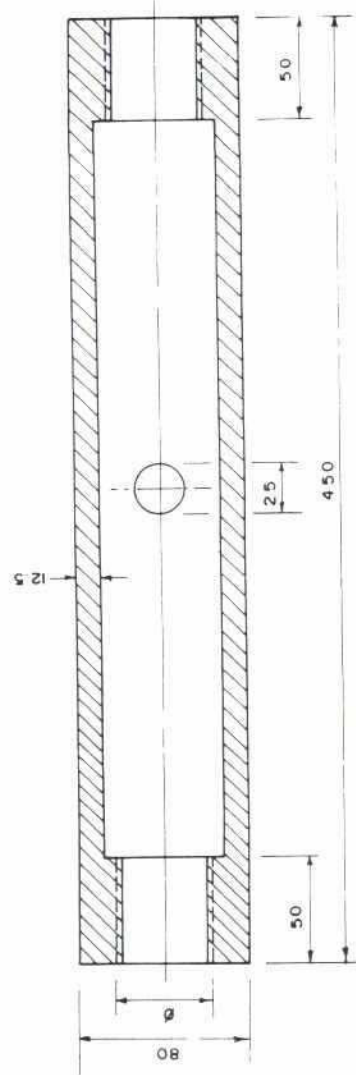
④ ANCHOR PLATE, St. 37-2
(36 Nos.)
Scale 1:5

NOTES

1. All dimensions are in millimeters
2. Steel grades as shown
3. All exposed edges chamfered
4. Anchor holes cut by mechanical tools
5. Material schedule depicted on Drawing KA-407/Sheet-1



⑤ ANCHOR PLATE, St. 37-2
(14 Nos)
Scale 1:5



③ TURNBUCKLE St. 52-3
(25 Nos)
Scale 1:2.5

DRAWING PHOTOREDUCTION BY 50 %

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MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (FPCO)

BANK PROTECTION PILOT PROJECT FAP-21



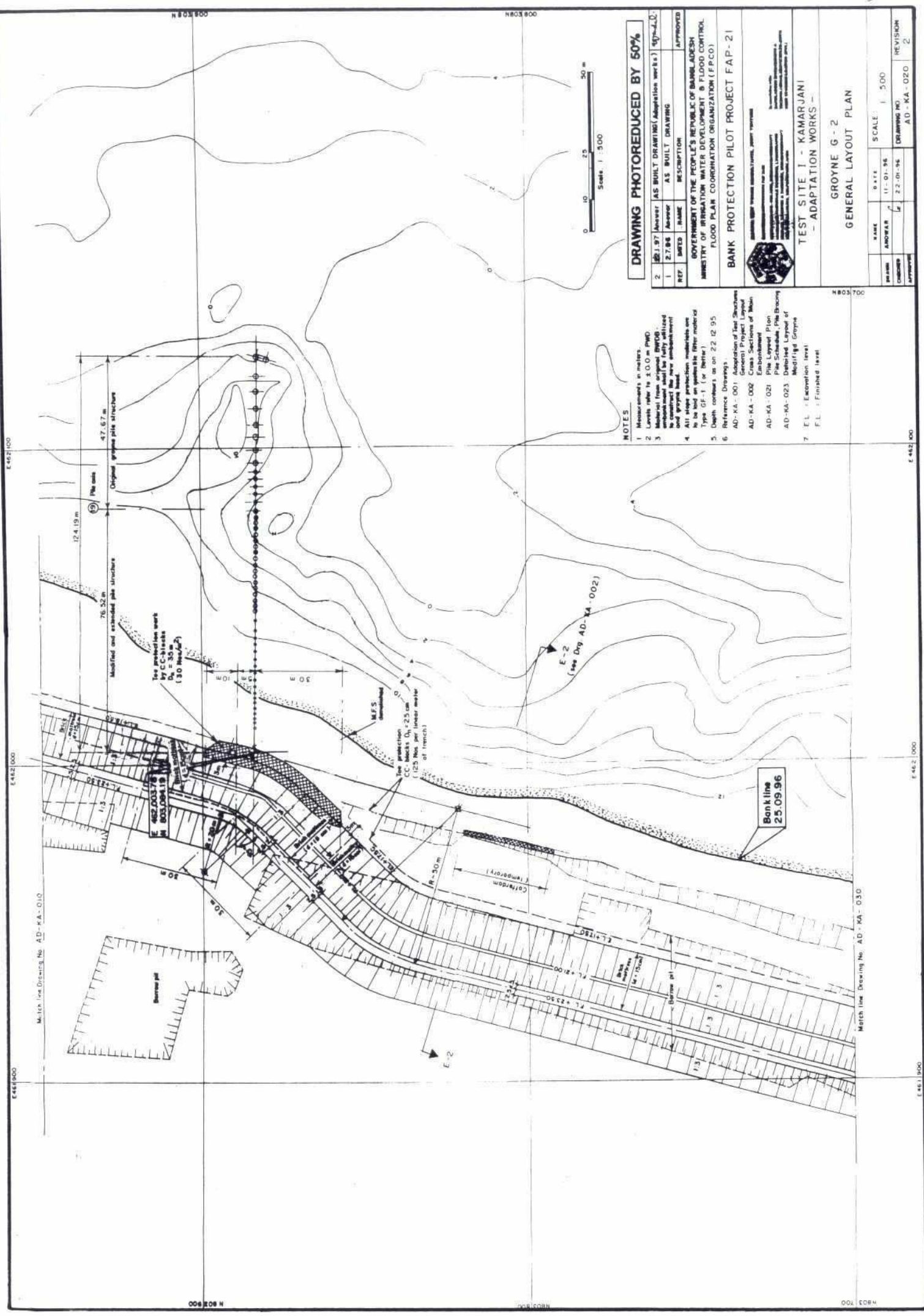
COMBATING CORRUPTION
BANK PROTECTION PILOT PROJECT FAP-21
GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (FPCO)

TEST SITE I - KAMARJANI

GROYNE G - A
CONCRETE SHEET PILE COFFERDAM DETAILS
OF ANCHORING - SHEET II

REV	DATE	NAME	DESCRIPTION	APPROVED	CHECKED	DATE	NAME	SCALE	REVISION
1	25.05.94	As built drawing				12-01-94		1:5, 1:2.5	
						12-01-94			
						10-02-94			

KA-407/II



DRAWING PHOTOREDUCED BY 50%

2	22.1.97	Answer	A.S BUILT DRAWING (Adaptation works)	Approved
1	2.7.96	Answer	A.S BUILT DRAWING	
REF.	DATED	NAME	DESCRIPTION	APPROVED

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (F.P.C.O.)

BANK PROTECTION PILOT PROJECT FAP-21



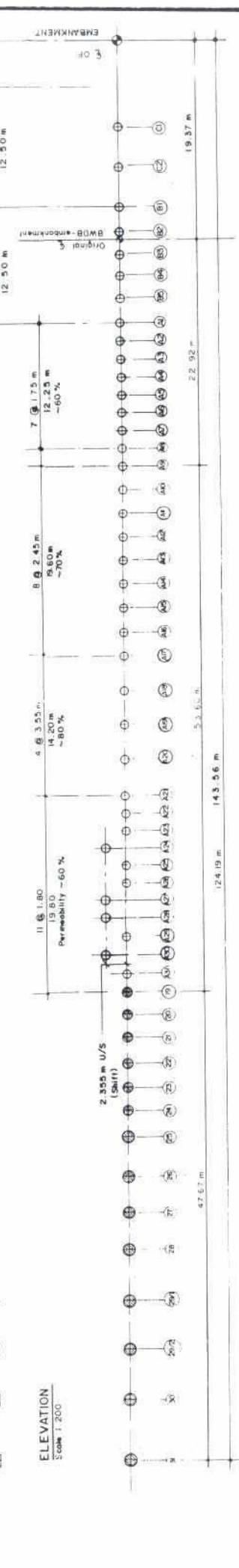
TEST SITE I - KAMARJANI
- ADAPTATION WORKS -

GROYNE G - 2
GENERAL LAYOUT PLAN

	NAME	DATE		SCALE: 1:500	REVISION
DESIGN	ANSWER	11-01-96			
CHECKED	IN	22-01-96		DRAWING NO.	AD-KA-020
APPROVED					2

NOTES

- | | | |
|-----------|--|--|
| 1 | Measurement in meters. | |
| 2 | Levels refer to ± 0.0 m PWD. | |
| 3 | Material from original eyebolt . | |
| 4 | Material from original eyebolt . | |
| 5 | Amount used as fully utilized | |
| 6 | Amount used as partially utilized | |
| 7 | All stage production materials are
to be sold on a system like fiber material
Type GF-1 (or better). | |
| 8 | Depth continues on at 22 ft 95 | |
| 9 | Reference Drawing. | |
| AD-KA-001 | Adoption of Test Sheet | |
| AD-KA-002 | Cross Sections of Soil | |
| AD-KA-021 | The Proposed Plan | |
| AD-KA-023 | Detailed Layout of | |
| 7 | Modified Grays | |
| F.L. | Excavation level | |
| F.L. | Finished level | |



PILE TYPE	LOCATION NO.	NOS	PILE HEAD		PILE POINT LEVEL (m PWD)	TOTAL PILE LENGTH (m)
			TOP LEVEL	AFTER INSTALLATION		
Bored pile Ø 700 mm	A1 - A7	12	+22.90		-1.10	24.00
	B1 - B3					
	C1 - C2	2				

DETAIL - 'X'
Piles at Axes A1 - A31
Scale: 1" = 20'

4	02/98	Amend	As Built (optional)	
3	6/10/97	Finalist	Extension of Pile	
2	25/1/97	Amend	Pile Drawing (Adaptation works)	
1	6/10/96	Finalist	Pile test at No. 31	25/1/97 for 14/1/97
REV	DATE	NAME	DESCRIPTION	APPROVED

REF.	DATE	NAME	ORGANISATION
			GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF WATER RESOURCES WATER RESOURCES PLANNING ORGANISATION (WARPO)
			BANK PROTECTION PILOT PROJECT FAF-21

TEST SITE 1 - KAMARJANI ADAPTATION WORKS (97 / 98)	GROYNE G - 2	PILE LAYOUT PLAN	PILE SCHEDULE
---	--------------	------------------	---------------

SCALE	1:200	REVISION	4
DRAWING NO.	AD-KA-021		
NAME	DATE		
F. LUSSAIN	06-10-96		
DRAWN	CHECKED	APPROVED	

DRAWING PHOTOREDUCTION BY 50%

Bench: 25.09.96
 (Water level + 16.80m PWD)

Cross-Section
 (see Drg AD-KA-004)

Toe protection by
 CC-blocks $D_n = 35\text{cm}$
 50 Nos./m²
 (0.5-bull)

Typical Cross-Section
 (see Drg AD-KA-004)

Total Length 124.19 m

53.60

47.67m

Existing pile structure

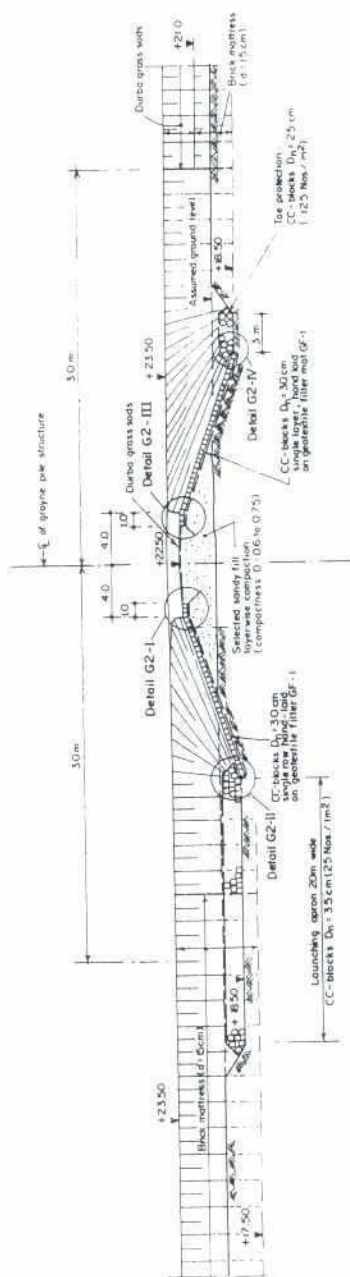
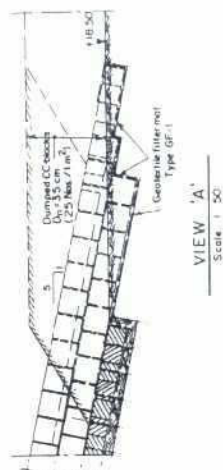
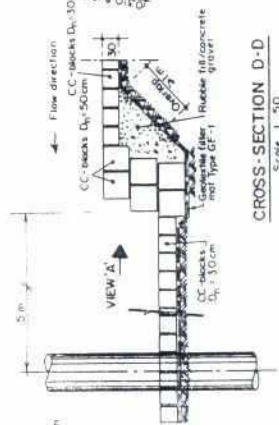
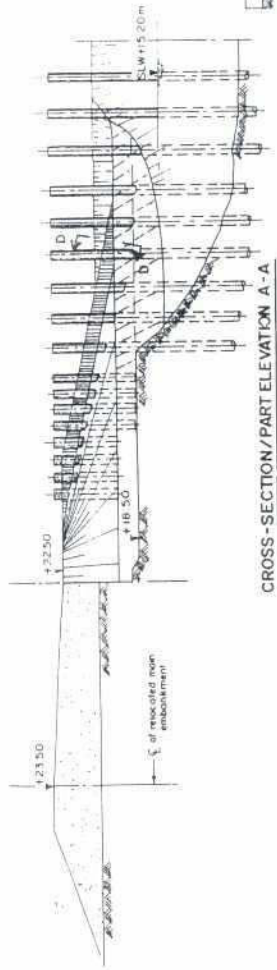
NOTES

- 1 All measurements in meters
- 2 Levels refer to 4.00m PWD
- 3 FL : Finished level
- 4 EL : Excavation level
- 5 Depth contour lines as on 25.09.96
- 6 Reference Drawings:
 AD-KA-002 : Cross Section of
 Main Embankment
 AD-KA-003 : Main Embankment
 Details of Reinforcement
 AD-KA-004 : Modification of Reinforcement
 Above Berm Level
 AD-KA-021 : Pile Structure
 Pile Schedule

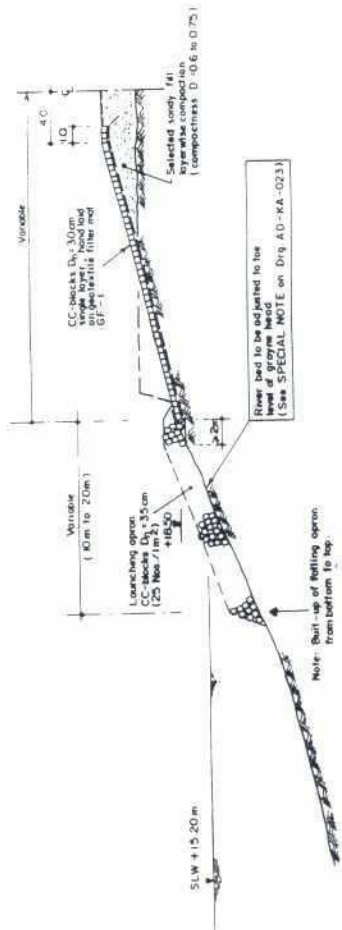


DRAWING PHOTOREDUCED BY 50%

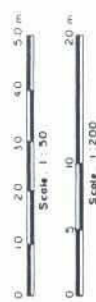
REV	DATE	AS BUILT DRAWING (As per work)	APPROVED
1	25.1.97	AS BUILT DRAWING (As per work)	Approved
GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF WATER RESOURCES WATER RESOURCES PLANNING ORGANISATION (WAR PO)			
BANK PROTECTION PILOT PROJECT FAP-21			
TEST SITE 1 - KAMARJANI ADAPTATION WORKS (1996/97)			
GROUYNE G-2 DETAILED LAYOUT OF MODIFIED GROUYNE			
DRAWN CHECKED APPROVED	NAME ANDWAR 1/97	DATE 08-10-96 15-1-97	SCALE 1:200 DRAWING NO. A.D - KA-02.3/1 REVISION 1



CROSS-SECTION B-B
Scale 1:200



CROSS-SECTION C-C
Scale 1:200



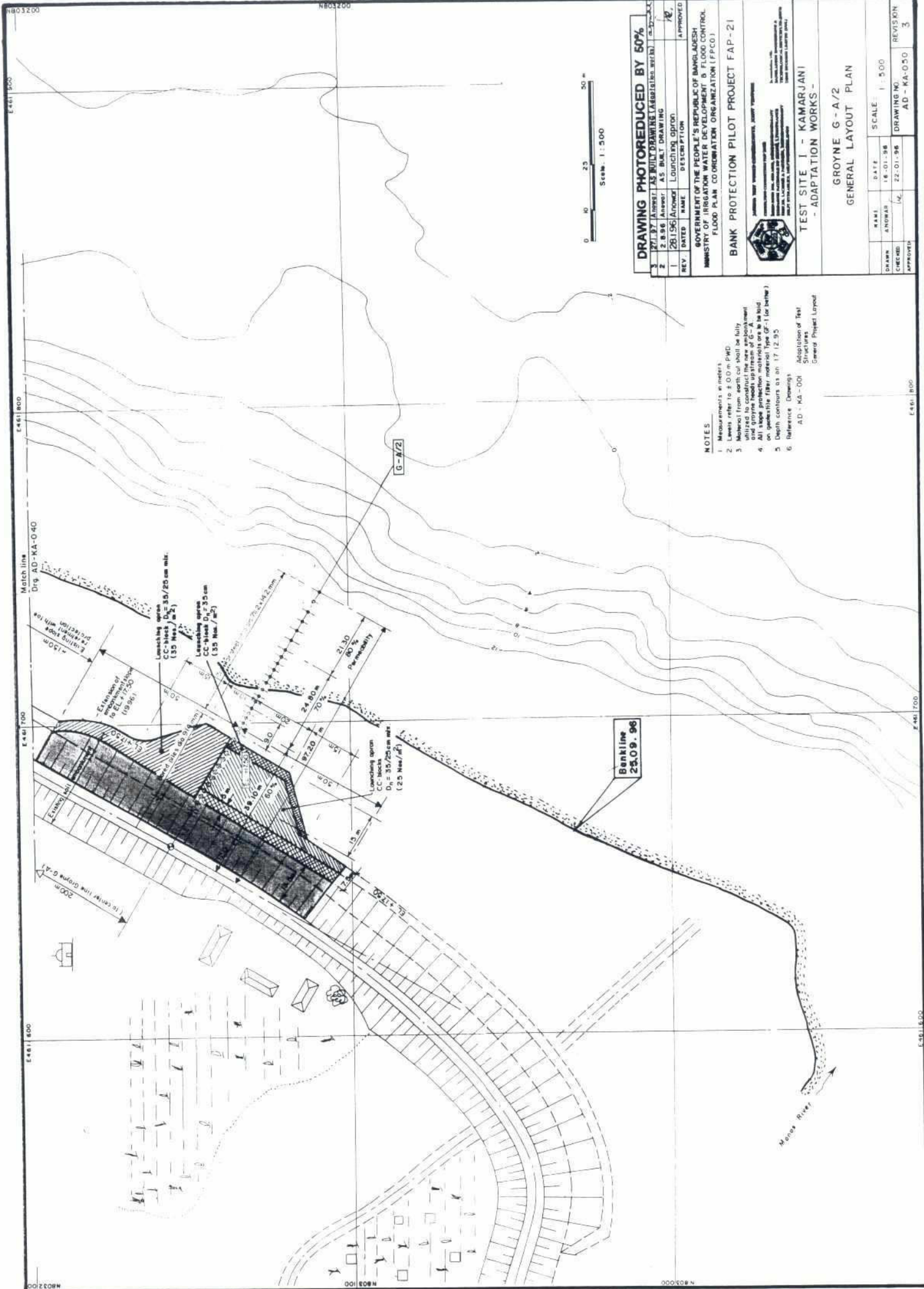
NOTES

1. All measurements in meter.
2. Levels refer to 8.00 m PMD.
3. S.W. = Standard Low Water.
4. E.L. = Erection Level.
5. Filling and composition of sandy soil as per Specifications, Section 900.
6. Filter materials and protection materials and their installation as per Specifications, Section 1000.
7. Location of groyne sections A, B, C and C-C are as per Drg. AD-KA-023.
8. Reference Drawing: AD-KA-021, AD-KA-022, AD-KA-023, AD-KA-025.

Pile Layout Plan
Pile Schedule
Detailed Layout of Modified Groyne
Details of River bank

REV.	DATED	NAME	DESCRIPTION	APPROVED
GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF IRRIGATION, WATER DEVELOPMENT & FLOOD CONTROL FLOOD PLAN COORDINATION ORGANIZATION (FPCO)				
BANK PROTECTION PILOT PROJECT FAP-21				
TEST SITE 1 - KAMARJANI - ADAPTATION WORKS -				
GROYNE G-2 CROSS-SECTIONS OF GROYNE HEAD				
DRAWN	NAME	DATE	SCALE	1:50, 1:200
CHECKED	ANDWAR	25-01-96		
APPROVED		29-01-96		
			DRAWING NO.	AD-KA-024
			REVISION	0

DRAWING PHOTOREDUCTION BY 50%



DRAWING PHOTO REDUCED BY 50%			
REV	DATE	NAME	DESCRIPTION
3	27.12.97	Approved	AS BUILT DRAWING (AS BUILT DRAWING)
2	2.8.96	Approved	AS BUILT DRAWING
1	28.1.96	Approved	Launching groyne

REV	DATE	NAME	DESCRIPTION
1	28.1.96	Approved	Launching groyne

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (FPCO)

BANK PROTECTION PILOT PROJECT FAP-21

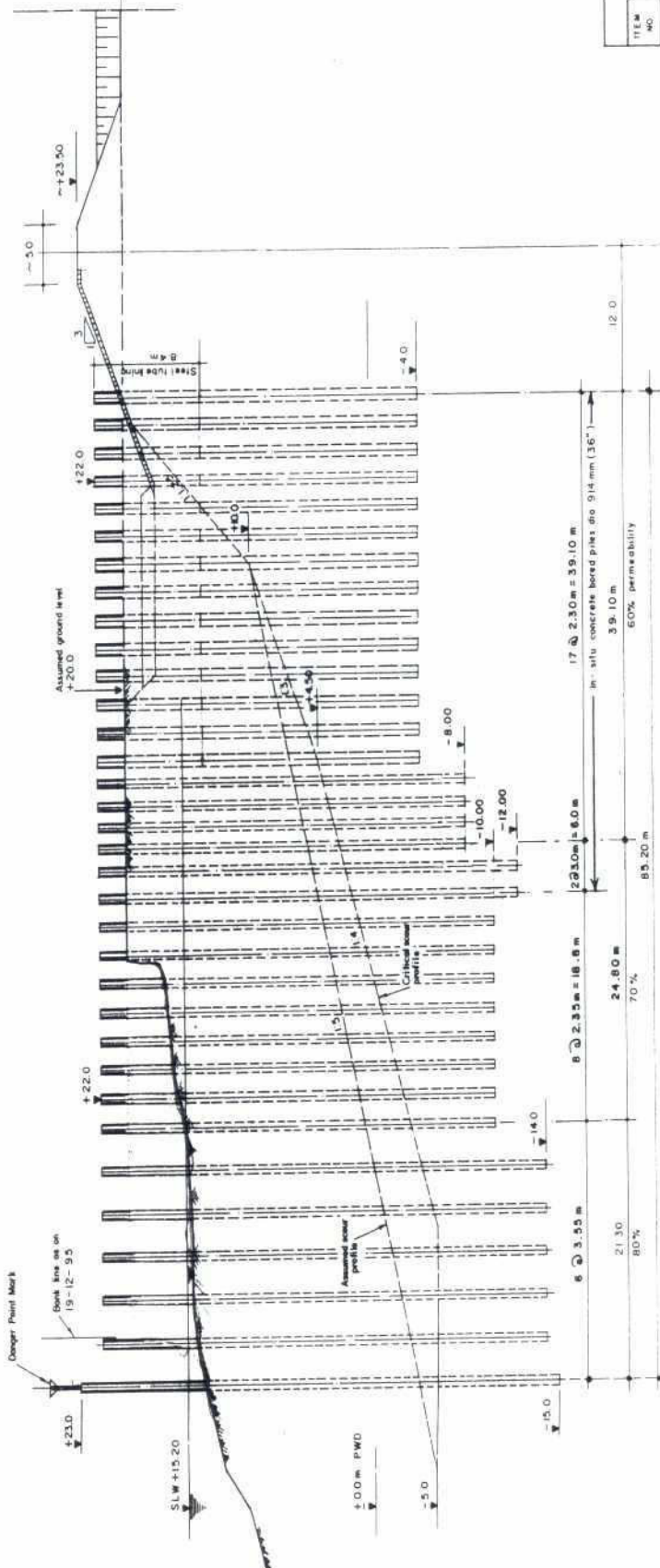
TEST SITE 1 - KAMARJANI
- ADAPTATION WORKS -

GROYNE G-A/2
GENERAL LAYOUT PLAN

NAME	DATE	SCALE
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CHECKED	22.01.96	
APPROVED		

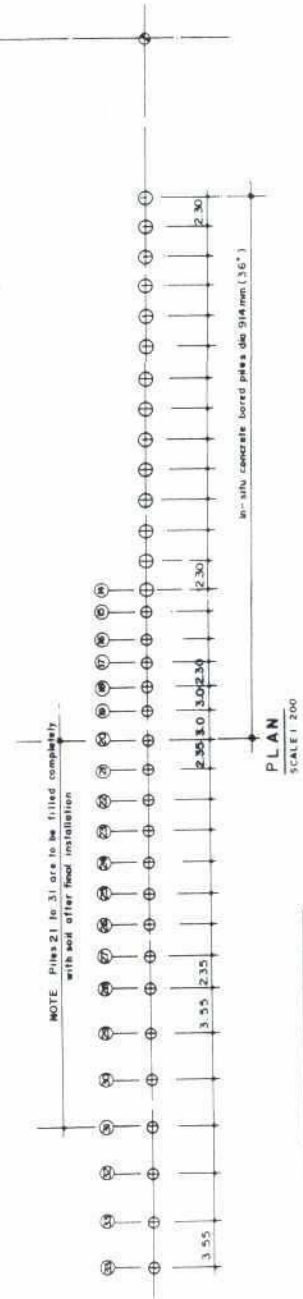
DRAWN	DATE	SCALE	DRAWING NO.	REVISION
AD - KA - 001	18.01.96	1:500	AD - KA - 050	3

- NOTES
1. Measurements in meters.
 2. Levels refer to 4.00 m PWD.
 3. Material from earth cut shall be fully utilized to construct the new embankment.
 4. All slope protection materials are to be laid on geotextile filter material Type GF-1 (or better).
 5. Depth contours at an 17 (2.95).
 6. Reference Drawings:
AD - KA - 001 Adaptation of Test Structures
General Project Layout



ELEVATION
SCALE 1:200

NOTE: Piles 21 to 31 are to be filled completely with soil after final installation



PLAN
SCALE 1:200

PILE INSTALLATION SCHEDULE				
PILE TYPE	LOCATION NO.	NOS	PILE HEAD LEVEL m PWD	PILE LENGTH m
Steel tubular dia 914 mm	1 to 14	14	+22.0	26.0
	15 to 18	4	+22.0	30.0
	19 to 20	2	+22.0	34.0
Steel tubular dia 711.2 x 14.2 mm	21 to 28	8	+22.0	32.0
	29 to 33	5	+22.0	36.0
	34	1	+23.0	38.0

NOTES:

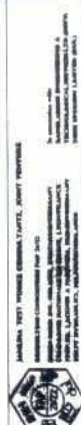
- All measurements in meters
- Levels refer to 20.0m PWD
- SLW = Standard Low Water
- Reference Drawings
- AD - KA - 001: Adaptation of Test Structures General Layout
- AD - KA - 050: General Layout Plan
- AD - KA - 052: Steel Pile Dia 711.2 x 14.2 mm Pile Schedule

DRAWING PHOTO REDUCED BY 50%

2	27.07	Answer	AS BUILT DRAWING (Adaptation work)	APPROVED
1	26.06	Answer	AS BUILT DRAWING	APPROVED
REV	DATE	NAME	DESCRIPTION	APPROVED

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF IRRIGATION, WATER DEVELOPMENT & FLOOD CONTROL
FLOOD PLAN COORDINATION ORGANIZATION (FPCCO)

BANK PROTECTION PILOT PROJECT FAP-21



TEST SITE 1 - KAMARJANI

GROYNE G-A/2

PILE LAYOUT PLAN

PILE INSTALLATION SCHEDULE

DANGER POINT MARK

DRAWN	DATE	SCALE	1:200, 1:20
CHECKED	DATE	DRAWING NO.	AD-KA-051
APPROVED	DATE	REVISION	2

BANK PROTECTION PILOT PROJECT

FAP 21

FINAL PROJECT EVALUATION REPORT

ANNEX 5

**THE GROUYNE TEST STRUCTURE;
PROCUREMENT AND CONSTRUCTION REPORT**

MAY 2001



FAP 21 - BANK PROTECTION PILOT PROJECT

FINAL PROJECT EVALUATION REPORT

ANNEX 5

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ATTACHMENTS

- Attachment-1: Selection of Design and Construction Drawings (Implementation)
Attachment-2: Selection of Design and Construction Drawings (Adaptation)

LIST OF ACRONYMS

ASTM	-	American Society for Testing and Materials
BTM	-	Bangladesh Transverse Mercator (Projection)
BWDB	-	Bangladesh Water Development Board
CC	-	Cement Concrete
DIN	-	Deutsche Industrie Norm (i.e. German Industrial Standard)
FAP	-	Flood Action Plan
FIDIC	-	Fédération Internationale des Ingénieurs-Conseils (International Federation of Consulting Engineers)
FPCO	-	Flood Plan Co-ordination Organisation
GoB	-	Government of Bangladesh
PWD	-	Public Works Datum
WARPO	-	Water Resources Planning Organisation (ex. FPCO)

GLOSSARY

TERM	DEFINITION
bed protection	layered systems placed on filters on a horizontal surface as protection against hydraulic forces and scouring
cover layer	outer protective layer of an embankment revetment or a bed protection
falling apron	multi-layer system of granular material placed directly on the existent subsoil or riverbed
filter	one-layer or multi layer system of well graded granular material or a geotextile or a combination of both
gabions	mattresses and rectangular baskets made from protected steel wire mesh and filled with loose material such as boulders, bricks etc.
geotextile	synthetic fabric (woven, non-woven, needle pinched) applied as a filter or used in tailored geotextile systems (mattresses, etc.)
khoa	brick chips (used as concrete aggregates and filter material)
launching apron	integrated and articulating mattress system placed on prepared slopes above and below water or in horizontal excavation well above SLW
revetment	layered systems placed on filters on a sloping surface as protection against hydraulic forces and scouring
rip-rap	layer of loose stones acting as cover layer in an embankment revetment, a bed protection or a falling apron
toe protection	systems to protect the toe of an embankment against instability due to erosion/scouring



SUMMARY

Well in advance to the design, the Consultant investigated the availability of construction materials in the country. It was found, that most materials required for the construction of efficient erosion protection structures are available or can be reproduced in the country. However, for the implementation of the test structure, geotextile fabrics and steel piles of large diameter had to be imported.

Also the availability of suitable construction equipment was inquired. The required equipment for general works was found in sufficient quantity and quality. For the installation of the groynes piles a 150-ton capacity crawler crane was hired and mounted on a flat barge.

After an open tender procedure, the contract for the civil works was awarded to the Consortium "The Engineers Ltd. & Corolla Corporation (BD) Ltd." in September 1994. The reconstruction of the main embankment was carried out under a separate contract under the responsibility of BWDB. Construction works were executed with mainly local staff.

During the flood season of 1995 the test structure was partially destroyed. After execution of some immediate repair measures, the design of the groynes was improved. The works for adaptation and extension of the groyne field started in December 1995 and were completed in the dry season 1996/97.

1 INTRODUCTION

1.1 AIM OF THE PROCUREMENT AND CONSTRUCTION REPORT

The objectives of the Bank Protection Pilot Project (FAP 21) are to find improved solutions for bank protection works against erosion, by designing, specifying and constructing different types of groynes and revetments using different materials and protective layers and investigating at the same time the suitability of local materials and construction methods. After construction of the test structures at different locations on the Jamuna, their behaviour was to be monitored for a period of several years. Finally suitable design criteria, cost-effective construction methods and maintenance strategies shall be developed and optimised which may serve as future standard solutions, most appropriate for the prevailing conditions at the Jamuna and other main rivers of Bangladesh.

One of the focal points of the study was the selection of sites where the test structures could be built with a high probability of exposure to the river attack during the subsequent Test and Monitoring Phase of the project. The first test site is situated on Jamuna right bank at the pre-selected location near Kamarjani as a permeable groyne structure, and a second one at the Jamuna left bank at the pre-selected location near Bahadurabad Ghat as a revetment structure.

The present Procurement and Construction Report deals with the groyne test structures at the so-called Test Site I – Kamarjani. Summarising and supplementing earlier reports [1] and [2] it shall highlight the

- Initial determination of construction methods and equipment;
- Procedures for procurement of construction materials and equipment;
- Pre-qualification of contractors, tender process and contract award;
- Applied construction methods and experience gained;
- Utilised construction equipment and experience gained;
- Rate of progress achieved;
- Compilation of material and construction cost, and
- Constraints experienced during implementation.

The following Annexes to the Final Project Evaluation Report [3] are supplementing this Procurement and Construction Report and may be consulted for reference:

- Socio-Economic Investigations [Annex 2];
- The Groyne Test Structure – Design Report [Annex 4];
- Evaluation of Hydraulic Loads and River Response [Annex 7], and
- Financial and Economic Evaluation [Annex 12].

The project Test and Implementation Phase started on 15 May 1993, after receiving the formal go-ahead from the Donors and FPCO. According to the agreed work program the works at Test Site I were scheduled to commence by 01 October 1994 and had to be completed within the construction window of the dry season 1994-95, i.e., latest in May 1995.

1.2 PREPARATORY ISSUES RELATED TO THE IMPLEMENTATION OF THE PROJECT

1.2.1 Preliminary Remarks

Before starting the implementation of the works at Test Site I, and as a matter of principle for any other construction site, two essential preparatory tasks had to be fulfilled besides the technical planning and procurement, namely the “Peoples Participation” and the “Land-Acquisition”.

Late or non-attendance to these issues will ultimately hamper, and in the worst case, lead to failure in the timely completion of the project within a given construction window.

1.2.2 Peoples Participation

For a successful implementation the local and directly involved or effected population must be informed about the aim and benefit of the planned project. Various aspects of advantage and disadvantage for the people and the region have to be explained to them and discussed in order to create a project friendly atmosphere. Therefore, prior to and during the land acquisition procedures information of and discussions with the people concerned are of utmost importance and an absolute must.

At the time of launching the “FLOOD ACTION PLAN” the Government of Bangladesh has prepared and published “Guidelines on Peoples Participation” which should be followed closely.

ANNEX 2 provides full disclosure of actions initiated within the FAP 21-project, as well as of positive but also negative response experienced.

1.2.3 Land Acquisition

1.2.3.1 Introduction

The bilateral agreement between the Governments of Bangladesh and Germany/France provides that the land required for the construction of the test structures shall be acquired by GoB through FPCO (now WARPO) and be made available to the Consultant in time before the physical start of construction works.

The Consultants' contribution to the land-acquisition was performed under the socio-economic component of this project. It was the Consultants' responsibility to indicate the desired areas as early as possible. This involves that long before the final determination of the precise structure location (which is only possible when the site conditions after the preceding monsoon season have been verified) the locality as such must be identified and about 50% more land area may have to be included in the preparations for acquisition than finally needed.

Land acquisition under GoB rules is performed by Land Acquisition Officers under the concerned DC-Office. It has been experienced that the land acquisition must be supported, assisted and followed-up closely by the Consultant to identify any constraints that would ultimately cause unnecessary delay in the commencement of construction works.

1.2.3.2 Selection of the Test Site Area

As said before, one of the focal points during the study phase of the project was the selection of sites where the test structures could be built with a high probability of exposure to the river attack during the entire Test and Monitoring Phase of the project. Thereby, through an analysis of the plan-form data and bank erosion rates presented by a satellite image of 8 March 1992 and images over the past 20 years period, the pre-selected test site area at Kamarjani was reconfirmed, also taking due account of environmental and socio-economic aspects. After the flood season of 1992, the Consultants continued to monitor the bank erosion around the Kamarjani area. The observations were compared with the earlier predictions and although some deviations could be noticed the general trend appeared to be in line with the predictions.

The Consultants continued to monitor the development within the selected test site area through analysis of more recent satellite images and detailed survey. The evaluation of satellite images of 1993 and the comparison with the previous investigations confirmed the earlier predictions. The morphological changes were analysed and future predictions for the area of the proposed Test Site I "Kamarjani" supported in a separate report [4].

Based on the results it was finally concluded in November 1993 between the Donors, FPCO and the Consultants that Test Site I with a series of permeable groynes shall be constructed immediately North of the mouth of the Ghagot river.

The test site Kamarjani is situated on the right bank of the Jamuna river at an approximate Latitude of 25°20'N (BTM 802 800 to 804 600 North and 461 600 to 462 200 East).

The location and general layout of the test structure as per initial layout and design is shown in Drawing No. KA-003/1, Attachment 1.

1.2.3.3 Land Acquisition Procedures

The main objective for the Consultant under the land acquisition procedure was to ensure timely hand over of the required areas of land. The following steps were initiated:

- a. Identification of the area where the test structure was about to be located;
- b. Preparation of a map for the area to be acquired;
- c. Collection of "mouza maps" (land register maps);
- d. Formation of a "Land Acquisition Committee" (LAC);
- e. Determination of number of households, houses, buildings and other installations;
- f. Determination of crop standing in the field;
- g. Estimate of land value, cost of house shifting and crop compensation;
- h. Comparison of the GoB assessment for land purchase, house shifting and crop compensation with the own estimate to avoid overestimation, and
- i. Accelerated payment of compensation to the people involved to ensure earliest clearance of the construction site areas.

1.2.3.4 Experienced Constraints

In the sector of bank protection projects it is practically impossible to start the land-acquisition procedure months or even years ahead in order to follow all legally necessary steps prior to the start of the construction works.

It has been experienced, however, that the landowners, being aware of the benefits of the project through early peoples participation, are allowing to occupy their land already after proper assessment of the value of the land including house shifting and crop compensation and after having received down payments for house shifting and crop compensation. In order to accelerate these processes such compensation has been advanced by the project. Finally, and after all legal procedures had been completed, final payments were arranged by the GoB to the affected people.

It is a special experience gained in rural areas of Bangladesh in this project, that landowners and other members of the local population try to get as much as possible influence in and advantage out of construction works on their lands. If not satisfied in their demands serious problems may arise, which ultimately may lead to suspension of the works.

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Photo 1.2-1: Groyne G-1



Photo 1.2-2: Groyne G-2



Photo 1.2-3: Groyne G-3



Photo 1.2-4: Groyne G-A



Photo 1.2-5: Groyne G-B/1



Photo 1.2-6: Main Embankment

2 DESIGN OF THE TEST STRUCTURE

For the design of the test structures Annex 4 may be consulted, which contains all original design data and principles related to the permeable groyne structures and associated works.

For the purpose of this Construction and Procurement Report a selection of design and construction drawings is included under Attachment 1.

3 LOCAL EQUIPMENT AVAILABILITY

3.1 General Equipment

Experience has shown that availability of construction equipment in Bangladesh is very rare. Available equipment is generally in poor condition and badly maintained. It is common practice in Bangladesh that a contractor when offering for or being awarded of a contract starts "looking around" for suitable equipment on lease base from GoB organisation or private companies.

Therefore it is very important and crucial for design and planning of a construction to investigate well in advance the availability and the condition of suitable equipment for the intended kind of construction.

The Consultant already during the Study Phase in 1992 investigated the availability of construction equipment. In search for cranes, bull-dozers, graders, excavators, concrete mixers, pumps of different capacities the Consultant visited several equipment yards and construction sites of private companies and Government organisations all over the country. Most of the equipment found was in bad or non-operational condition. Some key equipment like crawler cranes up to 60 ton capacity and excavators found would have needed major repair and upgrade to make it fit for any construction works.

At this stage of the project it was even considered to import the main key equipment, the heavy piling crane from outside Bangladesh. But finally a 150-ton capacity Manitowoc crawler crane was found, which needed, however, important repair and upgrading to be suitable for the envisaged works (ref. to Subsection 7.3)

3.2 Floating Equipment

In Bangladesh most goods are transported on the rivers. In the coastal region and the rivers up to Dhaka smaller cargo vessels and large country boats are the dominant types of transport. In the rivers further upstream medium and small country boats are used.

Transport on tugged or self-propelled barges is less common. Barges are mainly used as working platforms.

The Consultant during his pre-investigation in the availability of equipment found quite a number of barges and tugboats, but all in more or less bad conditions. Mooring winches of sufficient capacity could not be found.

Anchors up to 3 tons were found with the ship wreckers in Chittagong.

Together with the 150-ton crawler crane mentioned under Section 3.1 a flat barge of sufficient capacity of 400 ton was offered. The barge had worked with this crane already but its deck was too weak to accommodate the crane with its maximum counter weight and the heavy piling equipment and to support the working forces (piling, anchoring).

To make the barge suitable substantial reinforcement of the barge body was necessary. The equipment set up finally implemented by the Consultant is shown in Fig. 3.2-1.





Photo 3.2-1: Floating equipment:

- Manitowoc crawler crane (150 ton capacity)
- Float Barge (BG7) 400 ton capacity



Photo 3.2-2: Floating equipment:

- Manitowoc crawler crane (150 ton capacity)
- Float Barge (BG7) 400 ton capacity

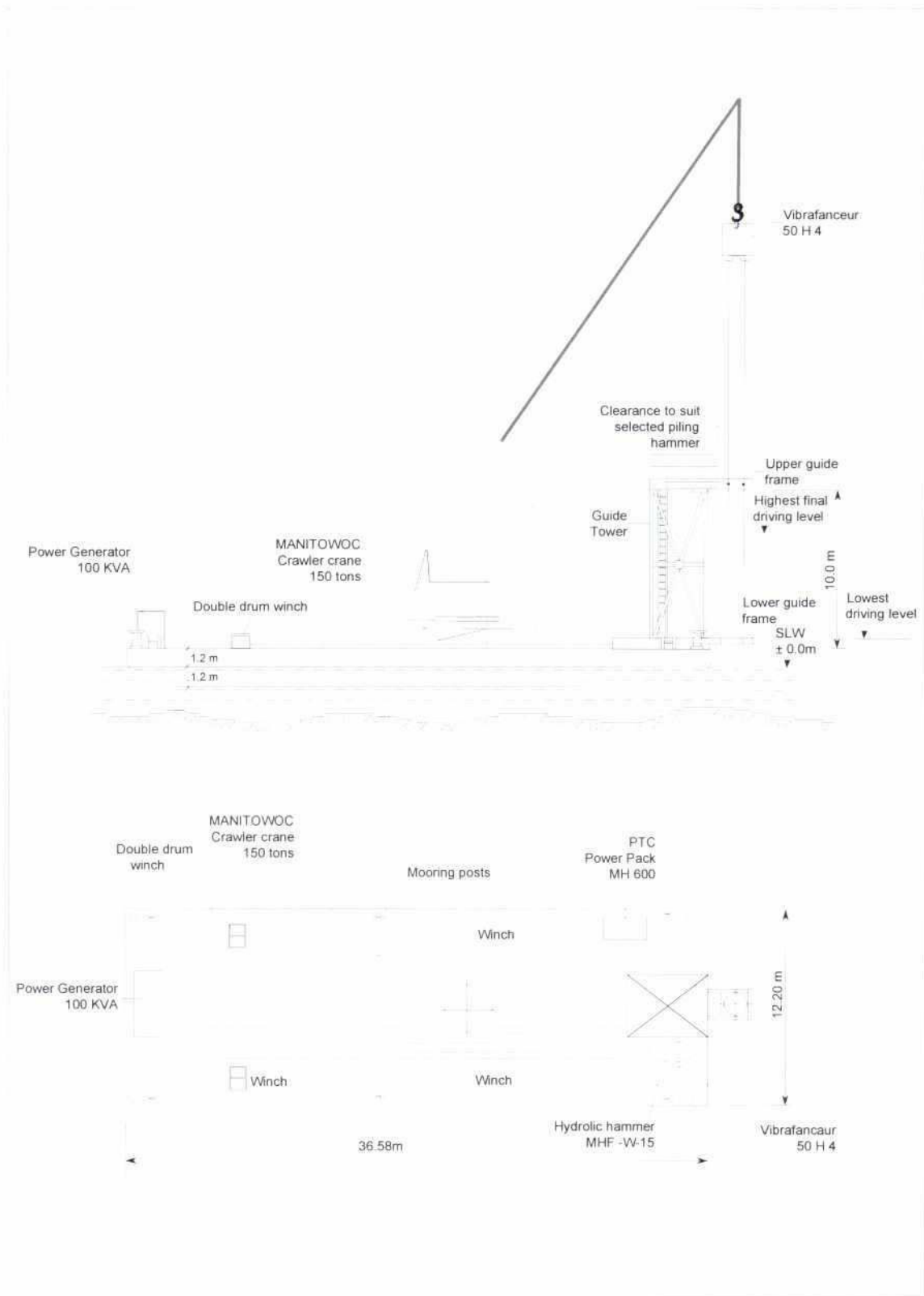


Fig. 3.2-1: General Layout of Piling Barge BG-7

4 SOURCES OF CONSTRUCTION MATERIALS

4.1 LOCAL MATERIALS

4.1.1 Bricks

Bricks are the most common construction material in Bangladesh and used in all fields of construction. Bricks are brick-field manufactured all over the country from clay excavated near the brick-field and burned with coal or fire-wood. Standard brick size is 240 x 120 x 70 mm.

Bricks are classified in 3 quality classes:

- First class bricks are sound, hard, evenly burned, well shaped and free from cracks;
- Second class bricks are almost same quality but irregular shaped and less evenly burned, and
- Third class bricks are soft bad shaped and not evenly burned.

For brick walls, brick masonry and quality Khoa (bricks cut into brick-chips/stone-chips) as aggregate for concrete only first class bricks should be used. For concrete of lower strength second class bricks will do. Third class bricks are used in local road construction.

Depending on the quantity of first class bricks available in an economic distance to the site one has to accept second class bricks well selected.

Cutting of bricks by hand is most common, but machine crushing is coming up too.

For large demands of bricks it is recommended to procure well (1year) in advance since brick production start only end November/early December and first deliveries start mid December. Only minor quantities are available in the market during and at the end of the monsoon season.

4.1.2 Concrete Aggregates

Concrete aggregates are available in three categories:

1. Khoa from bricks (ref. to Subsection 4.1.1).
2. Gravel hand collected from the rivers in Bangladesh (northern parts of the country only) and sorted to size classes.
3. Crushed aggregates from larger river stones (boulders) cut to size classes. Size and quantity of boulders are limited in Bangladesh. Therefore crushed aggregates from boulders imported from India and Bhutan are found in the market. Boulder cutting is mainly done by hand.

Sand is commonly taken from borrow pits, riverbed or seabed. The quality differs very much and in most areas sand from borrow pits is of low quality due to grain size, uniformity and mica content. Sand from the riverbeds is of better quality. High quality coarse sand is only available from the northern rivers of Sylhet and Panchagarh. Testing of sand prior of utilization is essential. Sieving, washing and mixing of different kinds/qualities may help to find the desired specification.

4.1.3 Cement

Cement of reasonable standard quality is produced in Bangladesh from imported clinker. At the beginning of the project the produced quantity was very small and therefore mainly imported cement from India, Indonesia and China was in use. In the meantime the production in Bangladesh has increased considerably and it is reported that market demand will be covered soon.

Attention is given to the fact that some times the market is flooded with altered cement. Cement is only available in 50 kg bags.

4.1.4 Pre-cast Reinforced Sheet-Piles

As demanded in the ToR as much as possible local construction material should be used and tested. Therefore the Consultant designed a pre-cast reinforced concrete sheet pile with groove and tongue with the dimensions of 490 x 250 x 8,000 mm. Dependent on the structural requirement, such precast cc-sheet piles could be factory produced up to 10 m.

These cc-sheet piles were produced in a modern factory for prefabricated cc-elements at Panchagarh in the northwest of Bangladesh.

Very accurate moulds had to be prepared to limit tolerances and to guarantee precise groove-tongue joints and straight pile points. High quality concrete (B45, DIN 1045) and steam curing were applied to achieve early and high strength. (The typical design is presented in Drawing No. KA-405, Attachment 1).



Photo 4.1-1: Pre-cast Reinforced Sheet Piles

4.1.5 Pre-stressed Spun Concrete Piles

Out of the same reason as mentioned above the Consultant designed pre-stressed spun concrete piles of diameter 500 mm and a manufacturing length of 10 meters. The piles were manufactured in the same factory as above mentioned. The factory already had extensive experience in producing conic spun piles for electric power transmission lines or light poles with modern, well-maintained equipment.

Although the spun equipment allows 12 m length, the pile length was limited to 10 m due to road transport regulations. To be able to produce 20 meter long piles the 10 m pile units received a

pecially developed pile joint with steel plates anchoring to the pre-stressing steel wires. To achieve early and high strength (B45, DIN 1045) the spun piles were steam cured.

This was the first time in Bangladesh to have manufactured pre-stressed piles with a special proven design for the pile joints (The typical design is presented in Drawing No. KA-403, Attachment 1).



Photo 4.1-2: Pre-stressed Spun Concrete Pile; Bending Test



Photo 4.1-3: Pre-stressed Spun Concrete Pile; Bending Test

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4.1.6 Granular Filter Materials

Granular filter is produced in classes from river gravel, crushed aggregates, khoa and sand. It is not ready made available in the market but must be mixed with the available ingredients according to demand/specification.



Photo 4.1-4: Storage of Granular Filter Materials

4.1.7 Stones and Boulders

Stones of size up to 20 cm are found in the northern rivers. Boulders are to be imported. (Ref. to Subsection 4.1.2), whereby sizes dia.15 .cm to dia 35 cm are more commonly available selection of larger sizes. Rubble is a mixture of different sizes of river stones or cut boulders. Rip-Rap is normally from boulders only.



Photo 4.1-5: Storage of Boulders

4.1.8 Hard Rock

At the start of this project hard rock was not available in Bangladesh except imported on demand from India, Bhutan or even Indonesia.

Hard rock has become available recently from a mining pilot project in the northwest of Bangladesh near the city of Dinajpur. Produced size and quantity up to today is reported to be small.

4.1.9 Corrosion Protection Coatings

Corrosion protection coatings for steel structures and steel piles are being produced in limited variations on paint basis by local companies. Specialized, long-lasting protective coats have to be imported or ready-coated products have to be procured.

4.1.10 Wire Mesh

Wire mesh is produced locally, hand made from galvanized or non-galvanized steel wire. The quality of galvanization and the mending of the wire mesh is often poor. For gabions, etc, ready made wire mesh cages should be imported.



Photo 4.1-6: Production of Wiremesh at site

4.1.11 Reinforcement Steel

Reinforcement steel in standard quality is available in the market up to 28 mm. Both mild steel and deformed steel bars are offered at reasonable price and in sufficient quantities. Production is from re-rolled steel from ship wrecking or from imported billets. Care must be taken to procure and have delivered the correct quality.

4.1.12 Structural Steel

Availability of new structural steel is limited to flat steel, small L-profiles and U-profiles. Other and heavier profiles are found as used material in limited quantities. Larger, heavier and H-profiles have to be imported.

4.1.13 Tubular Steel Pipes

Pipes are available in the water and gas sector in standard qualities and dimensions. The largest ready-made pipe available is a spiral welded gas pipe of diameter 200 mm. On demand pipes can be rolled and welded from steel plates up to wall thickness of 14.2 mm (steel grade St.37 or equivalent), diameter of 1800 mm and section length of 2.40 meter. Tubular steel piles can be locally produced by joint-welding of such section lengths.

4.2 IMPORTED MATERIALS

4.2.1 Tubular Steel Piles

In order to secure delivery in time of the designed tubular steel piles, local manufacture as per Subsection 4.1.13 was disregarded for the purpose of this project. The Consultant tendered and imported the steel piles from France.



Photo 4.2-1: Imported Tubular Steel Piles stored at the welding yard



Photo 4.2-2: Imported Tubular Steel Piles stored at the welding yard

4.2.2 Steel Sheet-Piles and Anchor Materials

Steel sheet piles and anchor material are not being produced in Bangladesh and have to be imported. The Consultant tendered and procured these items as per design and specification from France.

4.2.3 Heavy Steel Sections

Heavy H-type rolled steel sections as are required for the bracing of permeable groyne structures are not available in Bangladesh and have been imported from France.

4.2.4 Geo-Textile Filter Materials

Geo-textile filter material is not being produced in Bangladesh but limited types and quantities of imported material are available with sales agents. All types and quantities can be procured through local agents or by direct import. The Consultant procured the Geo-textile from Germany.

The Bangladesh Jute Industry together with the Jute Research Institute tried to produce durable filter mats from jute but failed until start of the constructions works in Kamarjani.

For test purposes a special composite (sandwich) filter mat from jute from Bangladesh as bottom layer and synthetic geo-textile as upper layer was manufactured in Germany. The mat was filled with original sand from the Jamuna to make the mat heavy enough for sinking to the riverbed.

4.2.5 Future Reproducibility of Imported Materials in Bangladesh

Availability of materials and products depends in Bangladesh, like in all other countries of the world, on the availability of appropriate raw materials and on demand.

With the exception of steel sheet pile and anchoring materials, all other materials which this project has imported due to non-availability in the local market would be reproducible in Bangladesh, if the respective demand would call for it.

Geo-textile materials could be produced by the local Jute and Carpet Industry with quite little investment costs. Such investment would be feasible if Bangladesh continues to use more and more geo-textile filter material in road and embankment works. Even combined products with natural jute material would be competitive products.

Steel tubes/pipes/piles (spiral welded or longitudinal welded) of all length, diameter and thickness can also easily be manufactured in Bangladesh from locally produced or from imported steel plates. Investment has to be made in production lines (steel plate bending machines) and modern automatic welding facilities.

Import of larger river stones (boulders) may in future be replaced by hard rock from local mining (see Subsection 4.1.8), provided the volume and exploration cost would suit the anticipate demand for river bank protection works.

5 PROCUREMENT OF MATERIALS AND EQUIPMENT

5.1 GENERAL INFORMATION

Most construction works of bank protection projects have to be executed during a construction window defined by the low water period between the monsoon seasons. Procurement and construction contracts can normally be awarded only after the final decision regarding location and design of the respective structure is approved. This is possible at the end of say October but more likely only after the end of monsoon season say by end November / early December. With a construction window expected to last from about mid October to mid April it would be too late to start the procurement of materials and equipment at this stage. Consequently, advance procurement under separate contracts or by the implementing authority have to be made in order to avoid delays for lack of materials and equipment. In case of procurements from outside Bangladesh sufficient time for shipping, custom clearance and payment of duties by the Client (or GoB) must be allowed.

Foreign firms do not have general import licenses and have to apply for import permits for every single supply contract through the implementing authority and other authorities, which can be a time consuming process.

Delay in cargo handling, customs clearance and duty payment often causes demurrage without any body taking responsibility. So finally the project has to pay the involved costs to get the cargo cleared and the works started.

In anticipation of the foregoing the Consultants in their function as Main Contractor initiated material as well as equipment procurement inside and outside of Bangladesh well in advance through individual supply contracts. The items were procured in the name of the Consultants and GoB through FPCO arranged for the payment of import duties and taxes. It was for this strategy that materials and essential construction equipment could be made available to the Sub-contractor for the execution of the works in due time.

For procurement of bricks for concrete, matressing or other purpose it must be considered that burning of bricks is being carried out only during the dry season wherefore new supplies are usually not available before mid December. Commonly only minor quantities of bricks are available in the market from previous productions at the end of the monsoon season. Therefore it was also important to procure even bricks much in advance of the start of construction works.

All procurement and material supplies followed the technical specifications elaborated for the purpose of the project and as also included in the Tender Documents for the construction works for information of the respective civil works contractor.

5.2 CONSTRUCTION MATERIAL PROCUREMENT

5.2.1 Local Procurement

In the following only the main material supplies from within Bangladesh are compiled, more details are presented in Table 5-P1.

- (a) **Pre-stressed spun concrete piles**, dia. 500 x 100 mm, length 10m,
ready for taking-over by the Subcontractor.
Supplied quantity: 142 Nos.

Procurement cost (without VAT)	Taka	3,683,559
Equivalent to	DEM	141,831

- (b) **Reinforced concrete sheet piles**, size 490 x 250 mm, length 8 m, stockpiled at Chilmari, ready for taking-over by the Sub-contractor.

Supplied quantity:	316	Nos.
Procurement cost (without VAT)	Taka	6,165,416
Equivalent to	DEM	237,391

- (c) **Boulders for rip-rap** in various gradation ranges, stockpiled at Chilmari, ready for taking-over by the Sub-contractor.

Supplied quantity:	4,660	m ³
Procurement cost (without VAT)	Taka	7,692,763
Equivalent to	DEM	299,054

5.2.2 Imported Materials

In the following only the main material imports into Bangladesh are compiled, more details are presented in Table 5-P2.

(a) Country of Origin France

- **Tubular steel piles** (dia. 711x14.2 mm, 1016x20 mm and 1220x20 mm)
Supplied quantity: 1,300 t
Procurement cost (cif Chittagong) DEM 1,685,180 (without VAT)
Equivalent to Taka 43,814,671 (without VAT)
- **Steel sheet pile material**, profile ARBED PU-6
Supplied quantity: 92 t
Procurement cost (cif Chittagong) DEM 121,013 (without VAT)
Equivalent to Taka 3,146,338 (without VAT)
- **Anchor material for steel and concrete sheet pile cofferdams**
Supplied quantity: 22.7 t
Procurement cost (cif Chittagong) DEM 78,907 (without VAT)
Equivalent to Taka 2,051,563 (without VAT)
- **Structural steel materials**
Supplied quantity: 44 t
Procurement cost (cif Chittagong) DEM 46,021 (without VAT)
Equivalent to Taka 1,196,540 (without VAT)

(b) Country of Origin Germany

- **Geotextile filter materials and sand bags/containers**
Supplied quantity: 18,360 m² (filter mats)
27,500 Nos. (sand bags)
Procurement cost (cif Chittagong) DEM 247,393 (without VAT)
Equivalent to Taka 6,432,220 (without VAT)

5.3 COST OF CONSTRUCTION EQUIPMENT

5.3.1 Local Procurement

Local equipment procurement was limited to purchase of Consultants' camp, survey boat with outboard engines and wireless communication equipment. Details are compiled in Table 5-P3.

Hire of the 150 ton crawler crane, barge mounted with accessories as well as additional local cost incurred in connection with local tailoring of geo-textile sand bags, CAR-insurance and port charges, forwarding and demurrage in connection with imports for the project, all of which are compiled in Table 5-P4.

5.3.2 Import of Construction and Ancillary Equipment

As outlined under Chapter 3 it was required to import construction equipment that was not available within Bangladesh at the time of project implementation. Besides, ancillary equipment such as mooring winches for upgrading the locally hired piling barge, special survey-monitoring equipment, etc. had to be arranged from overseas.

To the possible extent these items were procured from France in compliance with the terms of the project. Of all imported equipment the pile installation gear was the most expensive one:

(a)	Pile driving equipment		
	(cif Chittagong or Mongla Port)	DEM	1,758,588
(b)	Equipment for pile welding at site		
	. preparation, testing	DEM	102,478
	. bevelling, welding	DEM	266,235
	Total Equipment Import (France)	DEM	2,127,301

More details may be taken from Table 5-P5 (Procurement of Equipment Outside Bangladesh) and Table 5-P6 (Other Procurement Outside Bangladesh).

5.4 DELIVERY OF MATERIALS AND EQUIPMENT

The imported materials and equipment procured by the Consultants were delivered at Chittagong and Mongla Sea Ports or Dhaka Airport and transported by Consultants' sub-contractors to the site.

The suppliers of the local materials mainly delivered directly to the site respectively to strategic storage yards.

During the month of November 1994 all material and equipment imported by the project were delivered to the site except the geo-textile filter material, which could be delivered only in the middle of January 1995 after being held at Mongla Port for several months.

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Procurement

Test Site 1, Kamarjani, 1994/1995

Procurement of Construction Material in Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10
1 1a	Prestressed Spun Concrete Piles Ø500x100mmx10m Free Chilmari/Raselpur	GEMCON, Panchagar	1994	142	No.	25,500	3,683,550	141,830	incl. spare material
2 2a	Reinforced Concrete Sheet Piling Sheet piles Free Chilmari	GEMCON, Panchagar	1994	316	No.	20,750	6,165,425	237,392	Incl. spare material
3	Stones for Revetments / Bed Protections (ø = D50)						9,848,975	379,222	
3a	Range B, ø15cm	Exchange	1994	160	m³	1,651	264,703	10,290	
3b	Range C, ø20cm	International		850	m³	1,651	1,403,078	54,544	
3c	Range D, ø25cm	Ltd., Dhaka		1,350	m³	1,651	2,228,418	86,629	
3d	Range E, ø30cm			2,300	m³	1,651	3,796,564	147,590	
4 4a	Ceramic Backing Ceramic rings for pile welding	Bangladesh Oxygen Dhaka	Dec-94	180	No.	2,253	405,600	15,756	
5 5a	Geo-Jute Material Geo-jute filter material 1.22Lbs	Adamjee Jute Mills Dhaka	Feb-95	9000	m²	7.67	69,000	2,516	
6 6a	Geo-Textile Bags Geo-textile bags	DIRD Private Ltd Dhaka	Jul-95	3040	No.	250	760,000	26,569	
TOTAL MATERIAL PROCUREMENT IN BANGLADESH							18,776,338	723,117	

Table 5-P1: Procurement of Construction Material in Bangladesh

Procurement

Test Site 1, Kamarjani, 1994/1995

Procurement of Construction Material outside Bangladesh

page 1 of 2

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10	11
1	Geo-Textile Materials									
1a	Filter Mat Terrafix 1004 R	Naue Fasertechnik Germany	Aug-94 Oct-94	5,120	m ²	125	4.82	1,894,838	72,878	
1b	Filter Mat Terrafix 1004 RB			1,620	m ²	348	13.40	564,408	21,708	
1c	Filter Mat Terrafix 1004 RB Jute			1,620	m ²	368	14.15	595,998	22,923	
1d	Secutex Sand Bags			27,500	No.	109	4.21	3,010,150	115,775	
1e	Sewing Machines			4	No.	119,600	4,600.00	478,400	18,400	
1f	Tread			140	rolls	624	24.00	87,360	3,360	
								6,631,154	255,044	
								6,432,220	247,393	less 3% reduction
2	Tubular Steel Pile Material									
2a	Spiral Welded Pipe 711x14.2 mm	Starval France	Jul-94 Oct-94	1,287	m	6,361	244.67	8,187,158	314,891	
2b	Spiral Welded Pipe 1016x20 mm			672	m	12,726	489.44	8,551,548	328,906	
2c	Spiral Welded Pipe 1220x20 mm			1,080	m	14,806	569.46	15,990,440	615,017	
2d	Spiral Welded Pipe 1220x14.2 mm			36	m	13,673	525.89	492,237	18,932	
2e	Reinforcement Ring 711 mm			53	No.	9,162	352.40	485,604	18,677	
2f	Reinforcement Ring 1016 mm			21	No.	13,313	512.03	279,569	10,753	
2g	Reinforcement Ring 1220 mm			29	No.	15,140	582.31	439,062	16,887	
2h	Transport c/c Chittagong			1,297,652	kg	7	0.28	9,389,054	361,117	
								43,814,671	1,685,180	
3	Steel Sheet Piles & Anchor Material									
3a	Steel Sheet Piles, ARBED PU-6	Midi Acier France	Jul-94	92,009	to	34,196	1,315	3,146,338	121,013	
3b	Driving Cap (686 kg)			1	No.	109,967	4,230	109,967	4,230	
3c	Anchor Plates, Turnbackle, Bolts etc (for Steel Sheet Pile Cofferdam, G-1)			20,229	to	-	-	1,519,974	58,461	
3d	Anchor Plates, Turnbackle, Bolts etc (for CC Sheet Pile Cofferdam, G-A)			4,114	to	-	-	531,589	20,446	
								5,307,868	204,149	
								55,554,759	2,136,722	

to be carried forward

Table 5-P2-1: Procurement of Construction Material outside Bangladesh

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Procurement

Test Site 1, Kamarjani, 1994/1995

Procurement of Construction Material outside Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10	11
4	Structural Steel Material									
4a	Steel Sheet Plates	S.A. Lille Acier	Jul-94	1.026	to	40.459	1,556	41,511	2,136,722	
4b	Checkered Steel Sheet Plates	France		4.400	to	25.754	991	113,317	1,597	
4s	Steel Girders HEB 140			19.209	to	20.992	807	403,228	4,358	
4d	Steel Girders HEB 200			1.471	to	22.668	872	33,344	15,509	
4e	Steel Girders HEB 300			11.232	to	22.668	872	254,605	1,282	
4f	Steel Girders HEB 360			6.816	to	23.582	907	160,735	9,793	
4g	ClF			44.144	to	-	-	189,800	6,182	
								1,196,540	7,300	
									46,021	
5	Testing of Tubular Steel Piles									
5a	Acceptance Test	Thyssen Klönne Germany	Aug-94				1s	124,306	4,781	
TOTAL MATERIAL PROCUREMENT OUTSIDE BANGLADESH								56,875,606	2,187,523	

Table 5-P2-2: Procurement of Construction Material outside Bangladesh (cont.)

Procurement

Test Site 1, Kamarjani, 1994/1995

Procurement of Equipment in Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10
1	Survey Boat								
1a	Unsinkable boat	Ahmed's Trading, Dhaka Dynaglass, Singapore	May-94				835,423	33,620	incl. spare parts
2	Container Camp								
2a	20' and 40' containers	ACORN, Dhaka Seamore BV, Netherland	Jun-94	16	No.		4,423,938	175,511	
3	Communication Equipment								
3a	Transceiver HF, FT 180A	Logistic & Services		4	No.	56,829	227,317	8,743	incl. accessories
3b	Transceiver VHF, FTL 2011C	Dhaka		4	No.	16,838	67,354	2,591	incl. installation
3c	Transceiver VHF, FTH 2010	Yeastu Musen, Japan		10	No.	12,629	126,287	4,857	License fee
3e	Accessories & Divers						413,495	15,661	
3f	License fees	T&T Board, Dhaka					119,800	4,240	
3g							40,000	1,622	
			Nov-94			Total	994,253	37,713	
4	Outboard Engines								
4a	Yamaha 40 Hp Outboard Engine	B.F. International Ltd Dhaka	Jul-95	2	No.		174,000	6,088	
TOTAL EQUIPMENT PROCUREMENT IN BANGLADESH							6,427,614	252,933	

Table 5-P3: Procurement of Equipment in Bangladesh



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Procurement

Test Site 1, Kamarjani, 1994/1995

Other Procurement in Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	TOTAL Taka	TOTAL DM
1	2	3	4	5	6	7	8	9
1 1a	Import, Port & Demmorage Charges	Forwarding Agents Chittagong, Dhaka	Sep-94 Aug-95				5,078,922	190,710
2 2a	Insurance CAR, Hull, CPM, Marine insurance	Sadharan Bima Co Dhaka	Aug-94 Sep-94				4,777,054	188,453
3 3a	Hire of Equipment Hire of crawler crane and barge	Bengal Electric Dhaka	Mar-94 Jun-95				10,521,792	409,747
4 4a	Construction Works Construction Contract	The Engineers & Corolla	Sep-94 Mar-96				133,509,188	4,964,034
5 5a	Geo-Textile Bags sewing of geo-textile bags	DIRD Private Ltd. Dhaka	Jul-95	28000	No.	40	1,120,000	39,154
TOTAL OTHER PROCUREMENT IN BANGLADESH							155,006,956	5,792,099

Table 5-P4: Other Procurement in Bangladesh

Procurement

Test Site 1, Kamarjani, 1994/1995

Procurement of Equipment outside Bangladesh

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Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10	11
1	Piling Equipment									
1.1a	PTC Power Unit MH 600	PTC	Jul-95	1	NO.		283,433	7,369,255	283,433	incl. spare parts
1.1b	Hydraulic Hammer MHF 10-15	France	Oct-94	1	NO.		792,572	20,606,869	792,572	incl. spare parts
1.1c	Accessories for MHF 10-15			div.		-	-	5,658,989	217,653	incl. spare parts
1.1d	Vibrator 50H4			1	NO.		195,977	5,095,392	195,977	incl. spare parts
1.1e	Accessories for 50H4			div.		-	-	2,587,059	99,502	incl. spare parts
1.1f	Casing Guide Frame			1	NO.		49,653	1,290,987	49,653	
1.1h	CIF Chittagong					-	-	608,332	23,397	
1.1g	Instruction & Training					-	-	228,322	8,782	
								43,445,206	1,670,969	
1.2a	Drop Weight, Spare Parts for B.S.P Hydraulic Hammer	BsP International Found. England		div.		-	-	144,277	5,549	
2	Winches and Wire Ropes									
2a	Electric Wire Winch		Aug-94	2	NO.	1,109,349	42,667	2,218,697	85,335	incl. spare parts
2b	Steel Core Wire Rope 6x36, dia. 20 mm	Steen Germany	May-95	1400	m	237	9	331,622	12,755	
2c	Cif Chittagong					-	-	161,200	6,200	
								2,711,519	104,289	
3	Rubber-Tyred Rollers									
3a	Rollers with electric drive	PTOTEM	Apr-94	4	No.	513,381	19,745	2,053,523	78,982	
3b	Rollers without electric drive	France	Oct-94	8	No.	129,105	4,966	1,032,837	39,725	
3c	CIF Chittagong							227,072	8,734	
								3,313,433	127,440	
4	Shackles, Slings, Wire Ropes									
4a	Shackles & Slings for pile hoisting	Vlietdam	Jul-94	div.		-	-	714,798	27,492	
4b	Non Rotating Wire Ropes for crane	Netherlands	Feb-95	480	m	1,087	42	521,775	20,068	
4c	Cif Chittagong, Airfreight Dhaka							273,581	10,522	
								1,510,155	58,083	
to be carried forward								50,980,313	1,960,781	

Table 5-P5-1: Procurement of Equipment outside Bangladesh

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Procurement

Test Site 1, Kamarjani, 1994/1995

Procurement of Equipment outside Bangladesh

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Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10	11
5	Pendants, Wire Ropes & Spare Parts									
5a	Pendants, Wire Ropes & Spare Parts For upgrade and repair of the crawler Crane, 150 to Manitowoc.	Long International Bahrain	Sep-94 Dec-94	Div.		-	-	1,829,423	70,362	
6	Survey Equipment for Monitoring									
6.1a	Survey Computer, Echo Sounder, DGPS Tide Gauge, Data Transfer System Plotter, Printer, Monitor, Software Accessories, Training	OSAE Germany	Sep-94 Aug-95	div. div.		- -	- -	5,604,454 3,367,041	215,556 129,502	
6.2a	Electronic Current Meter Training, Repair	ADM Germany	Oct-94 Nov-95	div.		-	-	694,380	26,707	
6.3a	Leveling Instrument Wild NA 2 and Accessories	Gebr. Wichmann Germany	May-94	1	No	-	-	95,251	3,664	
6.4a	Transport, Insurance of 6.3a	Hemsoth, Gerling Germany	May-94	div.		-	-	18,158	698	
6.5a	Hydrograph, Rain Gauge	Lambrecht Germany	Jun-94	div.		-	-	43,735	1,682	
6.6a	Concrete Testing Equipment	Lackner & Partner Germany	Sep-94	div.		-	-	107,900	4,150	
6.7a	Spare Parts for Monitoring Boat	Wicking PEGEDO Germany	Jun-95	div.		-	-	6,822	262	
6.7b	Spare Parts for Monitoring Boat	Wicking PEGEDO Germany	Jun-95	div.		-	-	8,545	329	
6.8a	Photo Equipment (Cameras)	Knittel High Tech Germany	Jun-95	2	No	-	-	30,645	1,179	
6.8b	Photo Equipment (Video Camera)	Knittel High Tech Germany	Aug-95	1	No	-	-	55,956	2,152	
TOTAL EQUIPMENT PROCUREMENT OUTSIDE BANGLADESH								10,032,889	385,880	
								62,842,625	2,417,024	

Table 5-P5-2: Procurement of Equipment outside Bangladesh

Procurement

Test Site 1, Kamarjani, 1994/1995

Other Procurement outside Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM	Remarks
1	2	3	4	5	6	7	8	9	10	11
1	Management Support									
1a	Site Supervision and Management Support	Atlantic Dragage France	Jan-95 Oct-95					6,836,247	262,933	
2	Current Measurements									
2a	Current Measurements with GPS equipped drifter bouys, Reports	Labor fuer Wasserbau Fachhochschule Bremen Germany	Mar-96 May-97					3,904,832	150,186	
TOTAL OTHER PROCUREMENT OUTSIDE BANGLADESH								10,741,079	413,118	

Table 5-P6: Other Procurement outside Bangladesh

6 TENDER PROCEDURES – CIVIL ENGINEERING WORKS

6.1 PRE-QUALIFICATION PROCEDURES

In accordance with the terms of reference related to the Consulting Agreement the Consultants are required to carry out the construction works for the test structures as a General Contractor with the help of local subcontractors.

Already during the Study Phase of the Project an enlistment of local civil works contractors was carried out in September 1992 in order to obtain a realistic picture about the capacity and capability of local contractors and available construction equipment.

This enlistment (replacing a pre-qualification procedure was limited to so-called “A”-class contractors enlisted with BWDB and some specialised firms identified during the enlistment of contractors during the Study Phase. Out of more than 50 requests for submission of the PQ-documents a total of 36 pre-qualification applications were received by the Consultants. Some firms applied already at that stage as joint venture, even with participation of foreign firms.

After evaluation of the presented documents and supplementary information mainly obtained from banks the Consultants in agreement with their Client and the Donors qualified and short-listed 12 firms/joint ventures for participation in the tender for the construction of permeable groynes (reference Table 6.1-1).

6.2 TENDERING PROCEDURES

6.2.1 Tender Documents

The tender documents were drafted as per international standards and comprised of the following:
Volume I:

- Form of Tender, Appendix to Tender, Form of Tender Security
- Instructions to Tenderers
- Sample Form of Agreement
- Forms of Performance Security, Bank Guarantee for Advance Mobilisation Payment
- Bill of Quantities
- Schedules

Volume II:

- General Conditions of Contract
- Conditions of Particular Application

Volume III:

- Technical Specifications

Volume IV:

- Tender Drawings

Particular emphasis was given to the Technical Specifications, outlining and describing the individual construction works and methods in much detail. Thus all bidders were given the opportunity to clearly understand the scope of works, to elaborate their technical proposal and to select the respective construction equipment.

The General Conditions of Contract as well as the Instructions to Bidders were drafted on the principles of FIDIC, suitably adopted to take consideration of the General Contractor – Sub-contractor relationship.

Sl. No.	Name of Firms
1	Agrani Engineers and National Construction Corporation 149/A Baitush Sharaf Jame Mosque Airport Road Dhaka
2.	The Civil Engineers 53/1, New Elephant Road 2 nd Floor, Dhanmondi R/A Dhaka
3.	Soil Tech International Sena Kallayan Bhaban (15 th floor) 195 Motijheel C/A Dhaka Tel.: 862141-5
4.	M/S M. R. Sikder 76 DIT Road Malibagh Dhaka 1217
5.	The Engineers Ltd. and Corolla Corporation (BD) Ltd. 19/1 Kakrail Dhaka 1000
6.	The Engineers and Architects 10 Toynbi Circular Road Motijheel C/A Dhaka
7.	Mir Akhter Hossain House # 12, Road 08 Dhanmondi R/A Dhaka Phone 503627
8.	AML-Monico Consortium 9/A North Dhanmondi Kalabagan, Dhaka Tel. 311640, 324840
9.	Neptune Commercial 73 Siddeswari Road Dhaka 1217
10.	Wahidunnabi House # 89E, Road # 13C Banani Model Town Dhaka
11.	DIRD-PCM Joint Venture Plot # 1/B, Road 126 Gulshan Dhaka
12.	Bengal Electric Ltd. 11 Green Road Green Corner Dhaka

Table 6.1-1: List of Pre-Qualified Contractors

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6.2.2 Tender Period

The Invitation for Tender was issued to the pre-selected contractors on February 15, 1994. All 12 invited companies collected the tender documents on Friday 23, 1994.

A pre-bid meeting was held at the office of FPCO on Sunday, March 20, 1994. The bidders presented questions in writing and formulated additional questions during the meeting. All questions and answers were compiled and circulated to all bidders.

After a two-months bidding period the tender opening took place at the Consultants' Office on April 17, 1994, by which date the following five firms/joint ventures submitted their bids:

- Neptune Commercial Ltd.
- Consortium Soil Tech International & Wahidunnabi
- Consortium: The Engineers Ltd. & Corolla Corporation (BD) Ltd.
- The Bengal Electric Ltd.
- Consortium Agrani Engineers & Construction and National Construction Corporation

Since two of the invited bidders offered in joint venture, a total of 6 out of the 12 invited companies responded to the tender invitation. The tender opening result is presented in Table 6.2-1.

6.2.3 Technical Assessment of Bids

The assessment of the bids was carried out by Consultants' team in Dhaka. After a first review meetings were held with some bidders on June 05, 1994 in order to obtain clarification of their proposals. Finally the technical assessment of the submitted bids led to the following result:

- Technically most responsive was the offer from Soil Tech and Wahidunnabi;
- The second best technical offer was submitted by Neptune Commercial Ltd.;
- Technically third ranked followed the offer of The Consortium: The Engineers Ltd. & Corolla Corporation Ltd.;
- The bid of the Consortium Agrani Engineers & Construction and National Construction Corporation was technically low responsive, even after obtaining additional information and clarification;
- The proposal from Bengal Electric Ltd. was technically the lowest responsive one. Though piling works were presented professionally, but the intended execution of earth works and revetments works was unacceptable. Their technical bid showed clearly the strength in hiring out floating equipment but weakness in management and work experience in any other civil engineering works and river works.

Details of the technical assessment of the bids are compiled in [5].

FAP 21 TEST STRUCTURES

Construction of Permeable Groynes at Test Site 1 - Kamarjani

TENDER OPENING ON APRIL 17, 1994

TENDER INFORMATION

Sl. No.	Tendered By	Tender Security	Tender Amount			Total Tender Price
			Total of Bills	Total for Dayworks	Contingencies	
1/5	Neptune Construction Ltd	22,500,000 MD/159/1099/94 dt 16.4.94 ARS Bank Ltd	158,284,290	137,226,50	15,828,429	187,835,369 discount 3.1%
2/5	Construction Sait Tech International 2 Walichun Nabi	22,500,000 IFC/15/LG/151 94 dt 16.4.94 IFC Bank Ltd	201,447,150	12,976,025	20,144,715	234,567,890
3/5	Construction: The Engineers Ltd & Cosella Corporation	22,500,000 MB/156/1078/ 94 dt 17.4.94 ARS Bank Ltd	111,872,992	10,509,637	11,187,249	133,569,378
4/5	The Bengal Electric Ltd	22,500,000 ALC payc Cheque No. 483732 dt 17.4.94 State Bank of India	159,805,886	9,263,187	15,480,578	179,549,551
5/5	Construction: Asaan Engineers & Wasthoun and National Construction Co.	21,250,000 No. 30/94 dt 16.4.94 UTI Bank Ltd	108,910,610	7,286,440	10,891,061	127,088,311
		21,250,000 No. 35/1307/ 97/94 dt 17.4.94 Bangladesh State				

Table 6.2-1: Result of Tender Opening

6.2.4 Financial Assessment of Bids

Based on the bill of quantities the Consultants carried out a confidential detailed cost estimate prior to the tendering of works, arriving at an Estimated Total Net Tender Price of 173,750,988 Taka. The Total Net Tender Price is defined as total of all bills, plus daywork (Provisional Sum) and 10% contingencies for unforeseen works (Provisional Sum), but excluding VAT.

After bid opening all bids were checked arithmetically and corrected as far as permissible. After giving consideration to any offered discount the following result appeared:

- The offer from Soil Tech & Wahidunnabi was the highest with a Total Net Tender Price of
TK 228,996,390.00
which is 31.8% above the Consultants' estimate.
- The offer from Neptune Commercial Ltd. with a Total Net Tender Price of
TK 183,039,653.30
ranging 5.35% above the Consultants' estimate.
- The offer from Bengal Electric Ltd. with a Total Net Tender Price of
TK 179,167,665.50
ranging 3.12% higher than the Consultants estimate. Thereby the pile preparation and piling works were offered highest, but earth works, revetment works and other items were obviously misunderstood and wrongly calculated.
- The offer of The Engineers Ltd. & Corolla Corporation with a Total Net Tender Price of
TK 134,557,286.00
was 22.56% lower than the Consultants estimate, and
- The bid submitted by Agrani Engineers & Construction and National Construction Corporation amounted to a Total Net Tender Price of
TK 127,001,532.00
being the lowest bid and 26.91% lower than the Consultants estimate.

Details of the financial assessment of the bids including a comparison of unit rates and prices are presented in [6].

6.2.5 Contract Award

Based on the technical and financial assessment the Consultants did not consider the bid of The Bengal Electric Ltd. because of its weakness in the earth and revetment works and excessive pricing of the piling works. However, it may be mentioned here that the project took advantage of the experience of The Bengal Electric with piling works since the key equipment (crawler crane and barge) and the key operating staff was hired anyway from this company.

The bid of Soil Tech International & Wahidunnabi was not given further consideration due to its excessive pricing.

Because of technical weakness as well as lack of management resources the Consultants did not gain sufficient confidence that the joint venture Agrani Engineers & Construction and National

Construction Corporation would have the capacity to carry out and to complete the works within the given tight construction window, wherefore their bid was likewise not given further consideration.

In the final evaluation the Consultants took the offers of Neptune Commercial Ltd. and of The Engineers Ltd. & Corolla Corporation (BD) Ltd. into consideration. According to the technical evaluation both offers were close with a slight advantage for Neptune Commercial Ltd. But the financial evaluation showed a more important advantage for The Consortium: The Engineers Ltd. & Corolla Corporation (BD) Ltd. Therefore, the Consultants decided finally to sub-contract the works to this joint venture for a tentative net contract price (total of bills) of

TK 112,770,590.00

In the absence of an approved GoB budget for the payment of taxes and duties, the contract could not be awarded as scheduled (latest on July 15, 1994), but a Letter of Intent was issued to The Consortium The Engineers Ltd. & Corolla Corporation (BD) Ltd. on June 29, 1994.

After obtaining all clearances the Consortium: The Engineers Ltd. & Corolla Corporation (BD) Ltd. were formally informed on August 23, 1994 that their offer has been accepted.

On September 04, 1994 the Sub-contractor received instruction from the Consultants to proceed with the Works.

The Contract was signed on September 07, 1994.

7 EXECUTION OF WORKS

7.1 DESCRIPTION OF WORKS

The project comprises six permeable groynes, of which three groynes (G-1 to G-3 and G-A) are partly constructed off-shore and on-shore, while two groynes (G-B1 and G-B2) are constructed on the flood plain. All six structures launch from and are build against an embankment partly existent or constructed under the authority of the Bangladesh Water Development Board (BWDB).

Fig. 7.1-1 presents a general overview of the groyne field.

Groyne G-1:

- (1) Main Works On-shore
 - Construction of steel sheet pile cofferdam, length ~ 60m, width ~ 7.9 m;
 - Concrete block pavement on granular filter on top of the cofferdam;
 - Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 26 m;
 - Excavation for laying of bed protections and revetments;
 - Construction of revetments and bed protections, using stone material (rip-rap), laid on granular filter material or geo-textile filter mat.
- (2) Main Works Off-shore:
 - Installation of tubular steel piles, dia. 1016 x 20 mm, lengths up to 36 m and dia. 1220 x 20 mm, lengths up to 40 m;
 - Installation of steel girders on top of the installed piles. Steel constructions to be manufactured by the sub-contractor, utilising rolled steel beam sections, and
 - Laying of rip-rap as river bed protection, using stone material (rip-rap), laid on granular filter or geo-textile filter mat.

Groyne G-2:

- (1) Main Works On-shore:
 - Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 26 m;
 - Construction of the impermeable part of the groyne by compacted soil fill;
 - Excavation for laying of bed protections, falling aprons and revetments;
 - Construction of revetments and bed protections, using stone material (rip-rap), laid on granular filter material;
 - Construction of revetments by brick mattresses, laid on geo-textile filter, and
 - Laying of concrete blocks for falling apron.
- (2) Main Works Off-shore:
 - Installation of tubular steel piles, dia. 1016 x 20 mm, lengths up to 36 m and dia. 1220 x 20 mm, lengths up to 40 m;
 - Laying of concrete blocks for falling apron and bed protection, and
 - Installation of steel gangway and monitoring platform on top of the installed piles, steel constructions to be manufactured by the sub-contractor, utilising rolled steel beam sections.

Groyne G-3:

- (1) Main Works On-shore:
 - Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 26 m;
 - Construction of the impermeable part of the groyne by compacted soil fill;
 - Excavation for placing of bed protections, falling aprons and revetments;
 - Construction of revetments, using stone material (rip-rap), laid on geo textile filter mat;



- Construction of revetments by brick mattresses, laid on geo-textile filter;
- Slope revetment by turfing with grass sods, and
- Laying of falling aprons by concrete blocks.

(2) Main Works Off-shore:

- Installation of tubular steel piles, dia. 1016 x 20 mm, lengths up to 36 m and dia. 1220 x 20 mm, lengths up to 40 m;
- Laying of concrete blocks for falling apron as slope protection for the head of the impermeable groyne, and
- Installation of steel girders on top of the installed piles.

Groyne G-A:

(1) All works carried out on-shore:

- Construction of reinforced concrete sheet pile cofferdam, length ~60m, width ~8.0 m;
- Installation of pre-stressed spun concrete piles, dia. 500 x 100 mm, lengths of 20 m, comprising of two pile sections of 10 m each, to be joined during pile installation;
- Construction of reinforced in-situ concrete piles, dia. 914 mm (36"), lengths 24.5 m, including permanent steel tube lining, supplied by the sub-contractor;
- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 26 m;
- Construction of revetments, using stone material (rip-rap), laid on granular filter;
- Construction of revetments by brick mattresses, laid on granular filter;
- Reinforced concrete works for capping beams of the cofferdam;
- Concrete block pavement on granular filter on top of the cofferdam, and
- Turfing with grass sods on top of the cofferdam.

Groyne G-B/1

(1) All works carried out on-shore:

- Construction of the impermeable part of the groyne, length ~60m, by compacted soil fill;
- Excavation for laying of bed protections, falling aprons and revetments;
- Construction of revetments and bed protections, using cc-blocks, laid on geo-textile filter;
- Construction of revetments by brick mattresses, laid on geo-textile filter;
- Installation of pre-stressed spun concrete piles, dia. 500 x 100 mm, lengths of 10 m as well of 20 m, the later comprising of two pile sections of 10 m each, to be joined during pile installation;
- Construction of reinforced in-situ concrete piles, dia. 914 mm (36"), lengths 22 m, including permanent steel tube lining, supplied by the sub-contractor, and
- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 26 m.

Groyne G-B/2

(1) All works carried out on-shore:

- Construction of the impermeable part of the groyne, length ~46m, by compacted soil fill;
- Installation of pre-stressed spun concrete piles, dia. 500 x 100 mm, lengths of 10 m as well of 20 m, the later comprising of two pile sections of 10 m each, to be joined during pile installation, and
- Construction of reinforced in-situ concrete piles, dia. 914 mm (36"), lengths 22 m, including permanent steel tube lining, supplied by the sub-contractor.

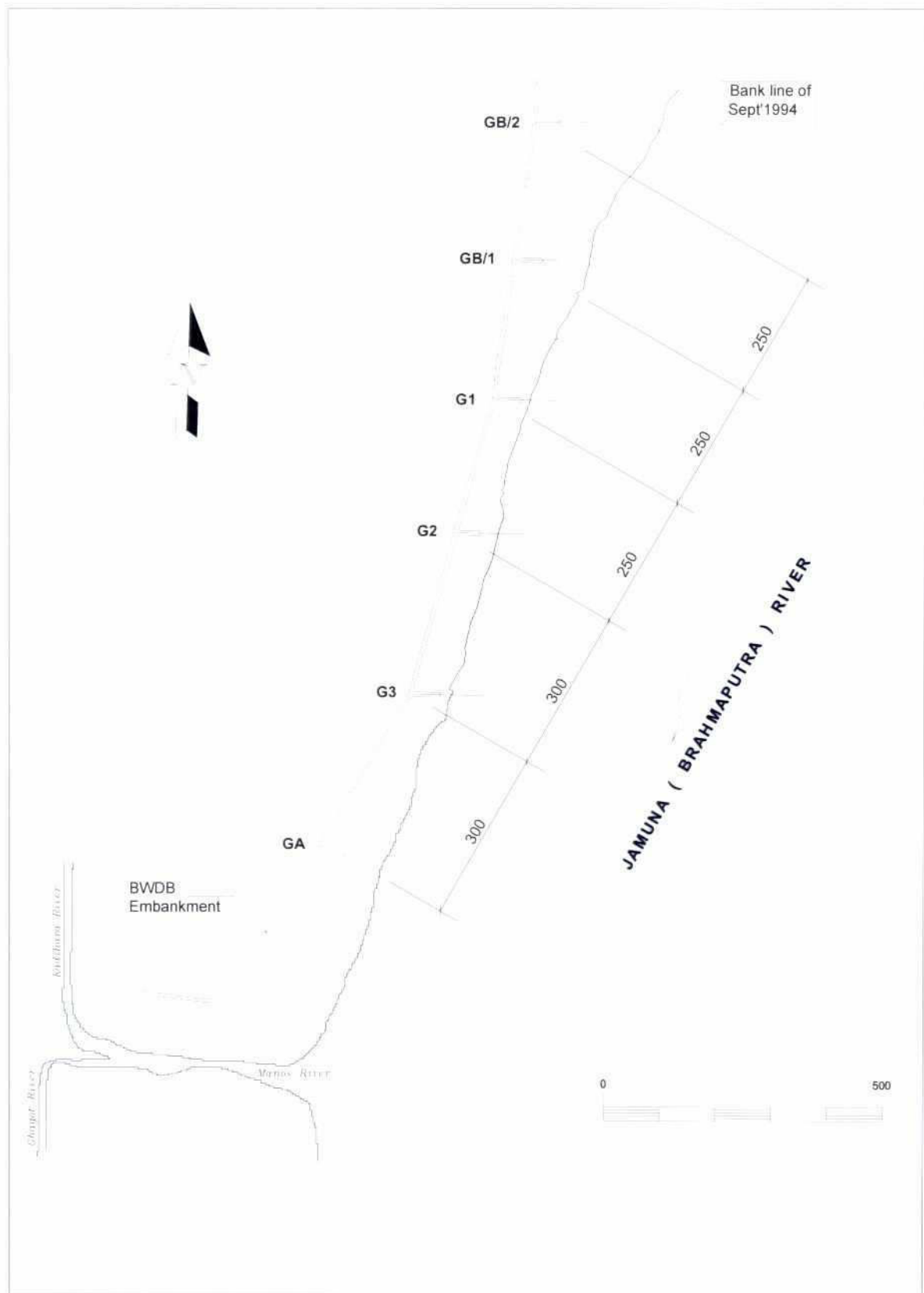


Fig. 7.1-1: General Layout of Groyne Field – Test Site I

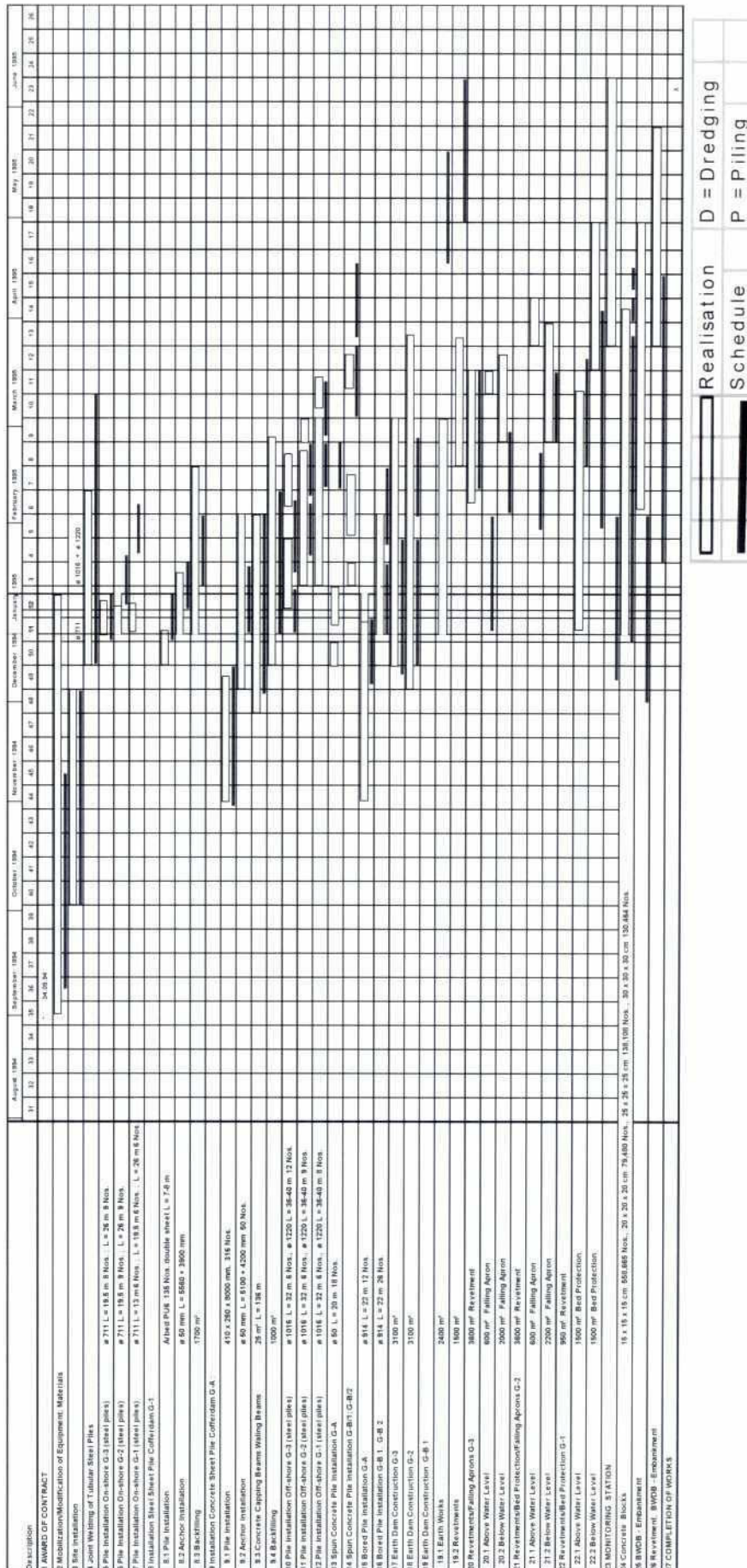
7.2 IMPLEMENTATION PLAN

The main sub-contract for the construction of the groynes and associated works was awarded to a joint venture of two Bangladeshi companies, refer to Chapter 6.

After approval of the final design and test site location by the Client and the donors the physical works at Test Site I, named after a small village of the area "Kamarjani Test Site", started on first of October 1994 with the site installation.

The piling works commenced on 01 November 1994, with installation of concrete sheet piles for the cofferdam for Groyne G-A.

All structures except the BWDB-embankment were substantially completed end of March 1995. The general Construction Time Schedule is presented in Fig. 7.2-1.



7.3 HIRE OF MAIN EQUIPMENT

In May 1994 the Consultants under their procurement program concluded a contract for hiring of a 400 ton flat top barge and a 150-ton MANITOWOC crawler crane. The equipment hire contract included provision of mooring winches and a 100 KVA generator installed on the barge, the reinforcement of the barge to carry the heavy crawler crane and the installation of water ballast tanks for trimming the barge during the pile installation operations.

The reinforcement for the stability of the barge was by additional bracing inside the barge body. The carryway for the heavy crawler crane was additionally reinforced inside the body and on the deck.

For the urgently needed repair and upgrading of the MANITOWOC crawler crane and the barge the Consultants prefinanced the procurement and supplies of ropes and spare parts which were subsequently accounted against the monthly rent.

The piling equipment, a pile guide frame, additional mooring winches and other equipment and material procured by the Consultant were loaded to the barge in Dhaka prior to the departure to the site and were finally installed, mounted and commissioned at site.



Photo 7.3-1: Hired Crane and Barge

7.4 SITE INSTALLATION

The site installation plans are presented in Fig. 7.4-1 and Fig. 7.4-2 as well as Fig. 7.10 showing layouts of steel pile assembly and welding yard, cc-block production yard, as well as of general site arrangements. These illustrate best the chosen site organisation and may serve also as a basis for future work planning.

The sub-contractor's general site installations comprised offices, material testing laboratory, workshops, storerooms, living quarters, which were arranged some 100 meters land-sided behind the construction site area.

In general all areas for the preparation or assembly of structural items were situated within the acquired land to minimise transport and costs.

The welding yard for the assembly of steel piles was located on the flood plain between Groyne G-3 and Groyne G-A. The assembly and welding yard covered an area of about 25,000 m². In addition, a stockpile area for completed steel piles was prepared near the riverbank.

The concrete yard for the prefabrication of cc-blocks was located on the flood plain downstream of Groyne G-A, covering an overall production area of 40,000 m². This included area for brick storage and chipping, washing of aggregates, concrete production as well as storage of produced cc-blocks.

A first-aid station was provided close to the main working area.

The sub-contractor maintained a liaison office in Gaibandha.





Photo 7.4-1: CC-Block production and storage yard



Photo 7.4-2: Steel pile welding and storage yard

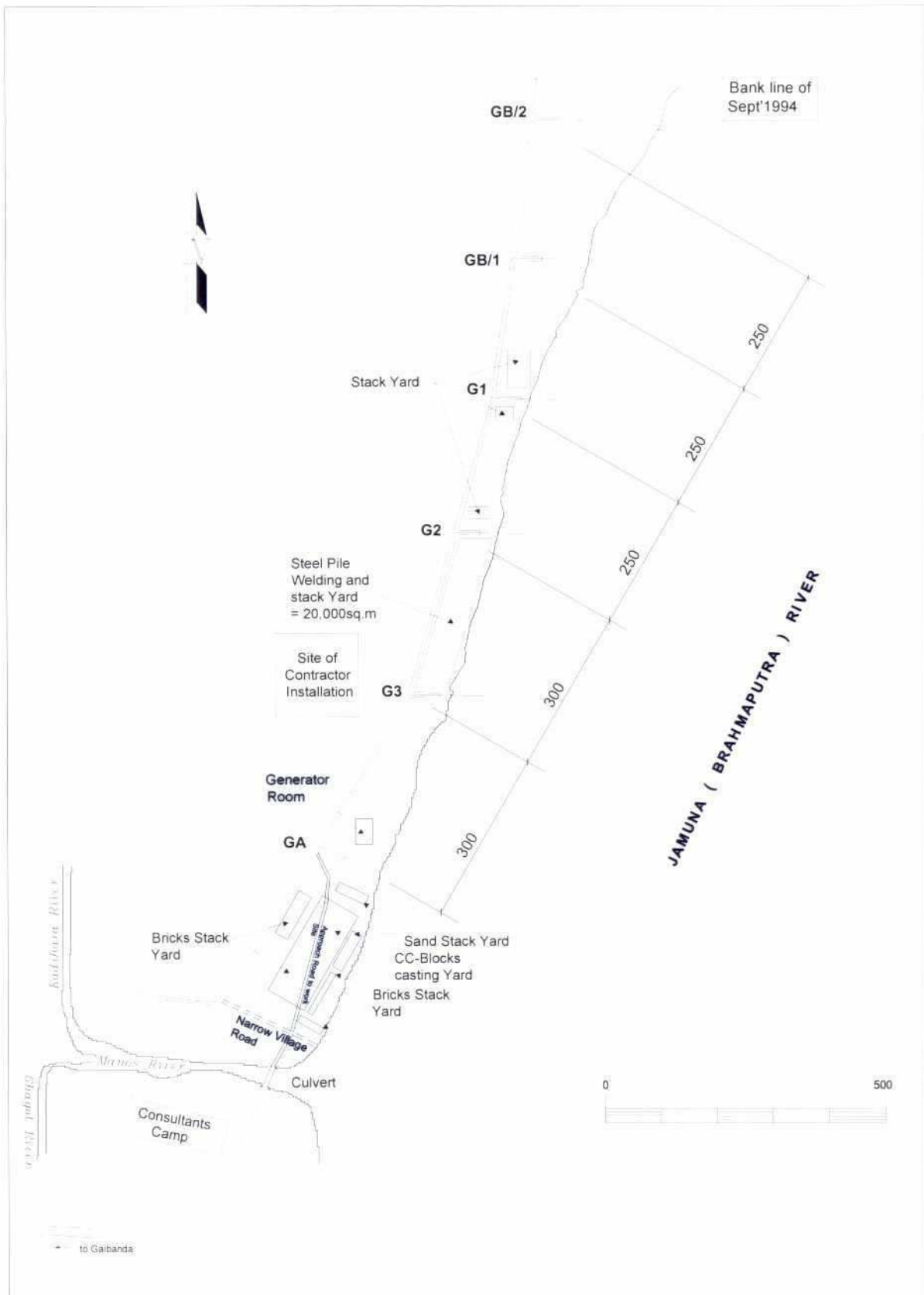


Fig. 7.4-1: General Layout of Site Installation – Test Site I (Kamarjani)

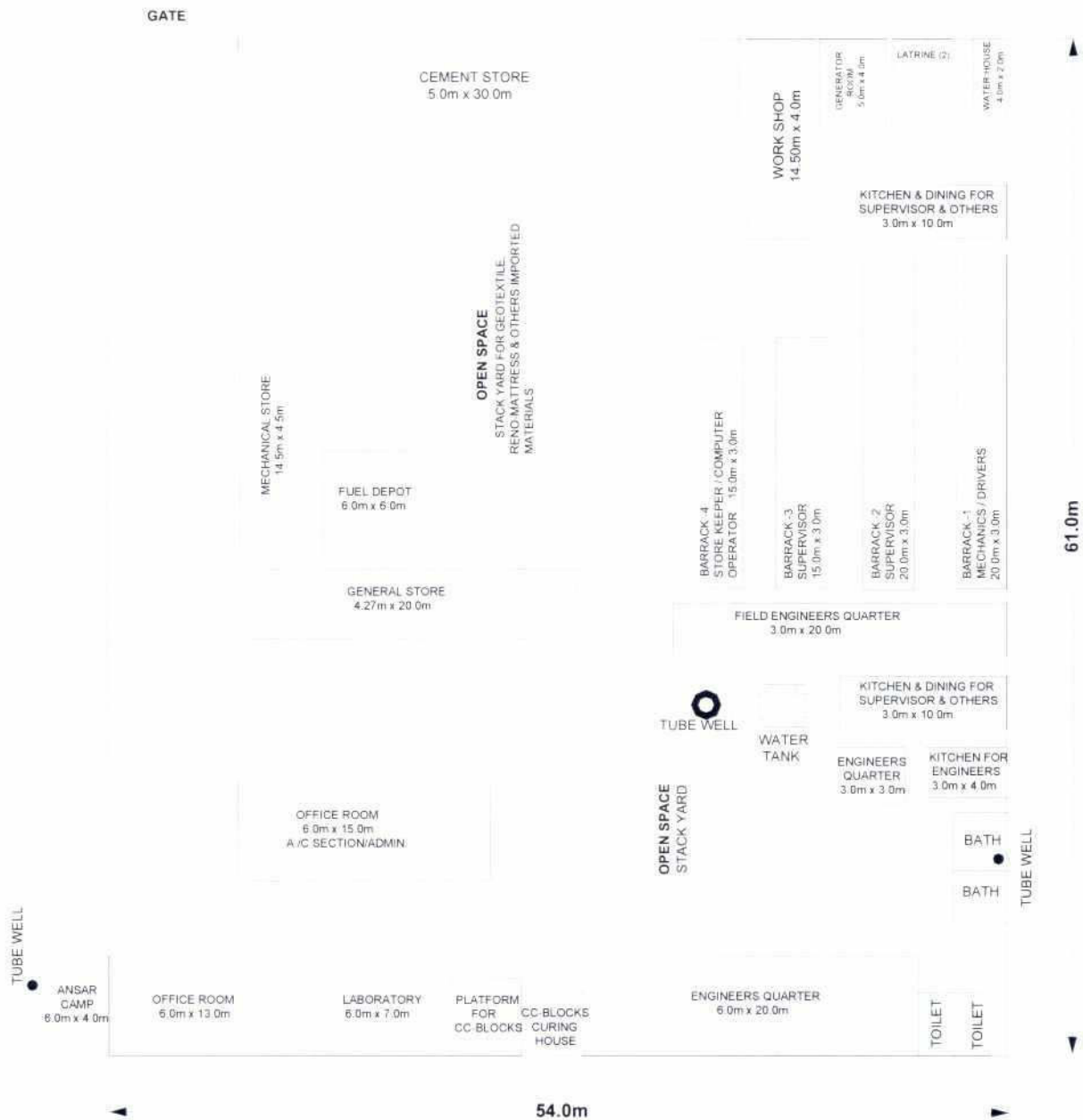


Fig. 7.4-2: General Layout of Contractors Camp – Test Site I (Kamarjani)

7.5 EARTH WORKS

7.5.1 General

According to the agreement between the Governments of Bangladesh and Germany/France the groyne test structure was to be linked to an existing main flood embankment. At the selected location for the first test site only fragments of BWDB main embankment were left due to ongoing bank erosion during the last years.

It was agreed between the parties that GoB through BWDB would construct a new embankment, which would be part of the general Gaibandha town protection program.

BWDB constructed the main embankment within the test site area between February and June 1995, employing 4 local contractors from the Gaibandha area.

The earth works under the contract for the test structures were limited to site clearance, filling of existing ponds and old borrow pits, preparation of work platforms and construction of the impermeable parts of the groynes.

All excavation and filling works in the dry were executed by hand, compaction was done by bulldozer over run and for the pond filling by small vibrator.

Some underwater excavation had to be carried out to facilitate installation of tubular steel piles by floating piling equipment also in the transition between river bank and off-shore part of the permeable groynes. For these dredging activities the crawler crane with bucket of the floating piling equipment was utilised to clear sufficient water depth along the alignment of the piles to be installed for the permeable groynes.

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Photo 7.5-1: Earth filling for Main Embankment



Photo 7.5-2: Manual excavation



Photo 7.5-3: Excavation at Borrow Pit



Photo 7.5-4: Clamshell dredging

7.5.2 Site Clearance

The site was cleared of all obstacles and remaining crops by bulldozer. At working platforms and groyne locations at least a layer of 20 cm of topsoil was removed.

7.5.3 Pond Filling

Ponds and old borrow pits within the alignment of the future main embankment and the groynes structures had to be filled (Fig. 7.5-1). Before filling the ponds were pumped dry and the mud was removed manually in buckets. Earth for filling was taken from new borrow pits and from the flood plain.

All filling work was carried out in layers of about 30 cm with compaction.

7.5.4 Backfilling of Sheet-Pile Cofferdam

The impermeable part of Groyne G-1 and Groyne G-A was designed and constructed as sheet-pile cofferdam. After piling of the sheet piles and installation of the horizontal anchors the cofferdams were back-filled with suitable soil and compacted in layers to a density of $D = 0.60$ to 0.75 . The achieved degree of compaction was confirmed through regular in-situ as well as laboratory tests.

At Groyne G-A a mineral granular filter of minimum 50 cm thickness was filled in between the reinforced concrete sheet-pile wall and the earth fill to prevent seepage of backfill material through the joints between the individual sheet-piles.

At the top the cofferdam of Groyne G-1 received cc-block pavement on a 30 cm granular filter layer, but Groyne G-A received a cover by turfing with grass sods.



Photo 7.5-5: Backfill of the Cofferdam of Groyne G-A



Photo 7.5-6: Backfill of the Cofferdam of Groyne G-1

7.5.5 Construction of Earth Dam for Impermeable Groyne Heads

The impermeable part of Groynes G-2, G-3, G-B/1 and G-B/2 is designed as earth-filled dam. The earth dams of these 4 groynes were constructed by suitable soil filling in compacted layers of about 30 cm thickness and according to the respective working drawings.

The earth filling was mainly done by hand (hand excavating, hand transport and hand dumping), partly by pushing the soil to the fill location by dozer (direct dozer operation from excavation to fill and compaction)



Photo 7.5-7: Construction of Earth Dam, Groyne G-3

7.5.6 Soil Compaction

The soil was compacted by 8 to 10 passes of a dozer (Caterpillar D 6) in layers of max. 30 cm. The degree of compaction was verified for every layer through in-situ tests and laboratory tests. The achieved degree of density was > 0.6 .

Typical laboratory reports on soil testing are shown in Fig. 7.5-2 to 7.5-4.

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Fig. 7.5-1: Location of Borrow Pits

DETERMINATION OF THE MOISTURE CONTENT FIELD DENSITY
AND DEGREE OF DENSITY THE SITE

Chainage: Km

to Km

Test Layer: Core/Cover Time:

Date: 18-11-94

R.L.:

R/S

G/S

G-1

Layer No. 9th

Groyne: G-1

S.P. (x) = 2.661

From C/S of
From C/L of

		28th	27-8th	25-5th
		0	8th N	4th S
A	MOISTURE CONTENT	Wt. in gram		
	Can No.	3	16	10
m	Mass of can + Wet Soil	94.9	97.4	96.3
m ₀	Mass of can + dry soil	83.9	85.6	84.2
m ₁	Mass of water $m - m_0$	11.0	11.8	12.1
m ₂	Mass of can	27.2	25.8	24.8
m ₃	Mass of dry soil $m_0 - m_2$	56.7	59.8	59.4
m ₄	% Moisture $m_1 \times 100 / m_3$	19.4	19.7	20.4
B	FIELD DENSITY (Sand Replacement Method)			
w	Mass of wet soil + container	6975	6875	7000
w ₀	Mass of container	1680	1680	1680
w ₁	Mass of wet soil $w - w_0$	5295	5195	5320
w ₂	Mass of sand + cylinder before pouring	9050	8275	8525
w ₃	Mass of sand + cylinder after pouring	3410	2650	2940
w ₄	Mass of sand to fill cone	1381	1381	1381
w ₅	Mass of sand to fill hole $w_2 - w_3 - w_4$	4259	4244	4204
w ₆	Bulk density of sand (from calibration)	1.369	1.369	1.369
w ₇	Gross volume of hole w_5 / w_6	3111	3100	3071
y _w	Field wet density w_1 / w_7	1.702	1.675	1.732
y _d	Field dry density (FDD) $100 \times y_w / (100 + m_4)$	1.425	1.399	1.438
C	DEGREE OF DENSITY			
	Loos dry density	0.991	0.991	0.991
	Porosity in loose state, max. n	0.627	0.627	0.627
	Maximum dry density	1.612	1.612	1.612
	Porosity in densest state, min. n	0.394	0.394	0.394
	Porosity of compacted soil, n	0.465	0.475	0.459
	Degree of density, $D = \frac{\text{Max. n} - n}{\text{Max. n} - \text{Min. n}}$	0.69	0.65	0.72
	Mean degree of density	0.68		

Tested by
Lab. Tech.
Contractor

Submitted by
Material Testing Officer
Contractor

Fig. 7.5-2: Moisture Content and Degree of Density of Soil at Groyne G-1

Groyne: G-1
 Station: 45 m from C/L
 Offset: 0.24 m N from C/L
 Date: 19-11-94
 Houring:

Max. DENSITY

Date: 19-11-94

Density Test-2

Determination No.	1	2	3
(W ₁) Wt. of Mold + Base Plate + Soil gm	3456	3511	
(W ₂) Wt. of Mold + Base Plate gm	1932	1932	
(W ₃) Wt. of Soil, W ₁ - W ₂ gm	1524	1579	
(V) Volume of Mold cc	943	943	
Max. density of soil $\frac{W_3}{V}$ gm/cc ²	1.616	1.674	1.645
Average of Max density dry		1.440	

Tests done in presence of Employer Mr. Zeller, Mr.
 Mr. John A. ...

Fig. 7.5-3: Maximum Density of soil at Groyne G-1

Project: FAP 21, Gaiabanda
 Groyne: G-1
 Station: 45 m from C/L
 Offset: 0.24 m N from C/L

LOOSE DENSITY

Date: 19-11-94

Density Test-1 & 2

Determination No.	1	2	3
(W ₁) Wt. of Mold + Base Plate + Soil gm	8227	8250	8270
(W ₂) Wt. of Mold + Base Plate gm	5450	5450	5450
(W ₃) Wt. of Soil, W ₁ - W ₂ gm	2777	2800	2820
(V) Volume of Mold cc	2811	2811	2811
Loose density of soil $\frac{W_3}{V}$ gm/cc ²	0.987	0.996	1.003
Average of loose density		0.995	

Test was done in presence of Employer (foreigners)
 A. ...

Fig. 7.5-4: Loose Density of soil at Groyne G-1

7.6 CC-BLOCK PRODUCTION

The total cc-block production was approximately 6,500 m³.

Produced were in total 400,000 cc-blocks:

- 100,000 nos. of 30x30x30 cm
- 125,000 nos. of 25x25x25 cm
- 105,000 nos. of 20x20x20 cm
- 70,000 nos. of 15x15x15 cm

These cc-blocks were mainly used for the construction of falling aprons at the groynes and slope toe protection of the embankment.

For pavement of the surface of the two cofferdams at groyne G-1 and groyne G-A 13,000 cc-slabs 20x20x8 cm were produced.

The blocks were manufactured in a so-called concreting-bed with steel framework using up to 15 nos. of small to medium size concrete mixers (350 l to 500 l capacity). Filling of the mixers with aggregates was done by hand using calibrated wooden buckets.

The average production rate was 60 m³ per day and 10 hour shift. The maximum production rate as 130 m³ per day. Approximately 180 labour per day were employed for mixing and casting. Curing was by wet kept geo-jute mats. The ready cc-blocks were stored in stock-piles.



Photo 7.6-1: CC-Block production



Photo 7.6-2: Brick cutting and CC-Block production



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7.7 INSTALLATION OF REVETMENT PROTECTION

(a) Groyne G-2:

The earth dam received a slope protection from boulder rip-rap (stone revetment) on granular filter layer. The part integrated in the main embankment received a slope protection of brick mattress on geo-textile filter layer.



Photo 7.7-1: Rip-rap and Brick Mattress at Groyne G-2



Photo 7.7-2: Rip-rap and Brick Mattress at Groyne G-2

(b) Groyne G-3:

The earth dam received at the head a slope protection from boulder rip-rap (stone revetment) on geo-textile filter layer. The other part received a slope protection of brick mattress on geo-textile filter layer. The part integrated in the main embankment received no special protection but was covered with grass sods on geo-jute soil saver.



Photo 7.7-3: Rip-rap on Geoextile Filter Mat at Groyne G-3



Photo 7.7-4: Brick-Mattress of Groyne G-3

(c) Groyne G-B/1:

The earth dam received at the head a slope protection from cc-blocks on geo-textile filter layer. The other part and the part integrated in the main embankment received a slope protection of brick mattress on geo-textile filter layer.



Photo 7.7-5: CC-Block pavement at the Head of Groyne G-B/1



Photo 7.7-6: Brick-Mattress on Geotextile Filter Mat at Groyne G-B1

(d) Groyne G-B/2:

The slope of the earth dam received a grass sods on geo-jute soil saver.

All above described works were executed by hand involving several hundred skilled and unskilled labour.

7.8 PLACING OF GEO- and GEO-SAND FILTER MAT

Before placing of the geo-textile filter mats the delivered strips of the mats were cut or sewed together to design length and width. Delivery size is in general 4 to 6 m width and depending on the thickness of 50 to 100m length. Sewing is done by hand or hand held sewing machines.

For the under water part it was at first tried to fix the geo- or geo-sand filter mat to a specially built steel frame of 12 x 12 m and to lower it to the riverbed by crane. Due to high current this method was not very successful because the frame with geo-textile mat behaved like a sail and it was quite difficult to bring the frame into the right position. Once the frame was in position boulders were dumped from country boats for ballast. The fixation of the mat to the connections to the frame were made weak enough, so that the frame could be lifted leaving the mat on the ground.

For the part with lesser water depth the more classical method of pulling the mat into the water with a pulling beam carried by a barge mounted crane. The barge is pulled by its winches and during the pulling procedure the so unrolled mat was ballasted with boulders for sinking to the riverbed. The boulders were dumped from a country boat.

At a small part of the bed protection around Groyne G-1 area a special innovative self-sinking filter mat from geo-jute and geo-textile was tested. Between the layers the mat was filled with sand as ballast for sinking. The idea was to use geo-jute material from Bangladesh, which is not durable for long time for the down part of the "sandwich" mat and to have the durable geo-textile mat on top. For this test the mat was manufactured in Germany (with jute and Jamuna sand from Bangladesh) because it was not (yet) possible to have the mat filled and sawn in Bangladesh.

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Photo 7.8-1: Geotextile Mat fixed to a Steel Frame



Photo 7.8-2: Geotextile Mat Fixed to a Steel Beam

7.9 INSTALLATION OF BED PROTECTIONS / FALLING APRONS

(a) Groyne G-1:

After finishing the piling works the river bed and the slopes around the cofferdam and the piles were profiled to design and different filter and protection layers were built in partly in the dry and partly under water. The different protection layers were laid by hand or dumped from country boats and/or flat barges.



Photo 7.9-1: Dumping of Boulders as Bed Protection from a Flat Barge



Photo 7.9-2: Dumping of CC-Blocks as Bed Protection from a Country Boat



Photo 7.9-3: Groyne G-1, Bed Protection, dumping of Boulders on Geotextile Filter Mat



Photo 7.9-4: Groyne G-1, Bed Protection, manually laid Boulders on Geotextile Filter Mat

(b) Groyne G-2:

Only a falling apron of different sizes of cc-blocks was built as bed protection around the impermeable dam and around the piles without any filter layer. The different protection layers were laid by hand or dumped from country boats and/or flat barges.



Photo 7.9-5: Groyne G-2, Bed Protection, dumped CC-Blocks (without filter layer)



Photo 7.9-6: Groyne G-2, Bed Protection, dumped CC-Blocks (without filter layer)

(c) Groyne G-3:

A falling apron was built similar at Groyne G-2, but only as a scour protection to the head of the impermeable groyne part.



Photo 7.9-7: Groyne G-3, Scour Protection, CC-Blocks



Photo 7.9-8: Groyne G-3, Scour Protection, CC-Blocks

(d) Groyne G-A:

A scour protection was built around the sheet pile cofferdam, from rip-rap on granular filter along the two parallel sides and from cc-blocks on granular filter around the head of the cofferdam.



Photo 7.9-9: Groyne G-A, Bed Protection, CC-Blocks

(e) Groyne G-B/1:

The short impermeable part received at the sides and at the head a falling apron from cc-blocks, the part integrated in the main embankment was covered with brick mattress all on geo-textile filter mat.



Photo 7.9-10: Groyne G-B/a, Bed Protection, CC-Blocks

(f) Groyne G-B/2:

This groyne received no bed protection since an immediate attack by the river was not anticipated. If needed, this will have to be supplemented at a later stage.

The placing of the bed protections and falling aprons was done by hand. Dumping of the materials under water from country boats or flat barges was done by hand too.

7.10 STEEL PILE ASSEMBLY, WELDING AND TESTING

The tubular steel piles of 711, 1016 and 1220 mm diameter were supplied in 6.5 m, 8 m and 12 m lengths. According to the design length the piles were welded together at site with one, two or three fully welded butt joints. The pipes were laid on the roller bed. The welding faces were chamfered and matched with each other keeping a root gap of 3 mm and tack welded. The root was welded from inside and outside of the pipe. The root welding was grinded to remove all imperfections and v-cut. The reverse side of the root was welded from inside the pipe. After the root welding was complete intermediate layers were welded to fill the v-cut. Each layer was cleaned by grinding to remove slag and imperfection.

To avoid welding from inside the pipes (mainly for the 711 mm pipe) it was intended to use special ceramic welding supporting rings pasted to the inside wall. But due to operational difficulties and imperfection the welders preferred to weld from in-and outside. During inside welding the pipes were ventilated with fans.

All welding was tested by ultra-sonic non-destructive method through an independent institute.

The welders (6 nos.) passed welder qualification tests in vertical upwards and down hand welding position on 16 mm plate at the "BANGLADESH OXYGEN LIMITED (BOL)" training centre, and the test welding passed Non-Destructive (ND) tests at the "Atomic Energy Commission of Bangladesh".

Welding was vertical upwards with two welders at the same joint at 180 degrees opposite position. During welding the pipes were turned on motor driven rollers in the roller bed with a speed adjusted to the welding progress.

Welding electrodes were dried and stored in the heating oven at 110 degrees Celsius before use. For welding a small quantity was kept in portable storage box.

The complete piles with a maximum length of 44 meters were stored at the welding yard. The piles for offshore piling were stored close to the river bank for loading to the pile transportation barges. (see Fig. 7.10).

A total of 105 piles have been welded together, i.e.:

59 piles of diameter 711 mm with 142 joints of 2.24 m welding seam length each

18 piles of diameter 1016 mm with 54 joints of 3.20 m welding seam length each

26 piles of diameter 1020mm with 78 joints of 3.20 m welding seam length each.

The in total 274 joints with a total welding seam length of 1,151 m have been welded in 54 working days with an average of 5 joints per day.

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(a) Main equipment used:

- Crawler cranes for pile handling (IHI 45 t crane, HITACHI 40 t crane, LINKBELT 45 t crane);
- Tractors and trailers for transport ;
- Welding machines BORLAC T-300;
- Cutting and bevelling machine, semi-automatic, chain travelling-type;
- Power Generator;
- Rubber tired roller bed for welding and turning of piles, 2 sets (one set means 4 free wheel stands and 2 motor driven stands);
- Drying oven for welding electrodes;
- Temperature controlled boxes for welding electrodes;
- Grinding machines;
- Gas cutting equipment, and
- Non-destructive ultrasonic testing equipment for testing the welding seams.

(b) Main material used

- Electrodes for root-pass: PIPECRAFT E 6010 mild steel, cellulose deep penetration type 3.25 mm dia.;
- Electrodes for filler-pass and capping: MULTICRAFT E 7016 mild steel low hydrogen type 4 and 5 mm dia.;
- LP gas and oxygen were used for cutting;
- Supplier of all materials: BANGLADESH OXYGEN LIMITED.

Fig. 7.10-1: Layout of Welding Yard



Photo 7.10-1: Pile Welding Yard



Photo 7.10-2: Pile Welding Roller Bed



Photo 7.10-3: Vertical upwards Welding on Roller Bed



Photo 7.10-4: Horizontal Welding "In-Situ Pile Joints"



Photo 7.10-5: Pile Gas cutting

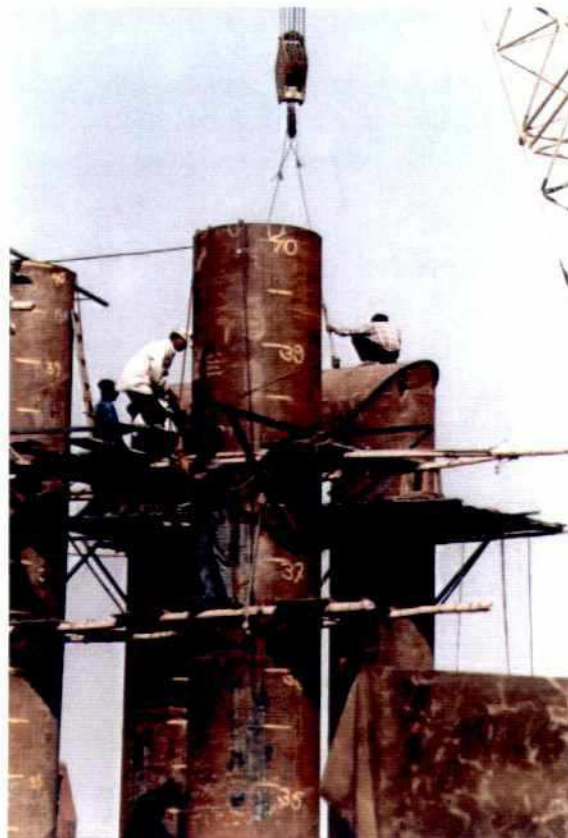


Photo 7.10-6: Pile assembly at the Head of Groyne

7.11 STEEL PILE INSTALLATION

7.11.1 General Information

The objective of this part of the report is to compile the piling method applied, the difficulties faced and solution found executing the piling works.

The works comprises on-shore and off-shore piling with different types of piles as described below:

- Installation of on-shore tubular steel piles dia. 711.2 mm x 14.2 mm wall thickness, length maximum 26 m at Groynes G-1, G-2, G-3, G-A and G-B/1;
- Installation of off-shore tubular steel piles dia. 1016 mm x 20 mm wall thickness, length 32 m each, at Groynes G-1, G-2 and G-3, and
- Installation of off-shore tubular steel piles dia. 1220 mm x 20 mm wall thickness, length up to 44 meter, at Groynes G-1, G-2 and G-3.

7.11.2 On-Shore Piling

All the 711 mm dia tubular steel piles were installed on-shore in groyne G-1, G-2 and G-3. From the welding yard the welded piles of design length were carried to the designated groyne by tractor and trolley. A 45 ton crawler crane was used to load the piles in the welding yard. The trolleys were unloaded at site. The surveyor put the peg in the centre mark of each pile. For piling 711 dia piles Kobe 22 piling rig with 25 meter leader, Kobe 22 water cooled diesel hammer was used as main equipment. Supporting equipment were generator, welding machine etc.

The piles were lifted and pitched in position by the winch of the piling rig and pulley on top of the leader. A guiding frame was placed at the bottom of the pipe and the other one was the helmet below the diesel hammer. The position was checked by the surveyor before piling started. Pile driving continued till the design depth was achieved.

711 mm diameter piles are of three different length:

13 meters	-	6 numbers,
19.5 meters	-	23 numbers,
26 meters	-	30 numbers. (25 piles of 22m plus 4m part welded on top after piling of 22m part; 5 piles of full 26m length)

For 13 meter long piles, the leader was long enough and the piles could be installed directly.

To place the 19.5 meter and 26 meter long piles directly below the helmet was not possible due to limited length of the leader which could accommodated up to 18.5 meter piles only below the helmet.

Therefore the ground level was reduced to approximately 19 meter PWD and 2 meter deep holes were excavated. This allowed to install the 19.5 meter piles.

For the 26 meter piles the hole was made deeper, protected by temporary casing of 2 meter length and 1 meter diameter. The pile was cut at the welding yard in two pieces of 22 meter and 4 meter.

First the 22 meter piles were installed to design depth, then the temporary casing was removed and the 4 meter pieces were welded to the driven piles to reach design top level of the piles at 23.5 meter PWD. The in situ welding was done by the certified welders and checked by random ultrasonic tests.

At groyne G-A and Groyne G-B/1 5 piles were installed on shore in full 26 m length with the BSP hydraulic hammer.

(a) Main equipment used for on-shore steel pile piling:

- Piling rig KOBE 22, two numbers, with 25 meter long leader, guiding the hammer by two round bars 70 mm diameter and centre to centre distance of 330 mm.
Lifting by electric winch with 6 ton capacity.
The leader could rotate on the chassis of the rig, the rig itself was running on rails parallel to the pile rows.
Movement and rotation by electric engines.
- R-B 600 SC (Lincoln) crawler crane (60 t) with 19 m boom and 27 m long fixed leader, Hydraulic Hammer Model BSP HH MK3 (357-7 series), 3 to 7 t adjustable drop weight and BSP Hydro Pack with Perkins T6-3544 diesel engine
- Generator for powering the rigs
Trademark and model:
Petbow 100 Kw.
- Kobe 22, water cooled diesel hammer, ram weight 2.2 ton, maximum energy 6.15 tonmeter at 45 blows/min.
- Kobe 13, water cooled diesel hammer, ram weight 1.3 ton, maximum energy 3.38 tonmeter at 45 blows/min.

(b) Material used:

Steel piles steel grade St 37-3, diameter 711.2 x 14 mm, length 13 m, 19.5 m and 26 m.

(c) Steel Piles dia. 711, performance data:

Total quantity	:	G-1: 20 piles (6x13m; 7x19.5m; 7x22m) G-2: 18 piles (9x19.5; 9x 22m) G-3: 17 piles (8x19.5; 9x22m) G-A: 3 piles (26m) G-B/1: 2 piles (26m)
Net piling depth	:	15.5 to 17.5 meter (19.5m and 22m piles)
Net working days	:	26 days (G-1, G-2, G-3 only)
Maximum per day	:	4 piles
Minimum per day	:	1 pile
Average per day	:	2 piles per day
Net piling time	:	65' to 200' per pile
Total installation time	:	3h to 6h per pile

Typical pile driving diagrams are presented in Fig. 7.11-1 and Fig. 7.11-2.



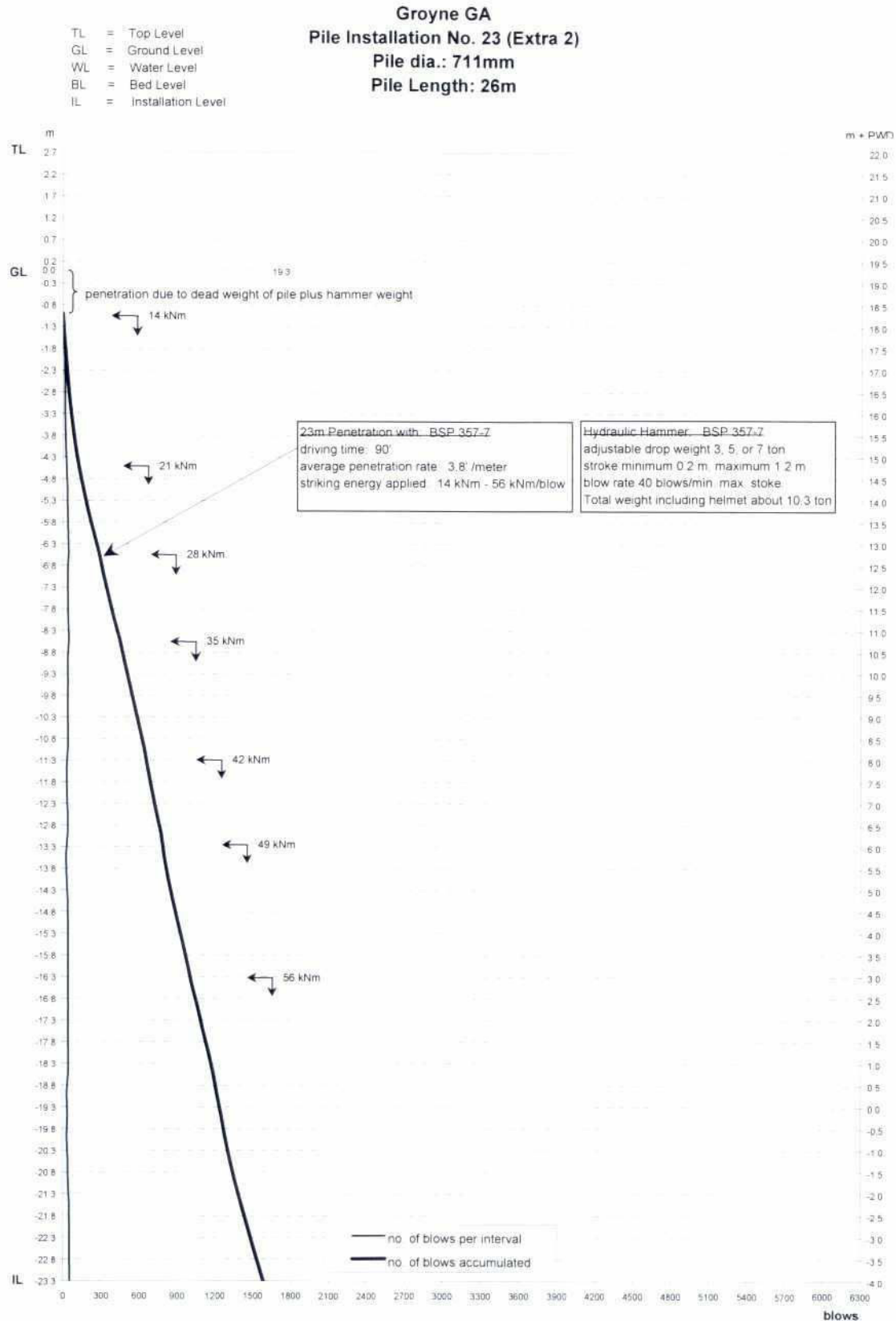


Fig. 7.11-1: Pile Installation No. 23 (Extra 2), dia. 711 mm – Test Site I (Kamarjani)

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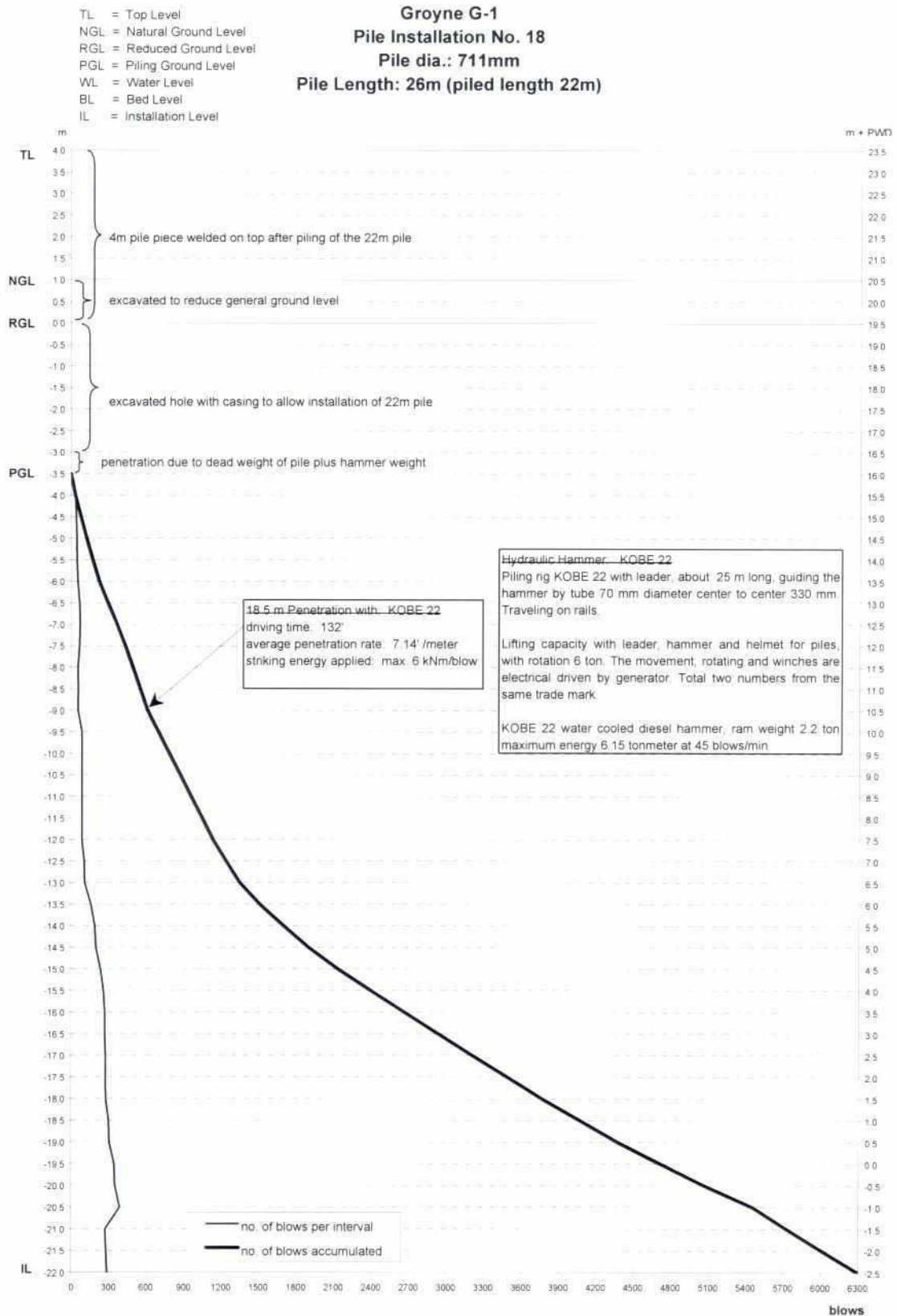


Fig. 7.11-2: Pile Installation No. 18, dia. 711 mm – Test Site I (Kamarjani)

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**Photo 7.11-1: On-shore Piling of Steel Pile dia 711 mm;
Piling rig Kobe 22**



**Photo 7.11-2: On-shore Piling of Steel Pile dia 711 mm;
Piling rig Kobe 22
Hydraulic BSP Hammer, Leader and R-B 600 SC
(Lincoln) Crawler Crane**

Handwritten signature or mark.



**Photo 7.11-3: On-shore Piling of Steel Pile dia 711 mm;
Piling rig Kobe 22**



**Photo 7.11-4: On-shore Piling of Steel Pile dia 711 mm;
Piling rig Kobe 22**

7.11.3 Off-Shore Piling

Prior to off-shore piling the river bed had to be dredged for some meters in the alignment of the groynes, 10 meter on either side from the centreline, to have a necessary water depth of 3 meter for the piling barge.

For lifting the piles were prepared with two holes 250 mm below the top, in 180 degree position, to fix special "release shackles". These shackles could be opened via a pulling cable for release, but only when not under force. Through a hole at the bottom of the pile, a rope was fixed to guide the pile, during lifting and pitching process.

Off-shore piling started at groyne G-3 with piles number 19 to 24, each pile 1016 mm dia. and 32 meter long. These 6 piles were lifted from the land side. The crane of the piling barge could reach the top of the piles while laying on the ground, a second crane was positioned to lift the bottom part. Both cranes lifted the pile keeping it in a more or less horizontal position. Then the barge crane continued lifting while the on-shore crane lowered until the pile came into vertical position under the hook of the piling crane. The free hanging pile was then clamped into the piling tower with its two clamp frames.

The barge together with the pile was brought in piling position with the winches of the anchor ropes.

The release shackles were opened and the hook was taken off. Before release of the pile, it was secured properly at the top level of the guide frame by wire rope for safety.

The crane on the barge then lifted the vibration-hammer and placed it over the pile head and the pile head was properly clamped into the vibration-hammer. The pile was then lifted from the ground and hang freely on the hook. The safety wire rope at the top frame was released and the upper frame was lifted vertical by the special hand winch. The pile was brought into the right vertical piling position by boom and barge movement. The position was checked by 1.5 meter spirit level and survey instruments. Then the vibrator was started, the pile was pitched and the piling continued. During piling process the vertical position was checked. All the six piles were driven to design depth by vibrator only.

The longer piles number 25 to 34 were transported by the barges and tug boat from the welding yard to the piling barge.

The piles were lifted as described above, but the second hook of the piling crane was used as second lifting unit. The rope of the second hook was fixed to the pile with two grommets placed 2 meters below the gravity point. Positioning of the piles took place as described before. When the time for penetration of 1m took more than 4 minutes the vibration-hammer was lifted off from the pile and the hydraulic-hammer was lifted and placed over the pile. When the hammer was in place properly, hammering was started and the pile was driven to designed depth Drawing No. KA-009, Attachment 1).

All other offshore piles were carried by the barges only and piled as described above.

(a) Total numbers installed at the four main groynes are:

- 1016 dia x 32 m long, 18 numbers
- 1220 dia x 36 m long, 15 numbers
- 1220 dia x 40 m long, 11 numbers.

(b) Main equipment used for offshore steel pile piling:

- Flat-Barge BG-7 used for offshore piling and dredging with Manitowoc crane

Main dimensions: 36.58 x 12.2 x 2.4 meter (120'x40'x8')

Average draft 1.22 m. Maximum load capacity 400 ton.

The stability of the barge was enhanced by complete reinforcement and additional bracing inside the barge body. The carry way for the heavy crawler crane was again stronger reinforced. Water ballast tanks were installed to compensate heeling of the barge by heavy loads.

The barge was equipped with:

- in total 4 winches. 2 single drum winches and 2 double drum winches, pulling force about 5 tons each, installed in the front and backside of the barge.
- two generator, one 100 KVA for general use and one 5 KVA for lightning
- one pile guiding tower with two guide frames at different levels. The position of the upper guide frame can be moved in the horizontal plan to adjust the pitch of a pile. Both guide frames have gates with easy opening and locking arrangement for installation of piles. The tower was welded to the deck of the barge at the front.
- 4 anchors weighing 3 ton each with anchor-boys for anchoring the barge over the 4 corners.

- Crane used for off-shore piling and general lifting work:

trademark and type:

Manitowoc, model 4000W VICON, 150 ton capacity

equipped with:

170 ft boom

three winches:

first hook with 4 wires

second hook with 2 wires

third hook with 1 wire

- Hydraulic vibration-hammer:

trademark and type:

PTC "Vibrofonceur model 50 H4"

Eccentric moment 500 Nm.

Total weight 6.5 ton

maximum amplitude 21 mm

Fitted with two 110 ton clamping heads for piles 1016 mm and 1220 mm.

Groyne G-1
Pile Installation No. 29A
Pile dia.: 1220mm
Pile Length: 40m

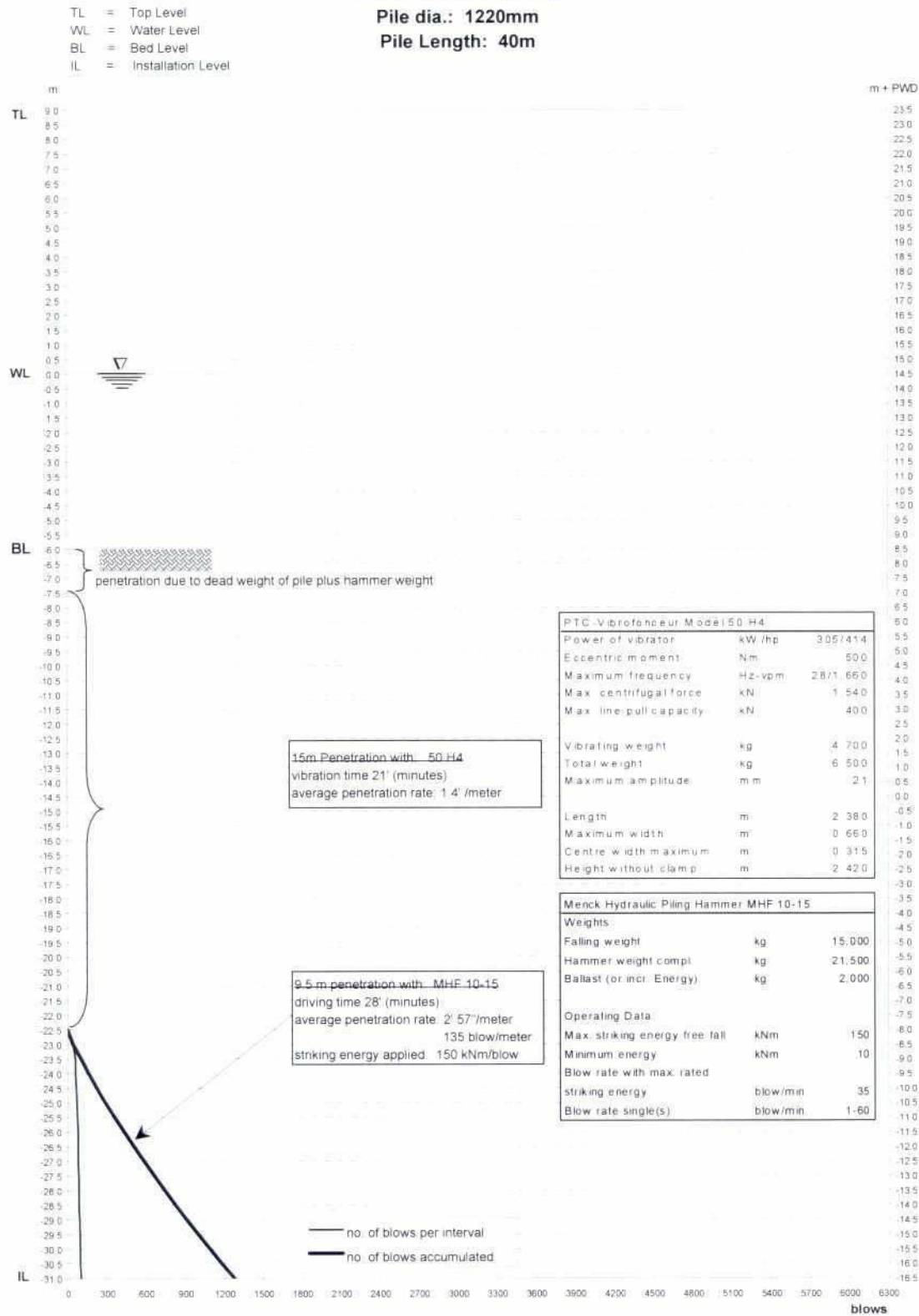


Fig. 7.11-4: Pile Installation No. 29A (G-1), dia. 1220 mm – Test Site I (Kamarjani)

Groyne G-3
Pile Installation No. 34A
Pile dia.: 1220mm
Pile Length: 40m

TL = Top Level
 WL = Water Level
 BL = Bed Level
 IL = Installation Level

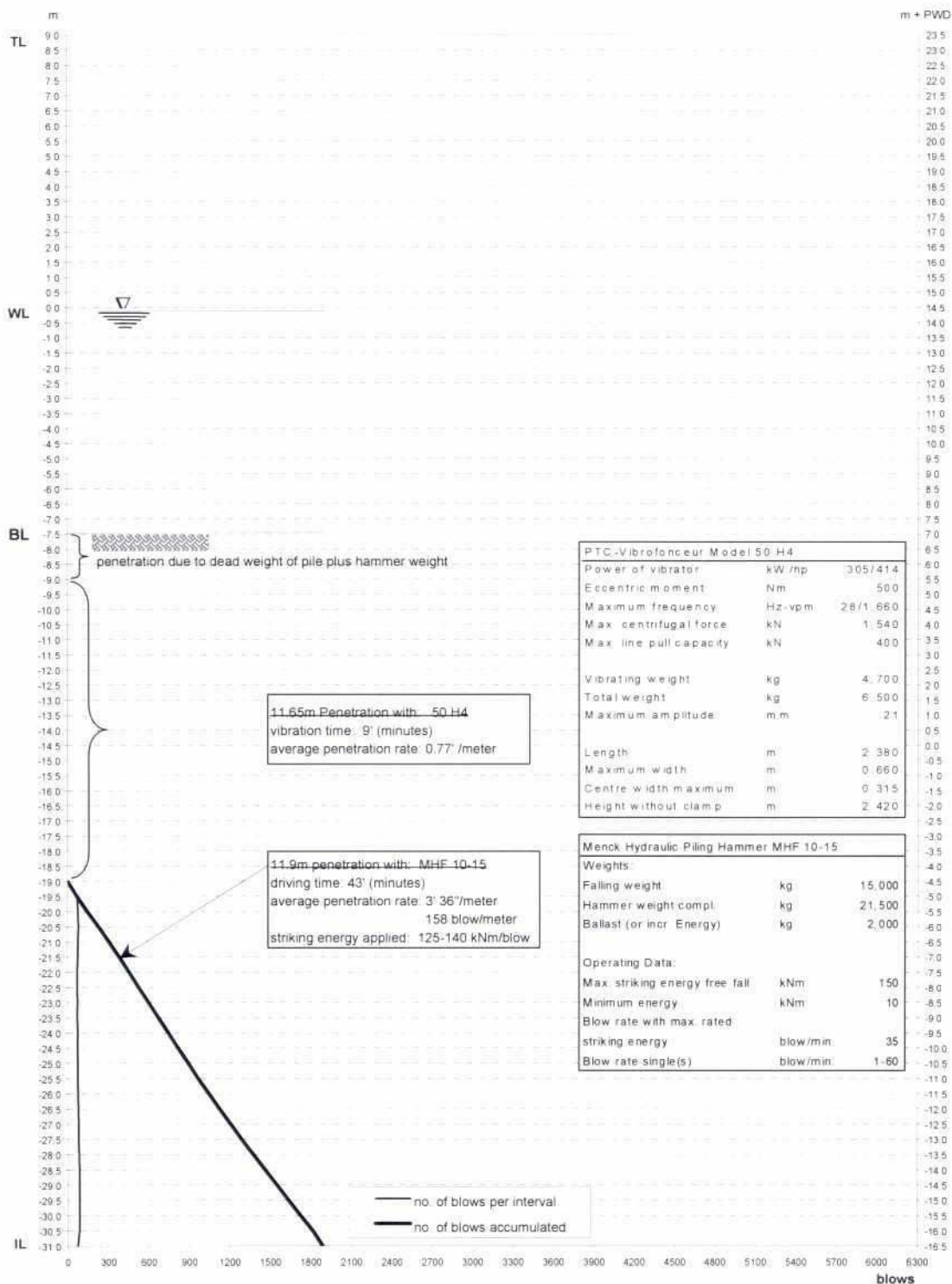


Fig. 7.11-5: Pile Installation No. 34A (G-3), dia. 1220 mm – Test Site I (Kamarjani)

- Hydraulic free fall piling hammer:
trademark and type:
Menck hydraulic free fall piling hammer. Model MHF-10-15.
Drop weight 15 ton, Stroke 0.1 meter minimum, 1.07 meter maximum.
Blow rate 40 blows/min on max. stroke.
Total weight including helmet 35 ton
- Power pack for both the vibration and the hydraulic free fall hammer:
trademark and type:
PTC power pack model MH 600.
With 375 kW (510 hp) caterpillar 3408 engine
Accessories:
Hydraulic hoses, remote command cables
- Crawler crane for off-shore dredging, pile handling and other lifting work.
trademark and type:
IHI, Japan, Model CCH-400
25 m boom
Maximum crane capacity 45 ton.
- Flat-Barge for carrying the piles for offshore piling from the welding yard to the piling barge and for off-shore dredging with IHI crane.
Main dimensions: 24.2 x 6.1 meter (80'x20') height about 2.0 meter, maximum load capacity 200 ton.
- Unifloat for miscellaneous works and to carry piles to the piling barge. Main dimensions: of each element 5.4 x 2.4 x 1.22 meter (18'x8'x4'). Total 12 elements including 4 end pieces.
Maximum load each 20 ton.
The elements are easily assembled to have floats of different size and capacity.
- Twin engine tugboat, 400 Hp, used for towing of barges and shifting of anchors.

(c) Material used:

Steel piles steel grade St 37-3, diameter 1016 x 20 mm, length 32 m and diameter 1220 x 20 mm length 36 and 40 m.

(d) Steel Pile dia. 1016mm Performance Data

Total quantity	:	G-1: 6 piles; G-2: 6 piles; G-3: 6 piles (all 32m)
Net piling depth	:	14 to 20 meter
Net working days	:	10 days
Maximum per day	:	4 piles
Minimum per day	:	1 piles
Average per day	:	1.8 pile per day
Net piling time	:	20' to 60' per pile, only vibration (20 m penetration) 20' to 30' per pile, only vibration (14 m penetration)
Total installation time	:	3h to 6h per pile

(e) Steel Pile dia. 1220mm Performance Data

Total quantity	:	G-1: 6 piles (3x 36m; 3x 40m) G-2: 8 piles (4x 36m; 4x 40m) G-3: 12 piles (9x36m; 3x40m)
Net piling depth	:	23 to 27 meter
Net working days	:	16 days
Maximum per day	:	2 piles
Minimum per day	:	0.25
Average per day	:	1.6 piles per day
Net piling time	:	30' to 60 ' for 13 to 14 m vibro piling 60' to 90' for hammer piling
Total installation time	:	7h to 8h per pile

**Photo 7.11-5: Pile Transport Barge**

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Photo 7.11-6: Pile lifting



Photo 7.11-7: Pile lifting



Photo 7.11-8: Pile positioning



Photo 7.11-9: Pile positioning and fixing into the Pile Guiding Frame

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Photo 7.11-10: Piling of Steel Piles dia 1016 mm and 1220 mm by Vibrator PTC 50 H4



Photo 7.11-11: Piling of Steel Piles dia 1016 mm and 1220 mm by Hydraulic Hammer Menck MHF 10-15

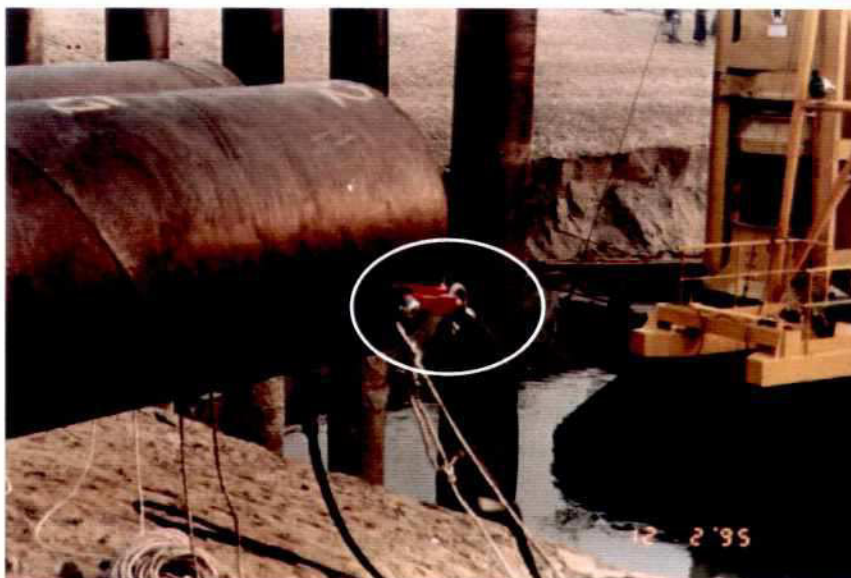


Photo 7.11-12: Detail: “Release Shackle”

7.12 INSTALLATION OF PRE-CAST CONCRETE PILES

This subsection covers the installation of pre-stressed concrete spun piles, dia. 500 mm x 100 mm wall thickness, length up to 20 m at Groynes G-A, G-B/1 and G-B/2.

The pre-stressed spun concrete piles were delivered in 10 m lengths. For a 20 m long pile two 10 m pile sections were welded together at special integrated steel butt plates at the top/end of the piles. The piles were aligned properly on a 12 m long HEB 300 H-beam and tack welded to ensure the correct alignment before definite welding took place.

Spun-piles were installed at following locations:

G-A	:	18 numbers of length 20 meter
G-2 (monitoring house)	:	4 numbers of length 10 meter and 2 numbers of length 20 meter
G-B/1	:	8 numbers of length 10 meter and 20 numbers of length 20 meters
G-B/2	:	9 numbers of 10 meter length and 20 numbers of 20 meter length.



Piling of prefabricated concrete spun piles started at the land side slope of the later to be constructed embankment at Groyne G-2 with 4 piles of 10m and 2 piles of 20 m length. These piles had been designed as foundation of the “monitoring house” and were considered as piling test. All piles could be installed to design depth without problems, except one 20 m pile where the head of the pile was damaged before reaching the design depth.

This result showed that special attention must be given to the pile helmet, the cushion and the pile force. The prefabricated concrete spun piles were placed on the ground horizontally at their place of installation. The crawler crane with the leader was placed in the exact position before lifting the piles. After lifting the pile and placing the helmet on the piles, the correct position was checked by spirit level and survey instruments. After final check the piles were driven to the design depth. Depending

on the scale of penetration the stroke of the hammer was adjusted. At all the piles plug formation occurred after 4 m of driving. All the 10 m piles were driven to the design depth with out any problem. In case of 20 m piles, it was observed that the rate of penetration is very slow after about 12 m of penetration. Two pile heads failed in groyne G-A, therefore a 20 mm reinforcement bar was welded at the bottom end of the pile to reduce the side friction. With this modification of the bottom ring all the remaining piles in G-A could be installed to design depth with out problem or damage to the pile.

At GB-1, the penetration of some piles was very slow even with the modified bottom due to plug building. So water jetting inside the pile to remove the plug was applied, 4.5 to 4 meter before reaching design depth. A 2 inch tube with a special end piece was introduced with a high pressure of 10 bar, 60 m³/hr pump. The water jetting with 10 bar pressure continued till the jetting lance reached one meter below the pile. Hammering continued till the piles reached the design depth. During water jetting the position of the leader was not changed to maintain the alignment of the piles and the leader. To avoid damage to the pile head when high pile force was needed, two layers from 65 mm of soft wood and coconut coir was used in the helmet to absorb the shock wave. The soft wood and coconut coir were replaced, when they became stiff. At the beginning of each pile new wood and coconut coir were used. But anyway, in two cases the design depth could not be reached (less than one meter) due to damage of the pile head.

(a) Main equipment used for concrete spun pile driving:

- Crawler crane: (concrete spun pile driving and general lifting jobs)
trademark and type:
R-B (Lincoln) model 600 SC super crane with:
19 meter boom.
two winches one for lifting the hammer and one for lifting the pile
Maximum crane capacity 60 tons
27 meter leader with special attachment to the crane (maximum pile length below helmet to ground level 20 meter).
- Hydraulic free fall piling hammer:
trademark and type:
International Foundation Ltd., Model BSP 357-7 HH MK 3
Adjustable drop weight 3, 5 or 7 ton (we used with maximum drop weight of 7 ton).
Stroke minimum 0.2 meter, maximum 1.2 meter.
Blow rate 40 blows/min on max. stroke.
Total weight including helmet about 10.3 ton.
- Power pack for BSP hammer (fixed as counterweight to the RB crane).
trademark and type:
Hydro pack with Perkins diesel engine 112 KW

Partly also used:

- Piling rig KOBE 22, with 25 meter long leader, guiding the hammer by two round bars 70 mm diameter and centre to centre distance of 330 mm.
Lifting by electric winch with 6 ton capacity.
The leader could rotate on the chassis of the rig, the rig itself was running on rails parallel to the pile rows.
Movement and rotation by electric engines.

- Generator for powering the rigs
Trademark and model:
Petbow 100 KW.
- Kobe 22, water cooled diesel hammer, ram weight 2.2 ton, maximum energy 6.15 tonmeter at 45 blows/min.
- High pressure water jet pump used for spun piles
trademark and model:
not known, driven by Cummins 125 HP engine.
Minimum pressure 10 bar at 60 litre/min.

(b) Concrete Spun Pile dia. 500 mm Performance Data:

Total quantity	:	81 nos. (21 nos. of 10 m length, 60 nos. of 20 m length)
Net piling depth	:	up to 19 meter
Net working days	:	36 days
Maximum per day	:	4 piles
Minimum per day	:	1 piles
Average per day	:	2.25 piles per day
Net piling time	:	70' to 200' per pile, only vibration (19 m penetration)
Total installation time	:	2h to 6h per pile

202

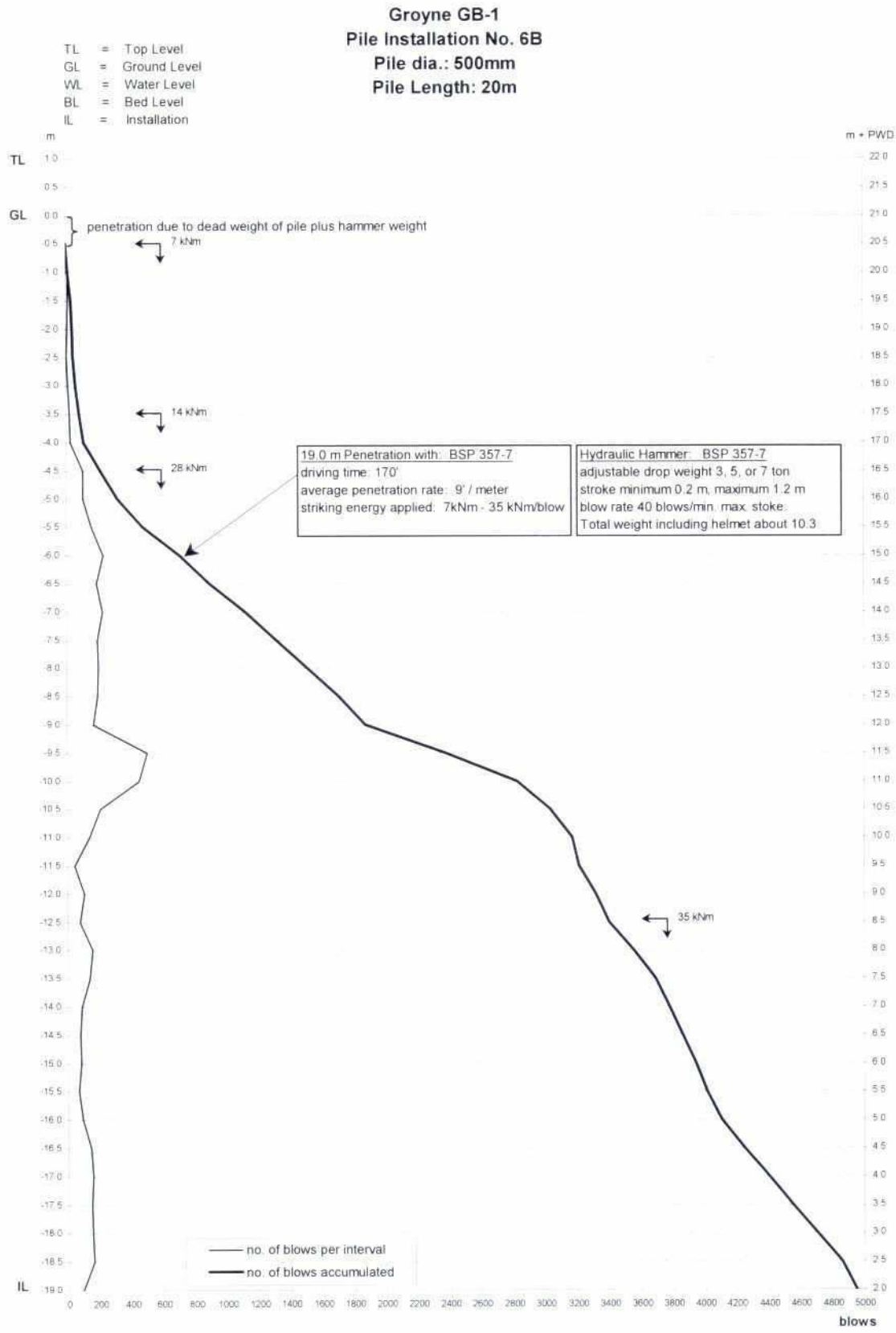


Fig. 7.12-1: Pile Installation No. 6B (GB-1), Spun Pile – Test Site I (Kamarjani)

206

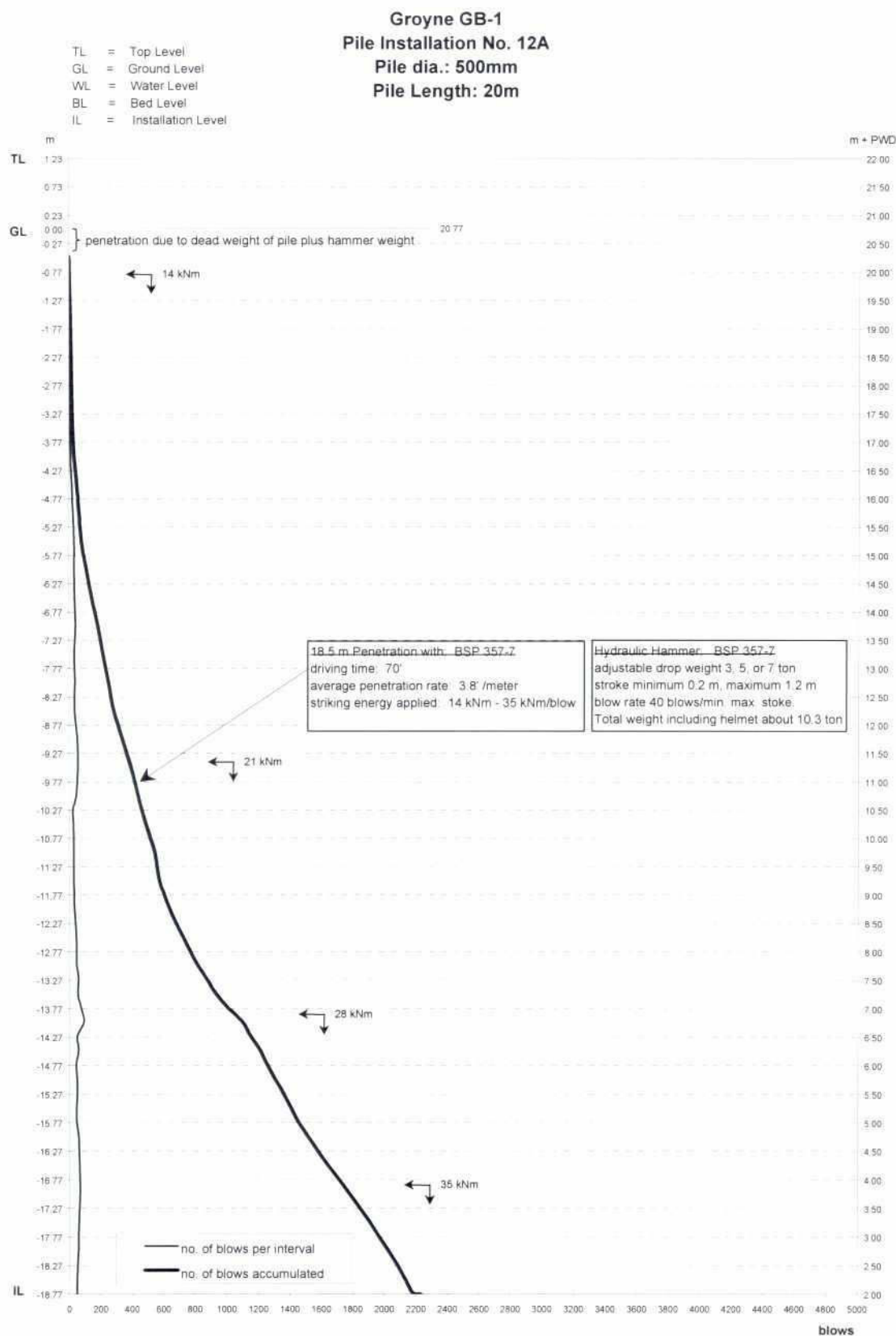


Fig. 7.12-2: Pile Installation No.12A (GB-1), Spun Pile – Test Site I (Kamarjani)



Photo 7.12-1: Piling pre-cast Concrete Spun Piles, Piling Rig, Diesel Pile Hammer Kobe 22



Photo 7.12-2: Piling pre-cast Concrete Spun Piles, Piling Rig, Diesel Pile Hammer Kobe 22



Photo 7.12-3: Piling pre-cast Concrete Spun Piles, Piling Rig, Hydraulic BSP Hammer



**Photo 7.12-4: Piling pre-cast Concrete Spun Piles, Piling Rig, Hydraulic BSP Hammer;
Detail: Fixing of Special Pile Head Cushion**

7.13 SHEET PILE INSTALLATION

This subsection covers the installation of sheet-piles for construction of the cofferdams as follows:

- Construction of steel sheet pile cofferdam at Groyne G-1, on-shore, length 60 m, width 7.9 m, and
- Construction of cofferdam at Groyne G-A, on-shore, with reinforced concrete sheet piles, length 60 m, width 8.85 m

Steel sheet-piles were supplied in exact dimensions, no further preparation was required at site. Reinforced concrete sheet-piles were supplied from the factory in exact dimensions. No further preparation was needed at site prior to installation.

7.13.1 Steel Sheet Piling

The sheet piles of the cofferdam at groyne G-1 were installed at the very beginning with piling rig Kobe 22 and diesel hammer Kobe 22 (10 sheets only). Later the Kobe rig and hammer were replaced by a Crawler crane and an electric driven vibration hammer in order to speed up the installation and to free the Kobe rig for other steel pile installation.

The double and single sheet piles of 7 and 8 meter length, some of them with corner profiles, were first installed with a two level guiding frame made from H-profile steel beams.

All steel piles were driven by "pilgrim method" (echelon formation) to design depth in different steps. At the final step the guide frame was reduced to one level. The alignment was, if necessary, slightly adjusted to assure correct locking of the corner profiles in the joints (ref. to Drawing KA-008, Attachment 1).

After piling was completed the cofferdam was partly back filled and the anchors were installed. The anchors were tensioned with turnbuckles in the middle of each anchor.

(a) Main equipment used for steel sheet piling

- Crane for steel sheet piling.
trademark and type:
Linkbelt crawler crane with
28 meter boom
Maximum crane capacity 45 ton.
- Electric vibration hammer.
trademark and model:
not known (no more labels)
- Generator used
trademark and model:
not known (no more labels)

Partly used:

- Piling rig KOBÉ 22, with 25 meter long leader, guiding the hammer by two round bars 70 mm diameter and centre to centre distance of 330 mm.
Lifting by electric winch with 6 ton capacity.

The leader could rotate on the chassis of the rig, the rig itself was running on rails parallel to the pile rows.

Movement and rotation by electric engines.

- Generator for powering the rigs
Trademark and model:
Petbow 100 KW.
- Kobe 22, water cooled diesel hammer, ram weight 2.2 ton, maximum energy 6.15 tonmeter at 45 blows/min.

(b) Material used

- Steel sheet piles ARBED PU-6, ST.SP 37/ STE240 SP
Double sheets, single sheets, corner sheets, length 7m and 8 m
- Anchor material:
round steel bar anchors, brackets, waling uu-profiles, anchor plates, bolts and nuts

(c) Steel sheet piles performance data

Total quantity	:	130 sheets (10 sheets piled with hammer)
Net piling depth	:	6 meter
Net working days	:	7 days
Maximum per day	:	41 sheets (vibrator), 6 sheets (hammer)
Minimum per day	:	14 sheets (vibrator), 4 sheets (hammer)
Average per day	:	17 sheets (Vibrator), 5 sheets (hammer)
Net piling time	:	1' to 2' (max. 4') per sheet (vibrator), 8' to 14', (max. 31') per sheet (hammer)
Total installation time	:	20' to 60' per sheet including shifting of rig and guide frame modification of upper level

204

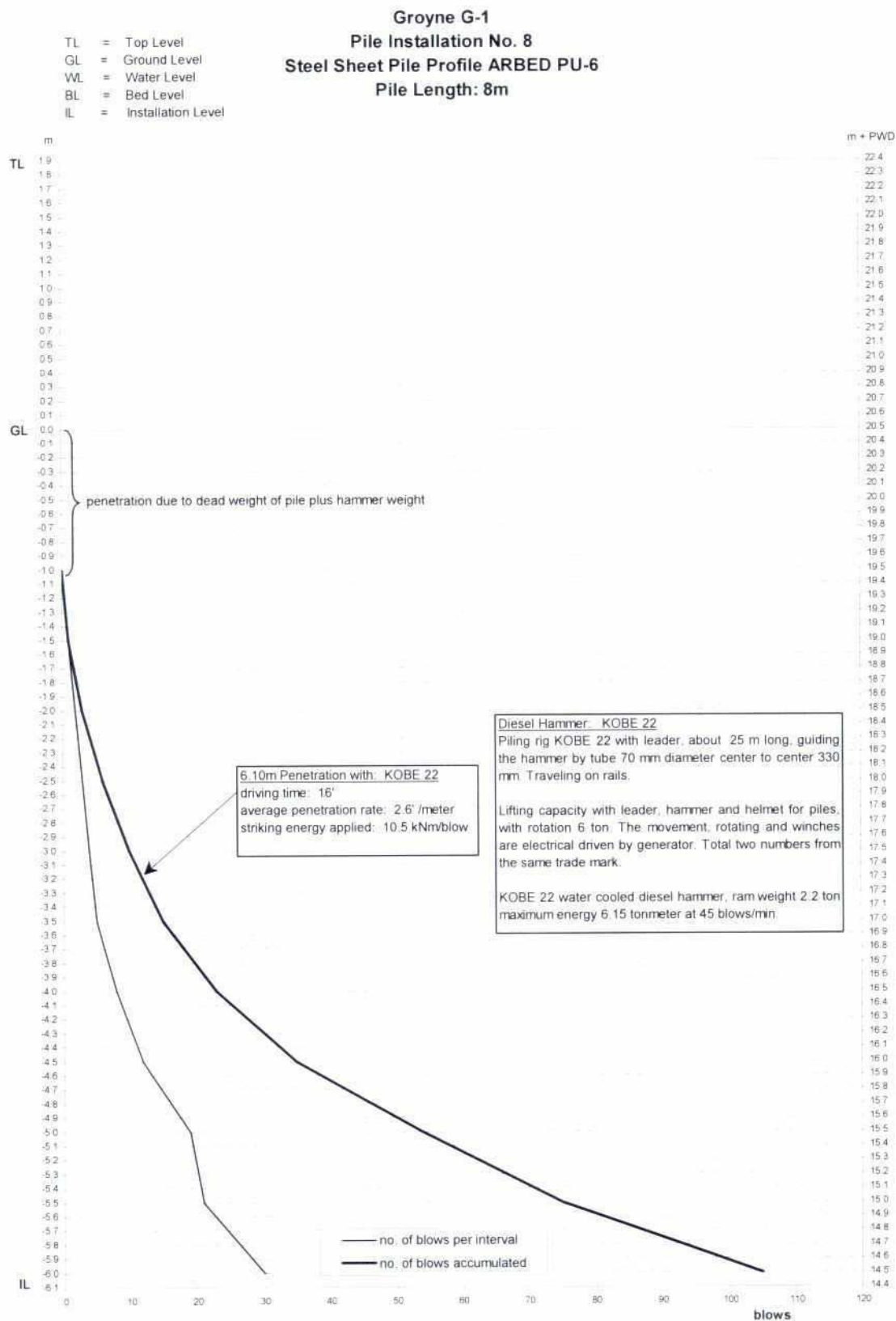


Fig. 7.13-1: Pile Installation No. 8 (G-1), steel sheet pile - Test Site I (Kamarjani)



**Photo 7.13-1: Steel sheet pile installation at Groyne G-1;
Piling with Electric Vibrator**



**Photo 7.13-2: Steel sheet pile installation at Groyne G-1;
Piling with Kobe Diesel Hammer**

7.13.2 Concrete Sheet Piling

Concrete sheet piles were installed in G-A only. At first a double line guide frame from H-beam steel profiles was made at the bottom and 2 m from the bottom. The concrete sheet pile was pitched in position, checked by the surveyor and driven with the KOBE 22 pile driving rig and the Kobe 22 diesel hammer. With this guide frame the pile could not be kept in vertical position. To solve the problem, the guiding frame was given lateral support at two elevations and in addition the head of the sheet pile was kept in position with cables manoeuvred by separate mechanical winch. With this improvement, all the piles were driven to design depth without problems. Special attention has to be given to the piling cushion and the piling force because the head of a concrete sheet pile is fragile.

(a) Main equipment used for steel sheet piling

- Piling rig KOBE 22, two numbers, with 25 meter long leader, guiding the hammer by two round bars 70 mm diameter and centre to centre distance of 330 mm.
Lifting by electric winch with 6 ton capacity.
The leader could rotate on the chassis of the rig, the rig itself was running on rails parallel to the pile rows.
Movement and rotation by electric engines.
- Generator for powering the rigs
Trademark and model:
Petbow 100 KW.
- Kobe 22, water cooled diesel hammer, ram weight 2.2 ton, maximum energy 6.15 tonmeter at 45 blows/min.

(b) Main material used

Locally manufactured concrete sheet piles 490 mm x 250 mm, 8m long

(c) Concrete sheet pile performance data

Total quantity	:	305 sheets
Net piling depth	:	3.8 meter
Net working days	:	29 days
Maximum per day	:	26 sheets/per day
Minimum per day	:	1 Sheet per day
Average per day	:	10.5 sheets per day
Net piling time	:	10' to 30' per sheet
Total installation time	:	30' to 60' per sheet

922

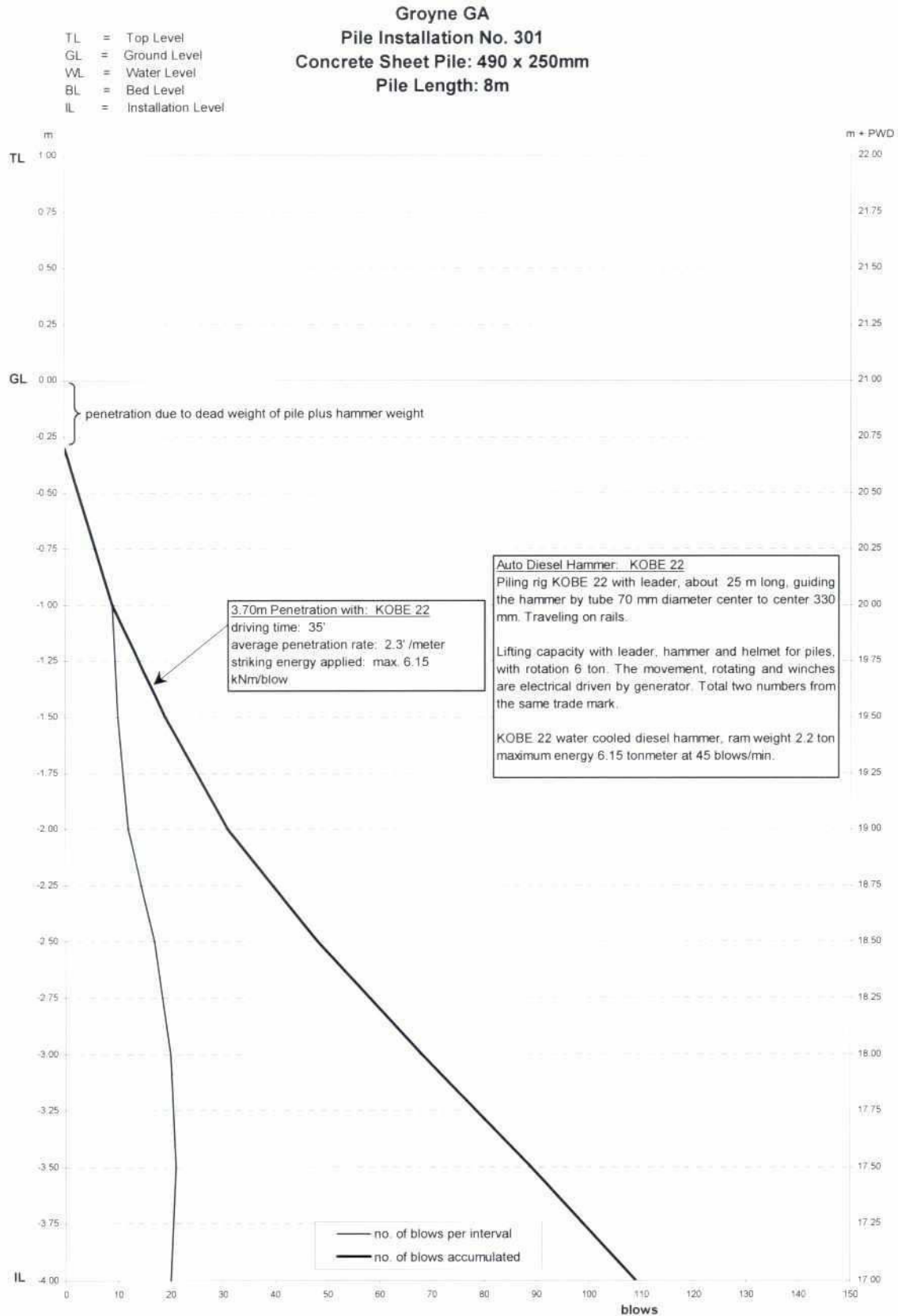


Fig. 7.13-2: Pile Installation No. 301 (GA), concrete sheet pile – Test Site I (Kamarjani)



**Photo 7.13-3: Concrete Sheet Pile installation;
Piling with Diesel Hammer Kobe
22**



**Photo 7.13-4: Concrete Sheet Pile installation;
Piling with Diesel Hammer
Kobe 22**



Photo 7.13-5: Concrete Sheet Pile Guide Frame



Photo 7.13-6: Concrete Sheet Pile Guide Frame

7.14 BORED CONCRETE PILE INSTALLATION

(a) Installation

This subsection covers the installation of bored in-situ cast reinforced concrete piles, diameter 914 mm, length 22 m (reinforced length) including permanent steel tube lining within the upper 11 m to 15 m (see Drawing KA-402/1 and KA-015, Attachment 1). In situ bored piles, diameter 914 mm were executed on-shore at Groyne G-A, G-B/1 and G-B/2. Drilling with Bentonite slurry started at row No. 9 of Groyne G-A. The drilling could be completed to the design depth without any difficulty, but the cage of reinforcement steel could not be installed to depth completely due to deposit of sand at the bottom of the bored hole. Ultimately this first bore hole had to be abandoned since the reinforcement cage could not be recovered for re-drilling.

The failures are attributed to some or all of the following reasons:

- 1 The quality of the bentonite slurry was not as per specification. (Bentonite available in the market is poor quality)
- 2 The arrangement to separate sand from the slurry during drilling was not adequate and as a result a lot of sand remained suspended and settled at the bottom of the bored hole during waiting time to lower the reinforcement cage.
- 3 Due to short lifting capacity the 22m long reinforcement cage could not be lowered into the bore hole in its full length and as such the cage were lowered in two parts with overlapping bars, welded together in the hole. This method is time consuming and allowed the settlement of sand at the bottom of the bore hole.
- 4 Disturbance in the soil due to nearby piling of concrete sheet piles at the cofferdam of groyne G-A.

Since improvement of the first two reasons was considered difficult and time consuming, it was tried to drill just with water, keeping a permanent over pressure in the casing pipe. A test boring was done up to design depth by reverse rotary drilling machine and left over night for observation. The result was that with the continuous supply of water to maintain the water level over ground level no change in the bore hole occurred except a deposit of about 300 mm of sand.

It was therefore decided:

- a To use a crane capable to handle and lower the reinforcement cage in full length in order to avoid welding of the cage and to allow an early start of in situ concreting of the piles.
- b To drill with water pressure by reverse rotary drilling machine and to drill the hole at minimum 1 m deeper than the design depth to create a reservoir for sand deposit.
- c To start drilling from the bank site of the river to reduce effect of the installation of sheet piles of the cofferdam.

The centreline and the centre point of the piles were marked on ground by the surveyors. A temporary steel casing pipe about 2 meter long with a side connection for water filling were installed manually. A reservoir was kept permanently full with water pumped from the river to guarantee the water level in the bore hole. At first the hole was drilled up to 1 meter above the level of permanent casing then the permanent casing pipe was installed. In this way all casing pipes of the groyne were installed before final drilling of the holes up to the design level (plus 1 meter) continued. After drilling the hole was "cleaned", exchanging the "drill water" with fresh water by continuous pumping till the water was sand free, but at least for 1.5 hours. After cleaning the drilling rig and the temporary casing were removed.

After installation and securing the correct position of the prefabricated reinforcement the bore hole was filled with concrete. Under water concreting started from the bottom of the bore hole using a 200 mm diameter concreting tremie pipe with a funnel on the top. During the concreting the tremie pipe was kept at least 50 cm in the already poured concrete to avoid mixture with water. The concrete flow was assured moving up and down the concreting pipe. The concreting pipe was shortened in intervals of 3 m according to the concrete progress.

Preparation and drilling of the bore holes started in general in the afternoon and was completed in the early morning hours of the next day, followed by concreting up to noon time. With this rhythm one in-situ bored pile could be installed per day.

At G-A all the bored piles were installed to the design depth except of pile number 13 and 14 where drilling had to be stopped about 3 meter before reaching the design level due to the presence of petrified wood. To remove this obstacle by chiselling with a 12 meter HEB 200 H-beam failed. Since the difference between the design level and the achieved level was small, the pile was installed with the shorter length.

The abandoned bore hole at row number 9 was filled up with dry non cohesive sand and replaced by two 20 meter long spun piles on the same axis.

At groyne G-B/1 and G-B/2 the in-situ concrete piles were installed in the same way as in G-A. No further difficulties were encountered in these two groynes. All the piles were installed to design depth.



Photo 7.14-1: Drilling of In-Situ Bored Piles, Reverse rotary Method



Photo 7.14-2: Lifting of Reinforcement Cage of In-Situ Bored Piles



Photo 7.14-3: Concreting of In-Situ Bored Piles, under Water Method



Photo 7.14-4: Ready In-Situ Bored Piles with the permanent Steel Casing

(b) Pile Integrity Testing

Selected piles have been tested by the pile integrity test utilising the so called “Low –Strain Method of Dynamic Pile Testing”.

This method measures the running time of a stress wave generated by hammer impact at pile held, which is reflected at the bottom of the pile. When the reflected stress waves returns to the pile top, a measurable pile top motion occurs. If this reflection occurs at the correct time and no other earlier reflection waves are received at the pile top, then the pile shaft is probably free of major defects.

The results of the pile tests are shown in Table 7.14-1

A typical wave diagram is shown in Fig. 7.14-1.



Photo 7.14-5: Pile Integrity testing of In-Situ Bored Piles



**Photo 7.14-6: Pile Integrity testing of In-Situ Bored Piles;
Measurement of the Stream Wave generated by
Hammer Impact**

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SUMMARY OF PIT RESULTS:**APPENDIX – A**

Pile No.	ID. No.	Results of Test	Remarks
28/G-B/2	141	No defect observed.	Pile O.K.
27/G-B/2	142	No pile toe reflection apparent.	Pile O.K.
24/G-B/2	144	Significant impedance changes due to poor compaction.	Pile O.K.
22/G-B/2	145	Toe reflection is at 21.71 M.	Pile O.K.
27/G-B/1	146	Toe reflection is at 21.71 M.	Pile O.K.
25/G-B/1	147	Toe reflection is at 20.91 M.	Pile O.K.
24/G-B/1	148	Diameter decrease at 12 M.	Pile O.K.
22/G-B/1	149	Toe reflection not significant.	Pile O.K.
21/G-A	150	Impedance changes due to poor compaction.	Pile O.K.
17/G-A	151	Significant impedance changes due to poor compaction.	Pile O.K.
14/G-A	152	Toe reflection is at 22 M.	Pile O.K.
11/G-A	153	Significant impedance changes.	Pile O.K.

Table 7.14-1: Results of Pile Testing of in Situ Bored Piles– Test Site I (Kamarjani)

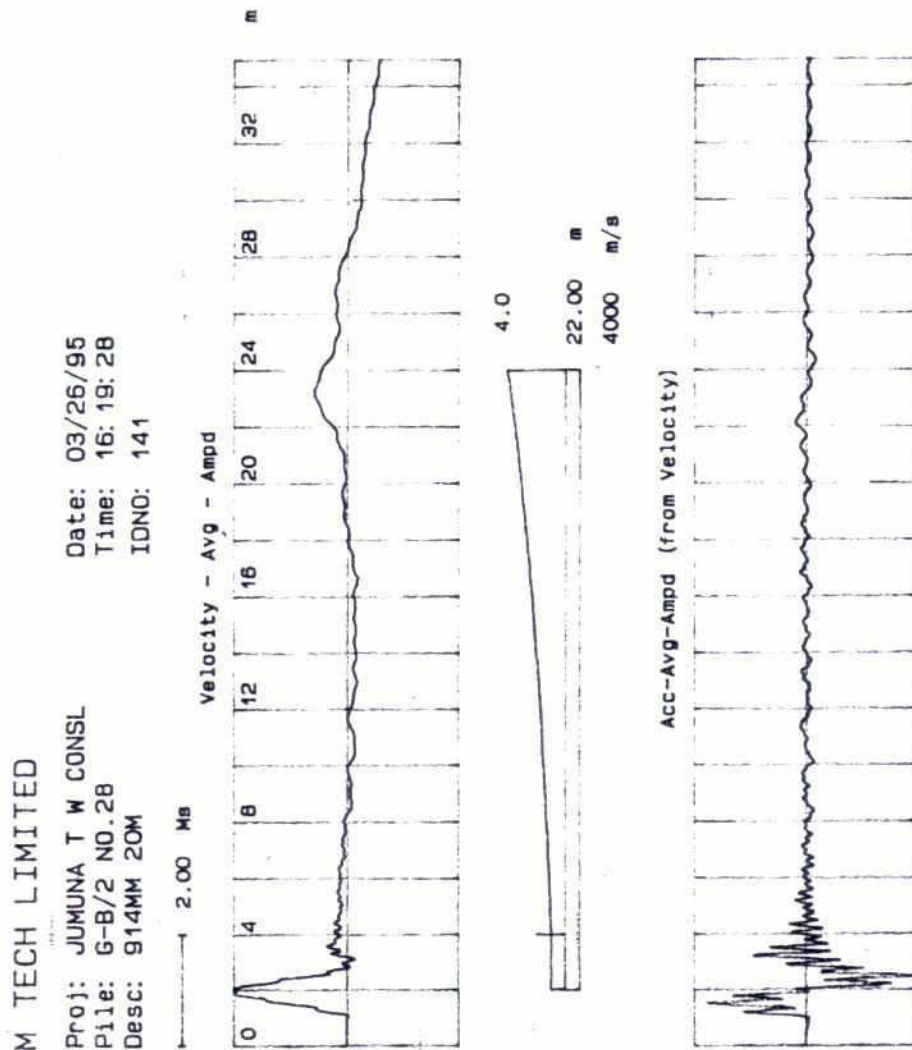


Fig. 7.14-1: Wave Diagram of in Situ Bored Pile No. 28 at Groyne G-B/2 – Test Site I (Kamarjani)

(c) Main equipment used


- 2 Nos. drilling rig mounted on a tractor, Trademark and model: Hydreq Hanor Reverse Rotary type 70 HP with hydraulic controlled drilling unit max. torque moment 500 kpm and maximum lifting capacity 2 ton. , mast length about 8 meter;
- 2 Nos. tripod lifting device, lifting height below hook 11 meter, lifting capacity about 2 ton;
- 4 Nos. concrete mixing machines 10/7 cubic feet capacity;
- Electrical winches 2 ton capacity;
- Mud pumps, capacity 3 cubic feet/sec with Lister diesel engines, and
- Water pumps, China, capacity 0.5 cubic feet/sec.

(d) Main material used

- Cement: local and imported Portland cement Type 1 according to specification and to ASTM;
- Sand: coarse sand FM.> 2.5, well graded;
- Aggregates: 16 mm downgraded crushed stones;
- Reinforcement steel: locally produced steel bars, Grade 60 according to ASTM A-615, and
- Steel tube lining (casing): steel pipes, 914 mm diameter and wall thickness 6 mm, Steel grade St 37 (produced locally from second hand steel plates).

(e) Bored Piles Performance Data:

Total quantity	:	36 piles nominal length 24 m (G-A; G-B/1; G-B/2 each 12) Reinforcement cage 22m; permanent casing 11 to 15 m
Net boring depth	:	22 to 24 meter (including additional 1.5 to 2.5 meter to allow settlement)
Net working days	:	36 days
Average per day	:	1 pile per day per equipment unit
Net boring time	:	5h to 6h during afternoon and night hours till morning
Net concreting time	:	4h to 5h (approx. 15.5 m ³ per pile; theoretical 14.5 m ³) morning to noon hours
Total net installation time	:	11h to 13h including installation of reinforcement cage and permanent casing

(PROJECT NAME) BORED PILE PRODUCTION REPORT		Page 1 of 3
1. BASIC DATA Contractor: <u>Consortium T.E.L/CL</u> Construction Site: <u>FAP-21 Kamanjani T/C works</u> Reference Drawings: _____		Pile Location No.: <u>G-31</u> Bored Pile Production No.: <u>23</u> Date: <u>10.01.95</u>
1. Pile Data 1.1 Nominal Diameter of Pile: <u>914</u> mm 1.2 Casing Data: - temporary casing: <u>2000</u> mm - permanent casing: <u>13000</u> mm 1.3 Drilling Tool (type): <u>Rotary drill</u> 1.4 Outer Diameter of: Drilling Tool: <u>470</u> mm Cutting Edge: <u>300</u> mm 2. Reinforcement of Pile 2.1 Reinforcement Drawing No.: <u>503</u> 2.2 Installation of Reinforcement Cage: before concreting: <input type="checkbox"/> after concreting: <input type="checkbox"/> 2.3 Spacers: Type: <u>Flat bar</u>  <u>20</u> mm Spacing in Longitudinal Direction: _____ mm 3. Concrete Mix 3.1 Type (Strength Class): B <u>35</u> 3.2 Consistency/Slump: <u>7609</u> cm 3.3 Mixing Plant: Type: <u>mix ture machine</u> Capacity: <u>0.165</u> m ³ /mix 3.4 Cement: <u>Citra song cement cleaner</u> Manufacturer: <u>Portland Cement</u> Type: <u>Type - 1</u>	3.5 Quantity per 1 m ³ Concrete: <u>400</u> kg/m ³ 3.6 Aggregates: Combined Aggregate Curve No.: <u>A-16, B-16</u> Max. Grain Size: <u>16</u> mm 3.7 Water-Cement Ratio: <u>0.50 - 0.55</u> 3.8 Concrete Additives: Type/Brand Name: _____ % of Cement Content: _____ (by Volume) 3.9 Retarder: Type/Brand Name: _____ Retard Time: _____ h 4. Concreting Procedure 4.1 Means of Placing Concrete: Tremie Pipe ϕ : <u>200</u> mm <input checked="" type="checkbox"/> Pumping Pipe ϕ : _____ mm <input type="checkbox"/> Other means: _____ <input type="checkbox"/> Description: _____ 4.2 Measures to Clean Bore Hole Bottom: <u>One hour washing by water size 40mm</u> 4.3 Measures to Separate Concrete from Bentonite Suspension at the Start of Concreting: <u>N/A</u> 5. Remarks/Comments <u>Add boring done to enter</u> <u>Side measurement of suspended solid</u>	

☐ Tick-mark where applicable

Fig. 7.14-2/1: Typical Bored Pile Production Report (Sheet 1 of 3)

(PROJECT NAME) BORED PILE PRODUCTION REPORT					Page 2 of 3									
II. VARIABLE BORED PILE PRODUCTION DATA														
Date of Boring: <u>10-01-95</u>			Pile Location No.: <u>G13/1</u>											
Date of Concreting: <u>10-01-95</u>			Bored Pile Production No.: <u>23</u>											
Soil Profile					1. Pile Data									
Depth		Type of Soil	Ground Water Table	Length of Casing	1.1 Check of Borehole Depth:									
Below Ground Level [m]	Referred to ± 0.0 m		Below Ground [m]	from... to... m	1.2 Bitdrilling from <u>2.00</u> m to <u>22.60</u> m below ground level									
0.00 2.00	20.368 18.368	clay			1.3 Deviations from Designed Location: X-Axis: <u> </u> cm Y-Axis: <u> </u> cm									
2.00 7.00	16.368 13.368		medium plastic clay											
7.00 9.00	13.368 11.368	Fine sand & silt			2. Data of Bentonite - Suspension (actual measured data)									
9.00 16.00	11.368 4.368	Fine to medium sand			<table border="1" style="width: 100%; border-collapse: collapse;"> <thead> <tr> <th></th> <th style="text-align: center;">At Start of Concreting</th> <th style="text-align: center;">After Concreting</th> </tr> </thead> <tbody> <tr> <td style="text-align: center;">T_p</td> <td></td> <td></td> </tr> <tr> <td style="text-align: center;">P_p</td> <td></td> <td></td> </tr> </tbody> </table>		At Start of Concreting	After Concreting	T _p			P _p		
	At Start of Concreting	After Concreting												
T _p														
P _p														
16.00 19.00	4.368 1.368	Medium to coarse sand			Level of Bentonite - Suspension (actual measured data): _____ m above bottom level of casing _____ m above ground water table									
19.00 22.60	1.368 2.232	Fine to medium sand			3. Reinforcement of Pile									
					3.1 Deviations from Reinforcement Drawing No.: <u>503</u>									
					3.2 Alteration of Pile Length: <u> </u> m									
					3.3 Alteration of Reinforcement (Reason): <u> </u>									
					3.4 Other Alterations: <u> </u>									
					4. In-Situ Concrete Special Observations: <u> </u>									
					5. Concreting									
					5.1 Level of Suspension at Start of Concreting: <u> </u> m above lower edge of casing									
					5.2 Verification of Concrete Consumption Theoretical: <u>14.43</u> m ³ Actual consumed: <u>15.685</u> m ³									

Fig. 7.14-2/2: Typical Bored Pile Production Report (Sheet 2 of 3)

(PROJECT NAME) BORED PILE PRODUCTION REPORT				Page 3 of 3 Pile Location No. <u>CR 3/1</u> Bored Pile Production No. <u>23</u>	
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6. Bored Pile Construction Time

Action	Surrounding Temperature °C	Execution Date		Execution Time	
		Start	End	Start	End
Drilling		10.01.95	10.01.95	2.00 AM	6.50 AM
Installation of Permanent Casing		10.01.95	10.01.95	9.30 AM	9.40 AM
Bitdrilling		10.01.95	10.01.95	2.00 AM	6.50 AM
Interruption		10.01.95	10.01.95	2.35 PM	3.15 PM
Concreting		10.01.95	10.01.95	10.30 AM	3.45 PM

7. Concrete Compressive Strength Tests

7.1 Size of Test Specimen 15 cm x 15 cm x 15 cm

7.2 Identification Numbers of Test Specimen _____

7.3 Compressive Strength Test Results

ID - No	7-days test [N/mm ²]
1	28.8
2	29.9
3	33.6
Average	30.8

ID - No	28-days test [N/mm ²]
Average	

8. Remarks Test result for 7 days as for 20 cm x 20 cm x 20 cm specimens

Deviations from Basic Data No.

9. Signatures

Contractor's Drilling Master : [Signature] Date: 10.01.95

Contractor's Superintendent : [Signature] Date: 10.01.95

Engineer's Representative : [Signature] Date: 10.01.95

Fig. 7.14-2/3: Typical Bored Pile Production Report (Sheet 3 of 3)

CONCRETE COMPRESSIVE STRENGTH TEST

[illegible]Submitted by
Lab. Incharge
Contractor

7.15 SECURING OF TUBULAR STEEL PILES

The top of all steel piles of the groynes were sealed with steel plates welded to the top. The full pile row of a groyne was braced with H-type steel girders to secure the pile against current induced vibration. A lockable gate at the shore-sided end of each groyne secured the unauthorised access.

At groyne no. G-2 a steel gangway of 1.20 meter width was installed, for the only purpose to serve the monitoring equipment platform installed at the river-sided end of this groyne.



Photo 7.15-1: Installation of Steel Gangway at Groyne G-2



Photo 7.15-2: Steel Gangway of Groyne G-2 with installed Monitoring Equipment

7.16 LABOUR AND WORKMANSHIP

As much as possible the works were executed by man-power. This holds especially for all earth works (excavation and filling) and for unloading, loading, placing operations of all construction materials like cc-blocks, boulders, granular filters, geo-textile filters and more.

It is major problem in Bangladesh to employ skilled labour gangs from outside the construction area. The local population, normally jobless except during planting and harvest periods, insists strongly or even fight for being employed.

A maximum of 1421 workers was employed in January 1995. For the earthworks of the main embankment the subcontractors of BWDB employed more than 2,500 workers in March/April 1995. Fig. 7.16 shows a graphic on the labour employment over the whole construction period (without BWDB works).



Photo 7.16-1: Workers at Earthworks

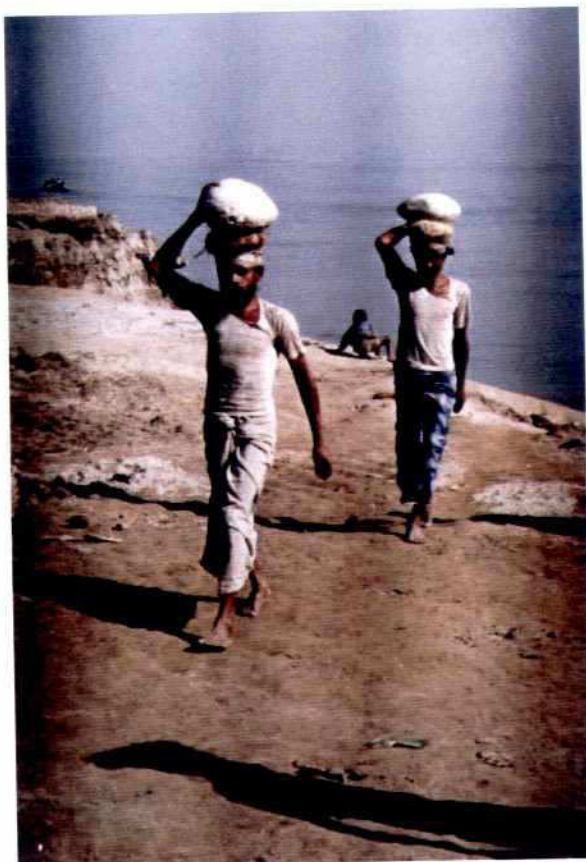


Photo 7.16-2: Workers carrying boulders



Photo 7.16-3: Workers carrying a very heavy boulders

LABOUR AT KAMARJANI

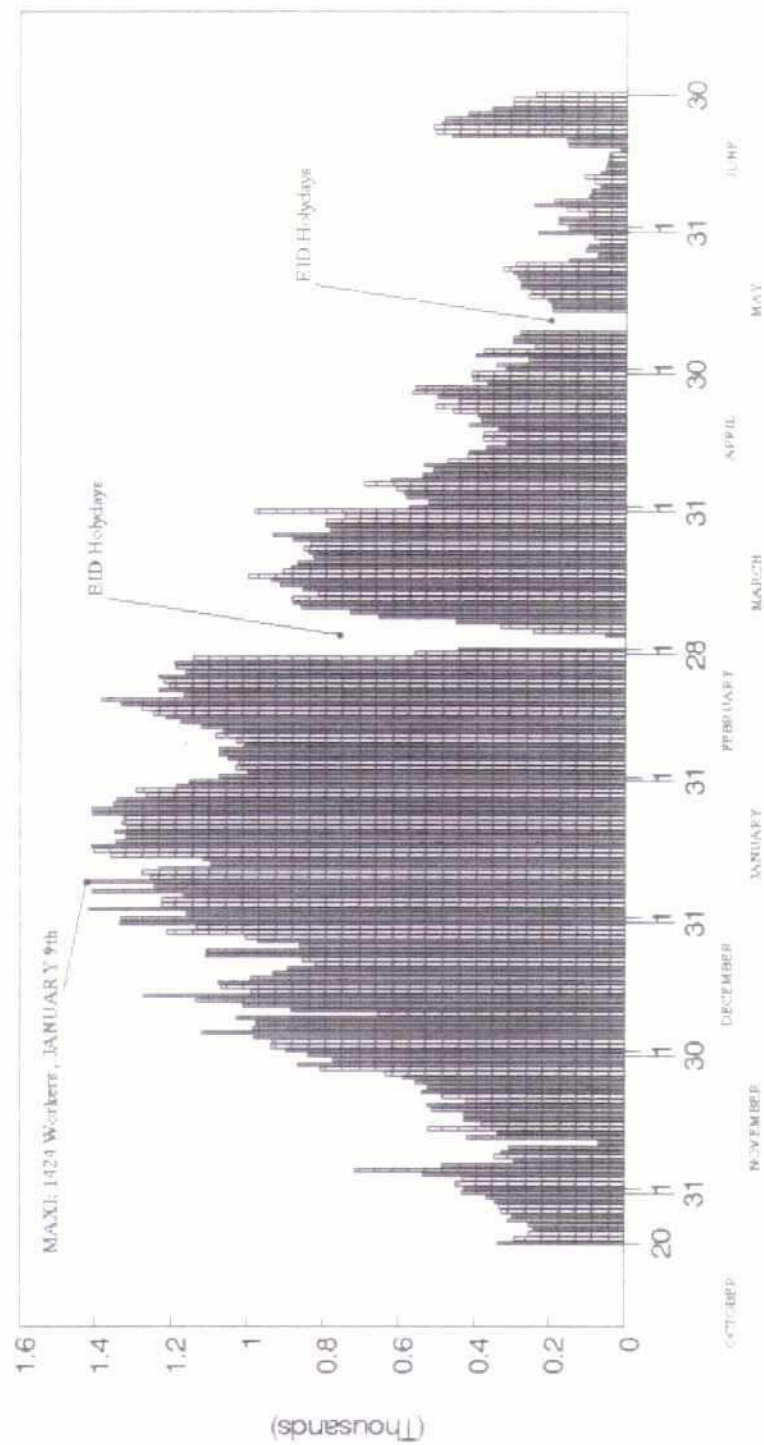


Fig. 7.16: Labour Employment – Test Site I (Kamarjani)

8 CONSULTANTS MANAGEMENT AND CONTROL OF THE WORK IMPLEMENTATION

8.1 CONSULTANT AS MAIN CONTRACTOR

The classical role of a Consultant is the planning, design and supervision of structures and construction sites. He is the representative of the employer.

In view of the character of this project, its tight dependence on strictly defined construction windows and the fact that bank protection structures of this scale have not been executed in Bangladesh before the Donors created a new concept. In this concept the Consultant is not only the "Employers Representative" but at the same time the "General Contractor" for the whole of the works of the test structures.

As General Contractor the Consultant has awarded contracts and subcontracts for procurement and works to local and international suppliers and contractors.

8.2 SPECIALIST SUPPORT

The installation of large diameter tubular steel piles required methods and skills not commonly available in Bangladesh at the time. Therefore, to ensure also transfer of technology, knowledge and experience for any similar future works it was agreed to provide specialist assistance to the local sub-contractor.

The Consultants arranged specialists on pile installation works (on-shore as well as off-shore) as well as placing of geo-textile mattresses under water, which guided and assisted the local personnel as well as the management of the local sub-contractors throughout the works.

8.3 CONSULTANTS SITE CAMP

During the initial site selection and preparation stage for Test Site I the Consultants rented a guesthouse cum office in Gaibandha, which was kept operative till October 1995. Since a decision on the final location of the test site would be made quite late and two test sites at different locations in consecutive years were planned the Consultants investigated in the possibility to procure a Container Camp for his own site office and accommodation. Procurement from outside Bangladesh was found to be too expensive, wherefore the Consultants investigated the possibility to fabricate office and accommodation containers in Bangladesh. This, in fact, proved to be a feasible option.

Incidentally the Consultants came to know, that a complete container camp of a Dutch/Bangladesh joint venture exploring gas fields near Srimongol was offered for sale. After inspection, the Consultants purchased 16 containers, which were shipped on a barge to the test site location.

The Consultants/FPCO's camp with site office and accommodation was installed on an area of 2000 m² prepared about 100 m downstream of the construction site within the scope of the works-contract.

The camp was commissioned on 05 November 1994 and remained accessible from Gaibandha by roads and trafficable embankment. Due to heavy bank erosion the camp had to be shifted to a safer place about 2 km towards inland in April 1996.

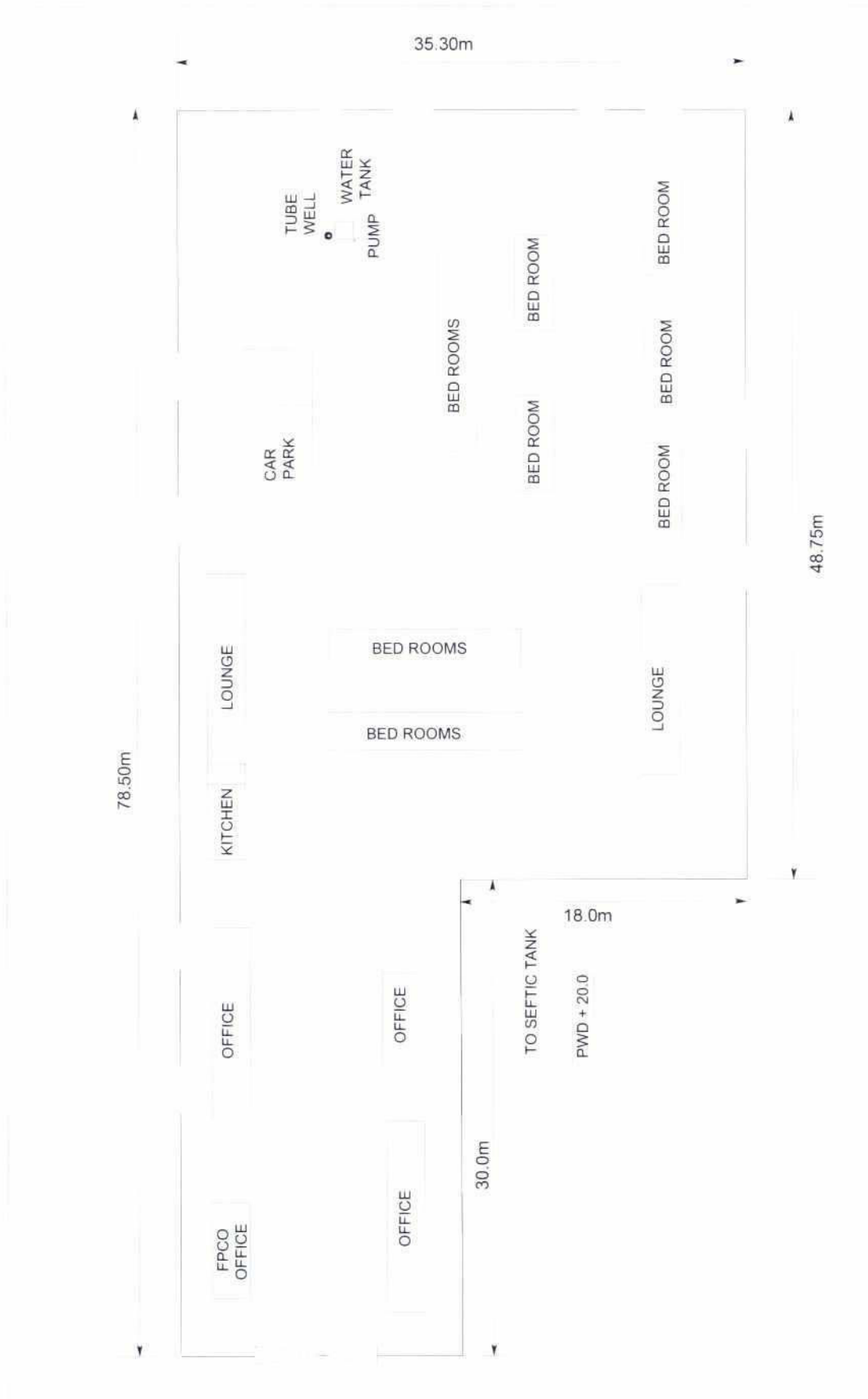


Fig. 8.3-1: General Layout Consultant/Employer's Camp – Test Site I



Photo 8.3-1: Kamarjani Container Camp

8.4 COMMUNICATION SYSTEM

Due to the remote location of Test Site I it was unfeasible to have a telephone connection by T&T network to the construction site. The Gaibandha offices of both the Consultants and their sub-contractor had telephone and fax connections, which however were mostly unreliable.

Therefore, the Consultants obtained an official license to procure, install and operate HF radio equipment for communication between the site and the Consultants and sub-contractor's offices in Dhaka. For communication between offices, sites, survey boats and Consultants guesthouse at Gaibandha, a VHF radio equipment was installed. Thus, the Consultants site staff as well as their sub-contractors could communicate also in emergency situations.

The radio link became operative only mid February 1995, after lengthy procedure for obtaining the license.



Photo 8.4-1: Radio Equipment

9 FINANCIAL SUMMARY

9.1 SUMMARY OF OVERALL COST

Details are shown in Table 9.1.

9.1.1	<u>Total cost of imported materials:</u>	DM 2,187,523	equivalent to	TK 56,875,600
9.1.2	<u>Total cost of local materials:</u>	TK 19,896,338	equivalent to	DM 762,271
9.1.3	<u>Total cost of imported (purchased) equipment:</u>	DM 2,293,071	equivalent to	TK 59,619,858
9.1.4	<u>Total cost of local equipment hire:</u>	TK 10,521,800	equivalent to	DM 409,747
9.1.5	<u>Total cost of other local procurement:</u>	TK 2,003,676	equivalent to	DM 77,421
9.1.6	<u>Total cost of construction works, cost of additional works, claims, etc.:</u>	TK 133,509,200	equivalent to	DM 4,964,034
9.1.7	<u>Total cost of consultants camp:</u>			
	a) Purchase/Procurement:	DM 175,511	equivalent to	TK 4,423,938
	b) Transport & Installation *:	TK 3,811,854 *	equivalent to	DM 145,610 *
	Total Camp:	TK 8,375,149	equivalent to	DM 321,121
9.1.8	<u>Total cost of management/specialist support:</u>	DM 542,620	equivalent to	TK 14,108,121
9.1.9	<u>Total cost of insurance premium :</u>	TK 4,777,054	equivalent to	DM 188,454
9.1.10	<u>Total cost for port and demmorage charges :</u>	TK 5,078,922	equivalent to	DM 190,710
9.1.11	<u>Total cost for VAT and Taxes born by G.O.B.:</u>			
	(a) VAT and Taxes on Imported Materials:	Tk. 56,895,000	equivalent to	DM 2,188,000
	(b) VAT and Taxes on Imported Equipment:	Tk. 15,593,000	equivalent to	DM 600,000
	(c) VAT on works (4.5% of contract value):	TK 6,007,914	equivalent to	DM 223,382

* Amount included in item 9.1.4 and 9.1.6

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JAMUNA TEST WORKS CONSULTANTS
FAP 21 TEST STRUCTURES KAMARJANI
Total Construction Costs of Test Structure

Description	Amount (D.M.)	Amount (TK.)
A. WORKS (Subcontract)		
Construction contract	4,964,034	133,509,188
B. Local Procurement & Supply		
Supply of prefabricated concrete Sheet piles and concrete prestressed spun piles	379,222	9,848,975
Hiring of equipment	409,747	10,521,792
Supply of boulders	299,054	7,692,763
Port Charges & Demurrage	190,710	5,078,922
Other	256,778	6,698,314
Insurance premium	188,454	4,777,054
Subtotal local	1,723,965	44,617,820
C. Non Local Procurement & Supply		
Tubular Steel pile Material	1,685,180	43,814,670
Rubber Tired Rollers (for Steel Pile Welding)	127,440	3,313,433
Steel Sheet Pile and Anchor Material	204,149	5,307,865
Construction Steel Material	46,021	1,196,558
Geo-Textile Filter Material	247,393	6,432,218
Piling Equipment	1,670,970	43,445,210
Winches, Ropes, Shackles, Pendants	232,734	6,051,091
Monitoring Equipment	242,263	6,298,835
Consultants Camp (purchase only)	175,511	4,563,295
Management/Specialist Support (Piling, Monitoring, Survey)	542,620	14,108,121
Other	70,916	1,843,797
Subtotal non local	5,245,197	136,375,091
Total	11,791,364	310,814,506

Table 9.1: Summary of Overall Construction Costs, Test Structure Kamarjani

9.2 ANALYSES OF CONSTRUCTION COSTS

In the following tables a breakdown of costs is given for the 6 groynes of the test structure and the improvement of the embankment constructed by BWDB.

The tables show the net construction costs for the "as built" situation including the cost for supply of material by the Consultant. General costs like site installation, Consultants' camp and equipment and material supplied by the Consultant are added (approximately pro rata of total costs or piling meter) at the end of each table.

The costs for emergency measures (material and works) after the occurrence of damage to the groynes like dumping of boulders, cc-blocks and earth-filled geo-textile and jute bags are included in the grand total of groyne G-2 and groyne G-3 in July 1995.

Analyses of Project Costs
Breakdown of total costs for groyne no. G-1

Title	Description	Works TK	Other TK	Total TK
1	Steel Pile Preparation and Installation:			
a	Procurement		12,374,840	
b	Transport to Site		964,023	
c	Preparation	162,853		
d	Installation	1,139,565		
e	Head Preparation	168,560		
	Subtotal 1	1,470,978	13,338,863	14,809,841
2	Steel Sheet Pile Cofferdam:			
2.1	Sheet Piling:			
a	Procurement		3,255,999	
b	Transport to Site		236,600	
c	Installation	848,080		
d	Head Preparation			
	Subtotal 2.1	848,080	3,492,599	4,340,679
2.2	Anchoring:			
a	Procurement		1,519,974	
b	Transport to Site		53,040	
c	Installation	93,965		
	Subtotal 2.2	93,965	1,573,014	1,666,979
2.3	Earth works and pavements:			
a	Excavation/Fill/Turfing	8,946		
b	Filter, granular	109,040		
c	Backfill	119,376		
d	CC-block pavement	224,850		
	Subtotal 2.3	462,212	-	462,212
	Subtotal 2	1,404,257	5,065,613	6,469,870
3	Revetment/ Bed Protection:			
a	Excavation/Fill/Turfing	2,062,410		
b	Procurement of geo-textile		976,352	
c	Transport to Site		31,739	
d	Filter, geo-textile	71,864		
e	Filter, granular	1,355,016		
f	Procurement of stones (boulders)		3,227,705	
g	Transport to Site		919,567	
h	Revetment by stones	362,617		
i	Bed protection by stones	682,602		
j	Falling Apron, CC-blocks	748,996		
k	Brick mattressing	569,170		
	Subtotal 3	5,852,675	5,155,363	11,008,038
	Total Groyne No. G-1, 1-3	8,727,910	23,559,839	32,287,749
4	General Items:			
a	Equipment, heavy, Procurement		14,927,510	
b	Hire local equipment		3,156,538	
c	Construction Steel, Procurement		374,397	
d	Equipment, monitoring		2,717,078	
e	Site Installation	3,641,338		
f	Camp	2,058,964		
g	Other, non specified		12,069,648	
	Subtotal 4	5,700,301	33,245,070	38,945,371
	Total 1-4	14,428,211	56,804,909	71,233,122
	Grand Total Groyne No.G-1		TK	71,233,120
			DM (rate 26)	2,739,735

Table 9.2-1: Breakdown of Costs of Groyne G-1

Analyses of Project Costs
Breakdown of total costs for groyne no. G-2

Title		Description	Works TK	Other TK	Total TK
1		Steel Pile Preparation and Installation:			
	a	Procurement		14,088,010	
	b	Transport to Site		1,055,860	
	c	Preparation	180,507		
	d	Installation	1,270,893		
	e	Head Preparation	156,041		
		Subtotal 1	1,607,441	15,143,871	16,751,312
2		Earth works:			
	a	Excavation/Fill	2,197,143		
		Subtotal 2	2,197,143		2,197,143
3		Revetment/ Bed Protection:			
	a	Excavation/Fill/Turfing			
	b	Procurement of geo-textile		121,560	
	c	Transport to Site		962	
	d	Filter, geo-textile	16,303		
	e	Filter, granular	2,272,344		
	f	Procurement of stones (boulders)		1,847,469	
	g	Transport to Site		543,838	
	h	Revetment by stones	763,683		
	I	Falling Apron, CC-blocks	5,080,779		
	j	Brick mattressing	430,990		430,990
		Subtotal 3	8,564,099	2,513,830	11,077,929
4		Monitoring Gangway & Platform:			
	a	Gangway, steel, supply & inst	372,472		
		Subtotal 4	372,472		372,472
		Total Groyne No. G-2, 1-4	12,741,155	17,657,700	30,398,855
5		General Items:			
	a	Equipment, heavy, Procurement		17,168,620	
	b	Hire local equipment		3,156,538	
	c	Construction Steel, Procurement		314,570	
	d	Equipment, monitoring		2,717,078	
	e	Site Installation	3,641,338		
	f	Camp	2,058,964		
	g	Other, non specified		12,069,648	
	h	Emergency Measures		8,500,000	
		Subtotal 5	5,700,301	43,926,354	49,626,655
		Total 1-5	18,441,456	61,584,054	80,025,510
		Grand Total Groyne No.G- 2		TK	80,025,510
					3,077,904

Table 9.2-2: Breakdown of Costs of Groyne G-2

Analyses of Project Costs
Breakdown of total costs for groyne no. G-3

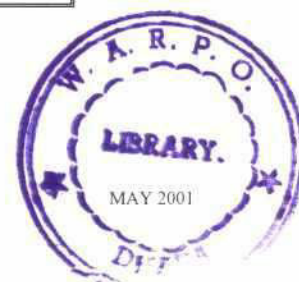
Title	Description	Works TK	Other TK	Total TK
1	Steel Pile Preparation and Installation:			
a	Procurement		16,261,273	
b	Transport to Site		1,275,052	
c	Preparation	191,840		
d	Installation	180,997		
e	Head Preparation			
	Subtotal 1	372,837	17,536,326	17,909,163
2	Earth Works:			
a	Excavation/Fill	2,266,608		
	Subtotal 2	2,266,608	-	2,266,608
3	Revetment/ Bed Protection:			
a	Excavation/Fill/Turfing	10,676		
b	Procurement of geo-textile		360,671	
c	Transport to Site		2,927	
d	Filter, geo-textile	48,926		
e	Filter, granular			
f	Procurement of stones (boulders)		1,358,773	
g	Transport to Site		402,521	
h	Revetment by stones	571,839		
i	Falling Apron, CC-blocks	8,378,298		
j	Brick mattressing	807,930		
	Subtotal 3	9,817,669	2,124,891	11,942,560
	Total Groyne No. G-3, 1-3	12,457,114	19,661,217	32,118,331
4	General Items:			
a	Equipment, heavy, Procurement		18,786,088	
b	Hire local equipment		3,156,538	
c	Construction Steel, Procurement		314,570	
d	Equipment, monitoring		2,717,078	
e	Site Installation	3,641,338		
f	Camp	2,058,964		
g	Other, non specified		12,069,648	
h	Emergency Measures		8,500,000	
	Subtotal 4	5,700,301	45,543,822	51,244,123
	Total 1-4	18,157,415	65,205,039	83,362,454
	Grand Total Groyne No.G-3		TK	83,362,454
			DM (rate 26)	3,206,248

Table 9.2-3: Breakdown of Costs of Groyne G-3

Analyses of Project Costs
Breakdown of total costs for groyne no. G-A

Title	Description	Works TK	Other TK	Total TK
1	Pile Preparation and Installation:			
1.1	Steel Piles:			
a	Procurement		661,336	
b	Transport to Site		49,479	
c	Preparation	72,222		
d	Installation	57,528		
	Subtotal 1.1	129,750	710,815	
1.2	Spun Piles:			
a	Procurement		918,000	
b	Transport to Site		220,056	
d	Installation	81,364		
e	Head Preparation	59,490		
	Subtotal 1.2	140,854	1,138,056	
1.3	Bored piles Piles:			
	Bored piles Inst.	1,105,440		
	steel tube lining	1,158,080		
	Reinforcement	1,138,764		
	Subtotal 1.3	3,402,284		
	Subtotal 1	3,672,888	1,848,871	5,521,759
2	CC-Sheet Pile Cofferdam			
2.1	Sheet Piling:			
a	Procurement		6,557,000	
b	Transport to Site		533,000	
c	Installation	517,373		
d	Head Preparation, capping beam	319,918		
	Subtotal 2.1	837,291	7,090,000	7,927,291
2.2	Anchoring:			
a	Procurement		531,589	
b	Transport to Site		10,920	
c	Installation	12,938		
	Subtotal 2.2	12,938	542,509	555,447
2.3	Earth works and pavements:			
a	Excavation/Fill/Turfing	393,278		
b	Filter, granular	673,569		
c	Backfill	168,080		
d	CC-block pavement	38,442		
	Subtotal 2.3	1,273,369	-	
	Subtotal 2	2,123,598	7,632,509	9,756,107
3	Revetment/ Bed Protection:			
a	Excavation/Fill/Turfing	64,780		
b	Filter, granular	986,622		
c	Scour protection by stones	67,680		
d	Scour protection, CC-blocks	825,913		
	Subtotal 3	1,944,995	-	1,944,995
	Total Groyne No. G-A, 1-3	7,741,481	9,481,380	17,222,861
4	General Items:			
a	Equipment, heavy, Procurement		1,344,145	
b	Hire local equipment		1,052,179	
c	Construction Steel, Procurement		125,828	
d	Equipment, monitoring		1,086,831	
e	Site Installation	1,456,535		
f	Camp	823,585		
g	Other, non specified		4,827,859	
	Subtotal 4	2,280,120	8,436,843	10,716,963
	Total 1-4	10,021,601	17,918,223	27,939,824
	Grand Total Groyne No. G-A		TK	27,939,824
			DM (rate 26)	1,074,609

Table 9.2-4: Breakdown of Costs of Groyne G-A



Analyses of Project Costs
Breakdown of total costs for groyne no. G-B/1

Title	Description	Works TK	Other TK	Total TK
1	Pile Preparation and Installation:			
1.1	Steel Piles:			
a	Procurement		440,891	
b	Transport to Site		135,200	
c	Preparation	48,148		
d	Installation	38,352		
	Subtotal 1.1	86,500	576,091	
1.2	Spun Piles:			
a	Procurement		1,224,000	
b	Transport to Site		293,408	
c	Preparation	56,400		
d	Installation	55,965		
e	Head Preparation	66,100		
	Subtotal 1.2	178,465	1,517,408	
1.3	Bored piles Piles:			
	Bored piles Inst.	1,124,240		
	steel tube lining	1,158,080		
	Reinforcement	1,138,727		
	Subtotal 1.3	3,421,047		
	Subtotal 1	3,686,012	2,093,499	5,779,511
2	Earth works and pavements:			
a	Excavation/Fill/Turfing	1,264,247		
	Subtotal 2	1,264,247	-	1,264,247
3	Revetment/ Bed Protection:			
	Excavation/Fill/Turfing			
	Procurement of geo-textile		325,832	
	Transport to Site		2,579	
	Filter, geo-textile	43,588		
	Falling apron, CC-blocks	4,852,131		
	Brick mattressing	1,077,146		
	Subtotal 3	5,972,865	328,411	6,301,276
	Total Groyne No. G-B/1, 1-3	10,923,124	2,421,910	13,345,034
4	General Items:			
a	Equipment, heavy, Procurement		896,097	
b	Equipment, monitoring		410,125	
c	Site Installation	1,456,535		
d	Camp	823,585		
e	Other, non specified		4,827,859	
	Subtotal 4	2,280,120	6,134,081	8,414,201
	Total 1-4	13,203,244	8,555,991	21,759,236
	Grand Total Groyne No. G-B/1		TK	21,759,236
			DM (rate 26)	836,894

Table 9.2-5: Breakdown of Costs of Groyne G-B/1

Analyses of Project Costs
Breakdown of total costs for groyne no. G-B/2

Title		Description	Works TK	Other TK	Total TK
1.1		Spun Piles:			
	a	Procurement		1,275,000	
	b	Transport to Site		305,634	
	d	Installation	112,365		
	e	Head Preparation	66,100		
		Subtotal 1.1	178,465	1,580,634	
1.2		Bored piles Piles:			
	a	Bored piles Inst.	1,124,240		
	b	steel tube lining	1,158,080		
	c	Reinforcement	1,138,727		
		Subtotal 1.2	3,421,047		
		Subtotal 1	3,599,512	1,580,634	5,180,146
2		Earth works and pavements:			
	a	Excavation/Fill/Turfing	111,150		
	b	Boulder mattressing	86,715		
	c	CC-block pavement	35,861		
		Subtotal 2	233,726	-	233,726
		Total Groyne No. G-B/2, 1-2	3,833,238	1,580,634	5,413,872
3		General Items:			
	e	Site Installation	728,268		
	f	Camp	411,793		
	g	Other, non specified		2,413,930	
		Subtotal 3	1,140,060	2,413,930	3,553,990
		Total 1-3	4,973,298	3,994,564	8,967,862
		Grand Total Groyne No. G-AB/2		TK	8,967,862
				DM (rate 26)	344,918

Table 9.2-6: Breakdown of Costs of Groyne G-B/2

Analyses of Project Costs
Breakdown of total costs for improvement of BWDB embankment

Title		Description	Works TK	Other TK	Total TK
1		Earth works and pavements			
	a	Excavation/Fill/Turfing	6,439,418		
		Subtotal 2	6,439,418	-	6,439,418
3		Revetment/ Bed Protection:			
	a	Procurement of geo-textile		814,580	
	b	Transport to Site		6,448	
	c	Filter, geo-textile	25,279		
	d	Filter, granular	4,401,852		
	e	Slope protection, CC-blocks	286,700		
	f	Brick mattressing	5,551,922		
	g	Turfing	124,511		
		Subtotal 3	10,265,753	821,028	11,086,781
		Total Improvement BWDB Embankment	16,705,171	821,028	17,526,199
		Grand Total Impr. BWDB Emb.		TK	17,526,199
				DM (rate 26)	674,085

Table 9.2-7: Breakdown of Costs of Improvement of BWDB Embankment

10 ADAPTATION OF KAMARJANI TEST SITE

10.1 PRELIMINARY REMARKS

During the flood season of 1995 damage occurred to the groynes G-1 and G-2 and to the main embankment in between these two groynes. For details refer to "Report on Monitoring and Adaptation at Kamarjani Test Site" September 1996.

With the results of investigations in the causes for the damage and results from additional model tests at the River Research Institute at Faridpur (ref. to Technical Report No. 5 – Additional Model Tests March 1996) the groynes have been redesigned for adaptation and repair. (For details refer to ANNEX 4). Due to the magnitude of bank erosion the main embankment had to be re-located between groyne G-1 and groyne G-A.

The general layout of the adapted test structures is shown in Drawing AD-KA-001, Attachment 2.

10.2 LAND ACQUISITION

Due to the re-location of the embankment additional land had to be acquired. The proceedings have been the same as described in Subsection 1.2.3.3.

10.3 DESIGN OF THE TEST STRUCTURE

For the adaptation design of the test structures ANNEX 4 may be consulted, which contains all design data and principles related to the permeable groyne structures and associated works.

For the purpose of this Construction and Procurement Report a selection of design and construction drawings is included under ATTACHMENT 1.

10.4 PROCUREMENT OF MATERIALS AND EQUIPMENT

10.4.1 General Information

For the adaptation works at the groynes remaining surplus material and recovered material from the first construction phase was used. Additional material was procured mainly from the same suppliers as before.

Since it was programmed to use the rented barge with its deck installations for the second test site all main key equipment was kept at Kamarjani after the completion of the works in standby position. The 400 t barge with the 150 t crawler crane and piling equipment was send back to a shipyard near Dhaka for maintenance and "safe anchoring" during high flood season. Therefore it was not necessary to procure equipment additionally.

10.4.2 Construction Material Procurement

10.4.2.1 Local Procurement

In the following only the main material supplies procured in Bangladesh are compiled, more details are presented in Table 10-P1.

(a) **Boulders for rip-rap** (imported from Bhutan) in various gradation ranges,

Supplied quantity:	9,000	m ³
Procurement cost (incl. VAT)	Taka	16,525,238
Equivalent to	DM	611,083

(b) **Geo-Textile Filter Material** (imported from France)

Supplied quantity:	22,800	m ²
Procurement costs (incl. VAT)	Taka	3,306,000
Equivalent to	DM	120,437

10.4.2.2 Imported Materials

For adaptation works at Kamarjani only Steel Pile material was imported into Bangladesh, more details are presented in Table 10-P2.

(a) **Country of Origin France**

- **Tubular steel piles** (dia. 711x14.2 mm only)

Supplied quantity:	655	t
Procurement cost (c.i.f. Chittagong)	DM	861,871 (without VAT)
Equivalent to	Taka	22,408,658 (without VAT)

10.4.3 Cost of Construction Equipment

10.4.3.1 Local Procurement

No local equipment procurement for adaptation works at Kamarjani.

In addition local cost incurred in connection with equipment rental, port charges, forwarding and demurrage in connection with imports for the project, all of which are compiled in Table 10-P3.

10.4.3.2 Import of Construction and Ancillary Equipment

Only spare parts for piling and monitoring equipment have been procured for adaptation works at Kamarjani.

(a) **Spare Parts for Pile driving equipment**

(cif Dhaka)	DM	16,192
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(b) **Spare Parts & Repair for monitoring Equipment**

(cif Dhaka)	DM	17,374
Total Equipment Import	DM	33,566

More details may be taken from Table 10-P4 (Procurement of Equipment Outside Bangladesh) and Table 10-P5 (Other Procurement Outside Bangladesh).

Test Site 1, Kamarjani-Adaptation, 1995/1996

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	TOTAL	TOTAL
1	2	3	4	5	6	7	8	9
1	Stones for Revetments / Bed Protections							
	(α = D50)							
3a	Range F, ø35cm, Truck Transport	Exchange	1996	3,213	m³	1,900	6,104,776	225,747
3b	Range F, ø35cm, Rail/Boat Transport	International Ltd., Dhaka		5,787	m³	1,801	10,420,463	385,335
								0
				9,000	m³		16,525,239	611,082
2	Geo-Jute Material							
2a	Geo-Jute filter material 1.22Lbs	Adamjee Jute Mills	May-96	5000	m²	11	55,121	2,053
		Dhaka						
3	Geo-Textile Filter Material							
3a	geo-textile fabri, BIDIM A-29	DIRD Private Ltd	Jan-96	22800	m²	145	3,306,000	120,437
		Dhaka						
TOTAL MATERIAL PROCUREMENT IN BANGLADESH							19,886,360	733,572

Table 10-P1: Procurement of Construction Material in Bangladesh

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Procurement

Test Site I, Kamarjani-Adaptation, 1995/1996

Procurement of Construction Material outside Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM
1	2	3	4	5	6	7	8	9	10
I	Tubular Steel Pile Material								
1a	Spiral Welded Pipe 711x14.2 mm	Starval	Jan-96	2,640	m	7,117	273.72	18,788,141	722,621
1b	Transport cif Chittagong	France	Mar-96	644,160	kg	6	0.22	3,620,517	139,251
								22,408,658	861,871
								22,408,658	861,871

Table 10-P2: Procurement of Construction Material outside Bangladesh

Procurement

Test Site I, Kamarjani-Adaptation, 1995/1996

Other Procurement in Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	TOTAL Taka	TOTAL DM
1	2	3	4	5	6	7	8	9
1	Import, Port & Demurrage							
1a	Charges	Forwarding Agents Chittagong, Dhaka	Mar-96 Jun-96				2,014,491	74,174
2	Hire of Equipment						3,020,000	110,936
2a	Hire of crawler crane and barge	Bengal Electric Dhaka	Mar-96 Dec-96				107,765,941	3,990,928
3	Construction Works							
3a	Construction Contract	The Engineers & Corolla	Sep-94 Mar-96					
4	Other							
4a	Other Procurements and supplies, repairs, C&F charges, licensee fees for radio, spare parts and maintenance mainly for monitoring operation	different in Bangladesh	Sep-94 to Nov-99				3,644,185	136,218
TOTAL OTHER PROCUREMENT IN BANGLADESH							116,444,617	4,312,257

Table 10-P3: Other Procurement in Bangladesh

Procurement

Test Site 1, Kamarjani-Adaptation, 1995/1996

Procurement of Equipment outside Bangladesh

Serial No.	Description	Supplier	Delivery Period	Quantity	Unit	Total per Unit Taka	Total per Unit DM	TOTAL Taka	TOTAL DM
1	2	3	4	5	6	7	8	9	10
1	Piling Equipment								
1a	Spare parts for:								
	PTC Power Unit MH 600	PTC	Nov-95	div.				420,993	16,192
	Hydraulic Hammer MHP 10-15	France	Jun-96						
	Vibrator 50H4								
2	Survey Equipment for Monitoring								
2a	Repair of Equipment	OSAE	Nov-96	div.		-	-	143,210	5,508
		Germany				-	-		
2b	Electronic Current Meter	ADM	May-96	div.		-	-	198,900	7,650
	Repair of Equipment	Germany	Nov-95						
2c	Spare Parts for Monitoring Boat	Wicking	Sep-97	div.		-	-	8,987	346
	Spare Parts for Monitoring Boat	PEGEDO	May-96	div.		-	-	77,340	2,975
		Germany	Sep-97						
2d	Photo Equipment (Cameras)	Knittel	Sep-96	1	No	-	-	12,016	462
		Germany	Sep-97	1	No	-	-	11,282	434
								872,727	33,566

Table 10-P4: Procurement of Equipment outside Bangladesh

Test Site 1, Kamarjani-Adaptation, 1995/1996

Other Procurement outside Bangladesh

Table 10-P5: Other Procurement outside Bangladesh

10.5 TENDER PROCEDURES – CIVIL ENGINEERING WORKS

10.5.1 Immediate Measures

The Adaptation Works at Kamarjani Test Site were not newly tendered but ordered under variation orders to the Sub-contractor of Kamarjani Test Site No. 1.

Immediately at the end of the monsoon season of 1995 and after the occurrence of damage to the groynes a contract and a bill of quantities for “Adaptation of Test Structures – Immediate Measures” was prepared by the Consulting and priced by the Contractor of Test Site 1 under the same conditions of contract and specifications as the main contract for Kamarjani Test Site. Works under this contract started on October 1, 1995. The contract had a value of TK 6,562,688 and dealt mainly with:

- re-mobilisation (partly);
- salvage and recovery of material at groyne G-2 (steel piles and gangway), and
- extension and re-driving of piles at groyne G-1; G-3 and G-A.

10.5.2 Adaptation Works

After redesign and preparation of a bill of Quantities for adaptation and rehabilitation works, and after discussions on technical and financial matters the adaptation works were ordered in form of a variation order to “The Engineers Ltd. and Corolla Corporation BD Ltd. on December 12, 1995 . The contract was closed under the same condition of the Contract as it was for the first test site signed on September 07, 1994.

The contract based on the initial unit rates of the contract as per variation order no. 4 issued on 12.02.1996 and extended in December 1996 had a value of:

TK 114,144,138, equivalent to DM 4,390,000

The works under this contract could not be executed according to schedule due to unforeseeable events. During the month of March and April 1996 the political situation in Bangladesh with continuous strikes (hartal) caused the late arrival of new steel pile material. So the works were finally executed in 3 phases:

1. Immediate Measures :November 1995 to February 1996
2. Piling of steel piles, installation of in-situ bored piles and other Adaptation works : May and June 1996
3. Remaining steel piling works at groyne G-2: October and November 1996

10.6 EXECUTION OF WORKS

10.6.1 Description of Works

Adaptation Works started with the salvage of material at the damaged groynes G-2. In particular:

- Salvage of three sections of the monitoring gangway of G-2, and
- Salvage of 12 piles diameter 711 mm from G-2, 2 piles 19.5 m long, 10 piles 26 m long. (some piles had been washed away by the river and could not be retraced).

Groyne G-1:

(1) Adaptation Works On-shore:

- Reinforcement of revetments and bed protections at the downstream part of the groyne, using stone material (rip rap) and cc-blocks.

(2) Adaptation Works Off-shore:

- Extension of some existing piles dia. 711.2 mm by 6.5 m and 12.5 m and re-driving.

Groyne G-2 (modified):

(1) Adaptation Works On-shore:

- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths 24 m;
- Construction of in situ bored piles of dia. 700 mm, lengths 24 m,
- Construction of revetments and toe protections at the new groyne head, using stone material (rip rap) and cc blocks and geo-textile filter mat;
- Brick mattresses on geo-textile filter mat as slope protection, connecting groyne head with the original (upstream) and new re-located (downstream) embankment;
- Laying of concrete blocks and boulders for falling apron at the groyne head.

(2) Adaptation Works Off-shore:

- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 32 m;
- Laying of concrete blocks and boulders for launching and falling apron at the groyne head;
- Installation of steel gangway on top of the installed piles, using partly the recovered sections and partly new constructed steel sections.



**Photo 10.6-1: Adaptation Works at Groyne G-2;
Construction of In-Situ Bored Piles**



**Photo 10.6-2: Adaptation Works at Groyne G-2;
Construction of In-Situ Bored Piles**

Groyne G-3 (modified):

(1) Adaptation Works On-shore:

- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 28 m;
- Brick mattresses on geo-textile filter mat as slope protection, connecting the groyne head with the new re-located embankment (upstream and downstream);
- Construction of revetments and toe protections at the new groyne head, using stone material (rip rap) and recovered cc blocks and geo-textile filter mat;
- Laying of launching aprons by concrete blocks.

(2) Adaptation Works Off-shore:

- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths 32 m;
- Extension by 12.5 m and re-driving of existing piles;
- Laying of concrete blocks for launching and falling apron as slope protection for the head of the groyne;
- Installation of pile head bracing on top of the new installed and re-driven piles.



**Photo 10.6-3: Adaptation Works at Groyne G-2;
Construction of In-Situ Bored Piles**



**Photo 10.6-4: Adaptation Works at Groyne G-2;
Construction of In-Situ Bored Piles
Detail: Modified Spacing of the piles; no Impermeable Part**

Groyne G-A:**(1) Adaptation Works On-shore:**

- Supplementary slope and scour protection by boulders and cc-blocks on the head and downstream of the cofferdam.

(2) Adaptation Works Off-shore:

- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths 32 m in a supplementary row parallel to the existing in situ bored piles;
- Supplementary slope and scour protection by boulders and cc-blocks on the head and downstream of the cofferdam.



Photo 10.6-5:
Groyne G-A; General
view after Adaptation



Photo 10.6-6: Groyne G-A; General view after
Adaptation

Groyne G-A/2 (additional new groyne)**(1) Main Works On-shore:**

- Excavation for laying of bed protections, falling aprons and revetments;
- Extension of embankment slope;
- Construction of launching apron, using cc-blocks, laid on geo-textile filter (supplied by the Consultants);
- Construction of slope revetments by brick mattresses, laid on geo-textile filter;
- Construction of reinforced in-situ concrete piles, dia. 914 mm (36"), lengths up to 32 m, including permanent steel tube lining;
- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 36 m.

(2) Main Works Off-shore:

- Installation of tubular steel piles, dia. 711.2 x 14.2 mm, lengths up to 38 m.



**Photo 10.6-7:
Additional Groyne
G-A/2**



Photo 10.6-8: Additional Groyne G-A/2

Re-located Embankment

Main Works:

- Earth works for re-construction of the main embankment using soil from the old embankment and from new borrow pits.
- Filling of old borrow pits in the alignment of the new embankment.

10.7 IMPLEMENTATION PLAN

The main sub-contract for adaptation and associated works was awarded on 12.12.1995 to the same joint venture companies awarded with the contract for the works for Test Site I in 1994, refer to Subsection 10.5.2.

After approval of the adaptation design the physical works started in January 1996 with the site re-installation.

Prior to the main adaptation works, works as "immediate measures" after the occurrence of damage to the groynes had started on 01.10.1995.

The adaptation works under this contract could not be executed/completed according to schedule due to unforeseeable events. During the month of March and April 1996 the political situation in Bangladesh with continuous strikes (hartal) caused work interruptions and the late arrival of new steel pile material. The earth works for the relocated embankment and the construction of bored piles were less hampered by the strikes and continued in March and April.

Due to the above mentioned reasons the construction time was extended to the dry season of 1996/1997. A second extension was ordered in November 1997 for the completion of the works at groyne G-2. Finally the works were executed in 3 phases:

Phase I:

- | | |
|-----------|---|
| Period 1: | Immediate Measures: November 1995 to February 1996 |
| Period 2: | Piling of steel piles, installation of in-situ bored piles and other Adaptation works:
May and June 1996 |
| Period 3: | Remaining steel piling works at groyne G-2: October to December 1996 |

Phase II:

General maintenance, cc-block and boulder dumping at various locations: January to October 1997.

Phase III:

Final works at groyne G-2, bored piles and re-installation of the monitoring gangway, general maintenance: November 1997 to January 1998.

General maintenance of the structure continued until October 1999.

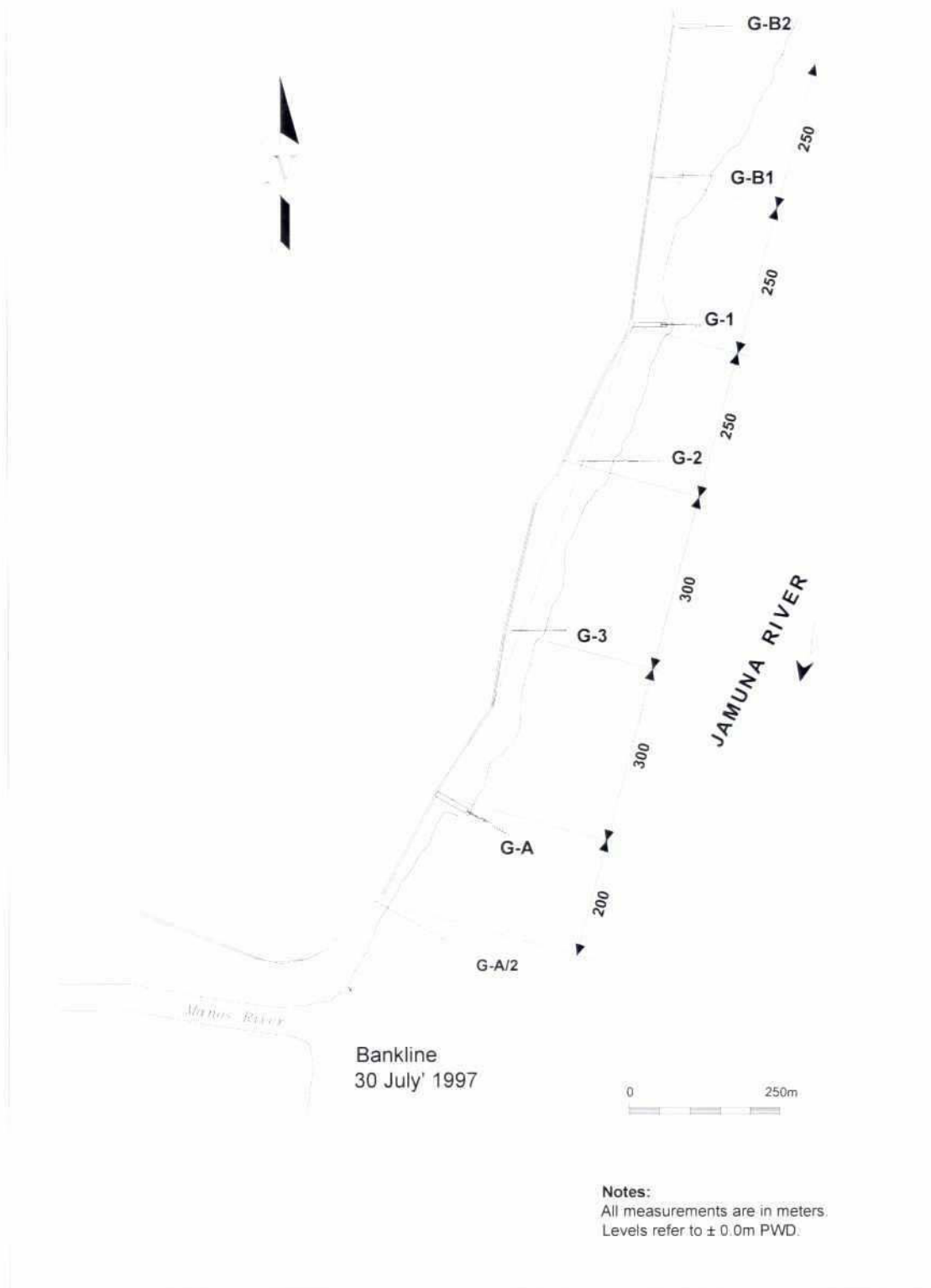


Fig. 10.7-1: General Layout of Groyne Field – Adaptation of Test Site I

10.8 HIRE OF MAIN EQUIPMENT

In May 1994 the Consultants under their procurement program concluded a contract for hiring of a 400 ton flat top barge and a 150-ton MANITOWOK crawler crane for the first test site. The equipment hire contract included the option of prolongation for the second test site. After completion of Test Site I the barge with the equipment was transported back to the yard of the renting company for maintenance and repair and "safe anchoring" during the monsoon season.

The barge and the equipment were called back to the Kamarjani site for the adaptation works and arrived at site on February 06, 1996.

10.9 SITE INSTALLATION

The site installation layout plans are shown in Fig. 10.9-1 and Fig. 10.9-2.

The sub-contractor's general site installations comprised offices, material testing laboratory, workshops, store rooms, living quarters. Due to limited space at the location of the site installation for Test Site I, a new site installation was arranged about 500 meters downstream of the construction site, close to the bank line.

Due to heavy bank erosion during the 1996 flood this site installation had to be shifted again towards the hinterland.

The welding yard for the assembly of steel piles was this time located on the flood plain between Groyne G-1 and Groyne G-B/1 because at other places between the groynes the remaining flood plain width was not sufficient.

The concrete yard for the prefabrication of cc-blocks was located on the flood plain downstream at the same location as contractors site installation.

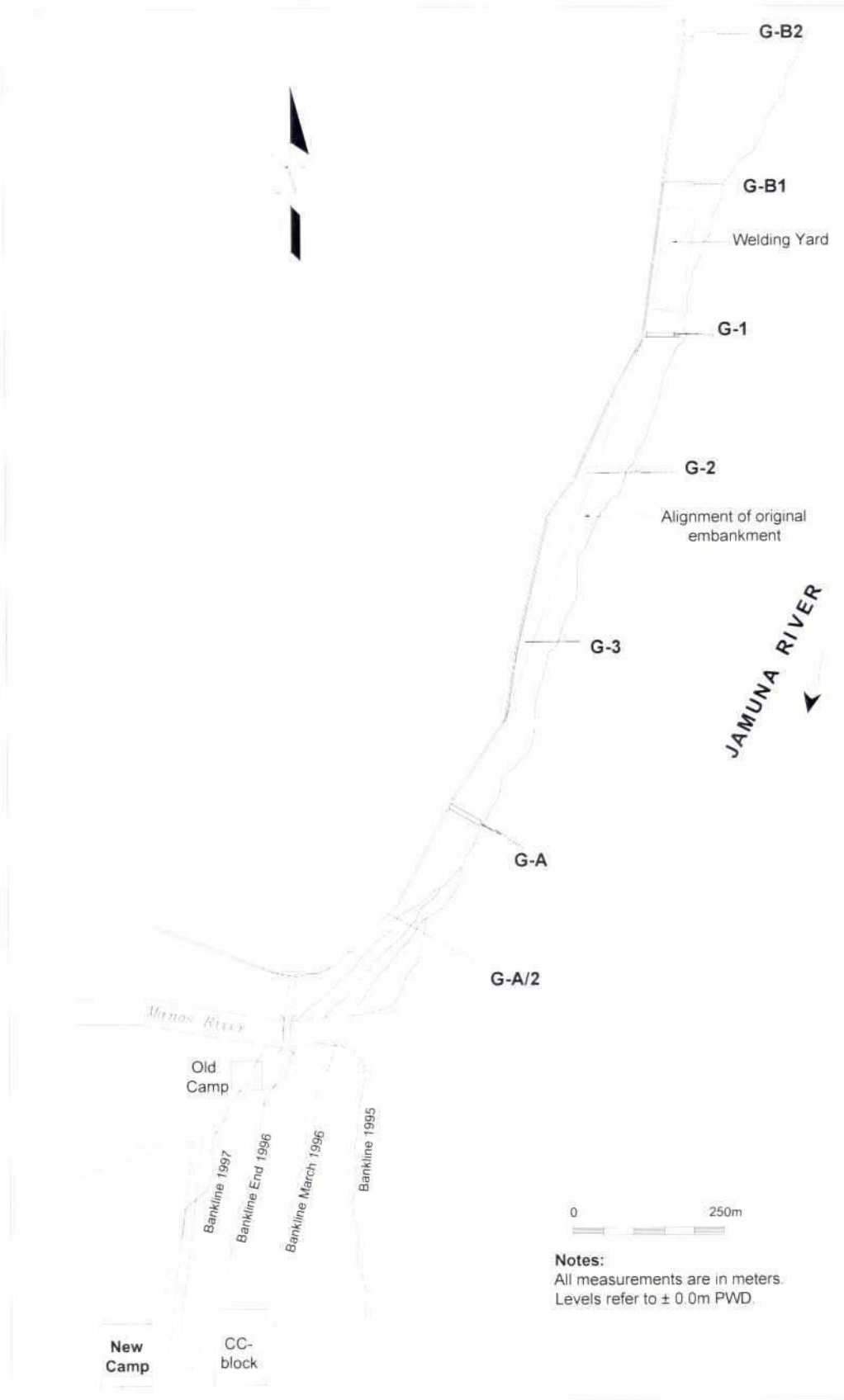


Fig. 10.9-1: General Layout of Site Installation – Adaptation of Test Site I

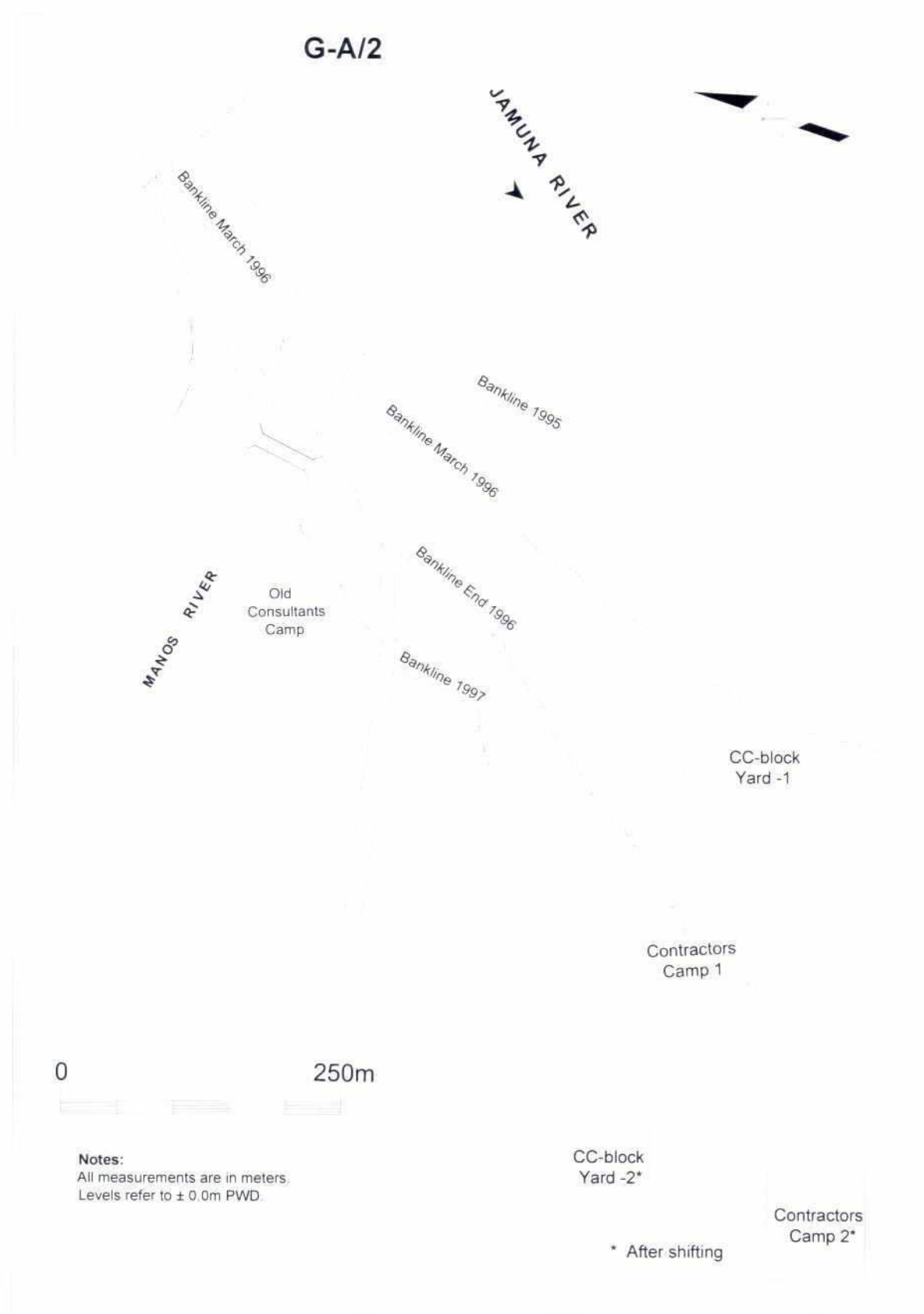


Fig. 10.9-2: General Layout of Contractors Camp – Test Site I

10.10 EARTH WORKS

10.10.1 General

The earth works for the adaptation of the groynes i.e. relocation of the main embankment and removal of remaining of the impermeable parts of Groyne G-2 and G-3 have been executed under the adaptation contract. Excavation and filling works in the dry were executed as before by hand (Ref. to Subsection 7.5-1).

10.10.2 Pond Filling

Ponds and old borrow pits laying in the alignment of the relocated embankment had to be filled. Before the filling the ponds were pumped dry and the mud was removed manually in buckets. Earth for filling was taken from the remaining parts of the old embankment and soil from new borrow pits. All filling work was carried out in layers of about 30 cm with compaction.

10.10.3 Construction of Re-located Embankment

For the construction of the re-located embankment soil was taken from the remaining parts of the old embankment and from new borrow pits.

The earth dam was constructed by suitable soil filling in compacted layers of about 30 cm thickness and according to the respective working drawings. The degree of compaction was verified for every layer through in-situ tests and laboratory tests. Construction period was January to May 1996.

The volume of the relocated embankment is 51,000 m³. For extension of the toe of the embankment at various location another 14,000 m³ were used.

10.11 CC-BLOCK PRODUCTION

The total cc-block production was approximately 6,110 m³. Together with the recovered blocks, about 6,360 m³ were dumped, according to 187,000 Nos. as follows:

- 10,000 nos. of 50x50x50 cm
- 87,500 nos. of 35x35x35 cm
- 21,500 nos. of 30x30x30 cm
- 68,000 nos. of 25x25x25 cm

These cc-blocks were mainly used for the construction of falling and launching aprons at the new groynes, reinforcement of the protection of the undamaged cofferdam heads and toe protection of the new embankment.

10.12 INSTALLATION OF REVETMENT / TOE PROTECTIONS / LAUNCHING AND FALLING APRONS

(a) Groyne G-1:

Reinforcement of slope protection downstream by 30 cm cc-blocks (301 m³).



(b) Groyne G-2:

Only a toe protection was built at the new embankment upstream and downstream of the groyne with 35 cm cc-blocks (924 m³).

(c) Groyne G-3:

Only a toe protection was built at the new embankment of the groyne upstream with 35 cm and 25 cm and downstream with 25 cm cc-blocks (924 m³).

(d) Groyne G-A:

The scour protection/falling apron around the sheet pile cofferdam was reinforced with 35 cm and 30 cm cc-blocks (2,100 m³) and with rip-rap/boulders (D50 = 35 cm, 4,465 m³) around the head of the cofferdam.

(e) Groyne G-B/1:

No adaptation works

(f) Groyne G-B/2:

No adaptation works

(g) Groyne G-A/2 (additional new groyne downstream of G-A):

A strong launching apron was built onshore upstream and downstream of the groyne in an excavated pit from 35 cm (1010 m³) and 25 cm (235 m³) cc-blocks. After filling the pit with the cc-blocks the area was recovered with soil in order to re-establish a smooth flood plain.

10.13 STEEL PILE ASSEMBLY, WELDING AND TESTING**10.13.1 General Information**

Same procedures and same welders were employed for preparing, assembling, welding and testing of the recovered and new procured steel piles as described for the initial construction works.

10.13.2 On-Shore and Off-Shore Piling**(a) Groyne G-1:**

No damage occurred to groyne G-1, all piles remained in the original position. The optional extension of piles nos. 9, 11, by 6.5 m and piles nos. 13, 15, 17 by 12.5 m was not executed.

(b) Groyne G-2:

Only piles no. 19 to 31 including the monitoring gangway on top remained in the original position after the damage of 1995 flood season. All other piles had been effected either being washed away completely or hanging loose on the twisted gangway or being bend by the collapsing gangway. Out of 18 piles dia 711 mm 10 piles and 26 m of the gangway could be recovered. Some of the piles could be recovered from the riverbed, some were extracted, others and gangway parts were recovered by towing to the barge or on shore. Salvaged piles and gangway parts were re-used after rectification (the piles for extension and re-driving at groynes G-A and G-3).

The adaptation piling works consisted of a complete re-construction of the damaged part by installation of new steel piles. The monitoring gangway was re-constructed from recovered parts and newly constructed parts.

The start of the piling works was scheduled for February 1996 but could not be executed before the monsoon season due to late arrival of the new pile material in April/May 1996. The piles were finally installed in October/November 1996 after the monsoon season.

New piles no. A8 to A16 of 32 m length were piled onshore up to level – 9.10 m in two pieces and two steps guided in a steel frame from H-beams. The piles could not be driven in full length due to the too short length of the leader of the onshore piling equipment.

New piles no. A17 to A42 of 32 m length were piled offshore in full length but in various spacing up to level – 9.10 m.

(c) Groyne G-3:

All original piles no. 2 to 35 remained in the position after the 1995 flood season only the impermeable earth dam part of the groyne was partly damaged.

The adaptation piling works consisted of a complete replacement of the impermeable part of the original groyne by steel piles. New piles nos. A1 to A13 of 24 m length, A14 to A22 of 28 m length and A23 to A26 of 32 m length all dia. 711 mm were piled onshore in two steps as described above. A working platform for the onshore piling rig was earth-filled up to level 19.5 m up to pile A26. New piles A27 to A44 all dia. 711 mm and of 32 m length were piled offshore in full length. These works were executed in May 1996 after arrival of the new steel pipes.

Original piles no. 3, 5, 7, 9, 11, 13, 15, 17 were extended by 12.5 m and re-driven offshore down to level – 9.20 m using remaining and recovered steel pile material already in February 1996.

New pile head bracing connected all new and old piles.

(d) Groyne G-A:

All original piles no. 2 to 35 remained in the position after the 1995 flood season except steel pile no. 24, which lost his vertical position. However, it was observed that the remaining embedded length of the bored piles was not more sufficient, consequently bored piles nos. 14 and 18 to 21 collapsed before and during the flood season of 1996.

Groyne G-A was adapted and reinforced in February 1996 by additional 4 steel piles, dia. 711 mm, length of 32 m in a parallel row to the existing in-situ bored piles nos. 18 to 21 (scheduled piles nos. 13 to 17 could not be installed due to late arrival of new pile material. The existing steel piles no. 24 were extracted and re-driven to vertical position, then nos. 22 to 24 were extended by 4.5 m and re-driven to level – 12.80 m. The groyne received 3 additional piles at the groyne head nos. 25 of 31.5 m length and nos. 26 and 27 of 32 m length, all dia. 1016 mm. Steel piles were surplus material or recovered material from the first site.

Later in May 1996 after arrival of the pile material the remaining 5 additional steel piles nos. 13 to 17 were installed already under difficult conditions. Due to raising water level and flow velocity it became difficult to keep the barge in a stable position. All piles were driven offshore. Pile head bracing connected all new and old steel piles.

(e) Groyne G-A/2:

At the new additional groyne G-A/2 downstream of G-A 14 nos. new steel piles dia. 711 mm were installed offshore in June 1996. Piles no. 21 to 28 are of 32 m length, no. 29 to 33 of 36 m and no. 37 of 38 m length. (Nos. 1 to 20 are bored piles).

10.14 RE-DRIVING OF PRE-CAST CONCRETE PILES

In order to get pre-cast concrete spun piles in to a deeper position re-driving of the piles was tested but failed due to damage at the pile head. Several tests with different kind of piling cushions remained without success.

10.15 BORED CONCRETE PILE INSTALLATION

Under the adaptation works in-situ bored piles were installed at the new additional groyne G-A/2 downstream of groyne G-A and at groyne G-2. The same equipment and method was used as before.

In March and April 1996 groyne G-A/2 received 20 in situ-bored piles dia. 914 mm. Piles nos. 1 to 14 of 26 m length, piles nos. 15 to 18 of 30 m and piles 19 and 20 of 34 m length.

At groyne G-2 12 nos. bored piles of dia. 700 mm and 24 m length were installed in December 1997 and January 1998.

10.16 MANUFACTURE OF STEEL GANGWAYS

The monitoring gangway at groyne no. G-2 was re-constructed on top of the new piles using recovered and repaired elements from the damaged part and a new steel construction with a slightly modified design in order to save steel quantity.

10.17 CONSULTANTS MANAGEMENT AND CONTROL OF THE ADAPTATION WORKS IMPLEMENTATION

10.17.1 Specialist Support

To ensure again good quality of work and further transfer of technology, knowledge and experience the same specialists as for the first test site gave assistance to the local sub-contractor for the adaptation works during phase 1 (ref. to 10.5.2). For the two following phases no assistance was required anymore since the contractor and his staff had gained enough knowledge to do the work without help from outside, but under the direction of the Consultant.

10.17.2 Consultants Site Camp

Due to heavy erosion downstream of the groyne field in 1995/1996 the camp had to be shifted to a safer place towards inland in April 1996. The new location and the new layout of the camp after shifting are shown in Fig. 10.17-1 and Fig. 10.17-2.

In August 1999 the camp was closed. Some containers were shifted to the second and third test site at Bahadurabad and Ghutail some were handed over for use as hospital and some were left to the landlord in compensation for campsite rehabilitation.

10.17.3 Communication System

The HF radio equipment for communication between the site and the Consultants and sub-contractor's offices in Dhaka and the VHF radio equipment for communication between the office, site and survey boats remained operative for the adaptation works.

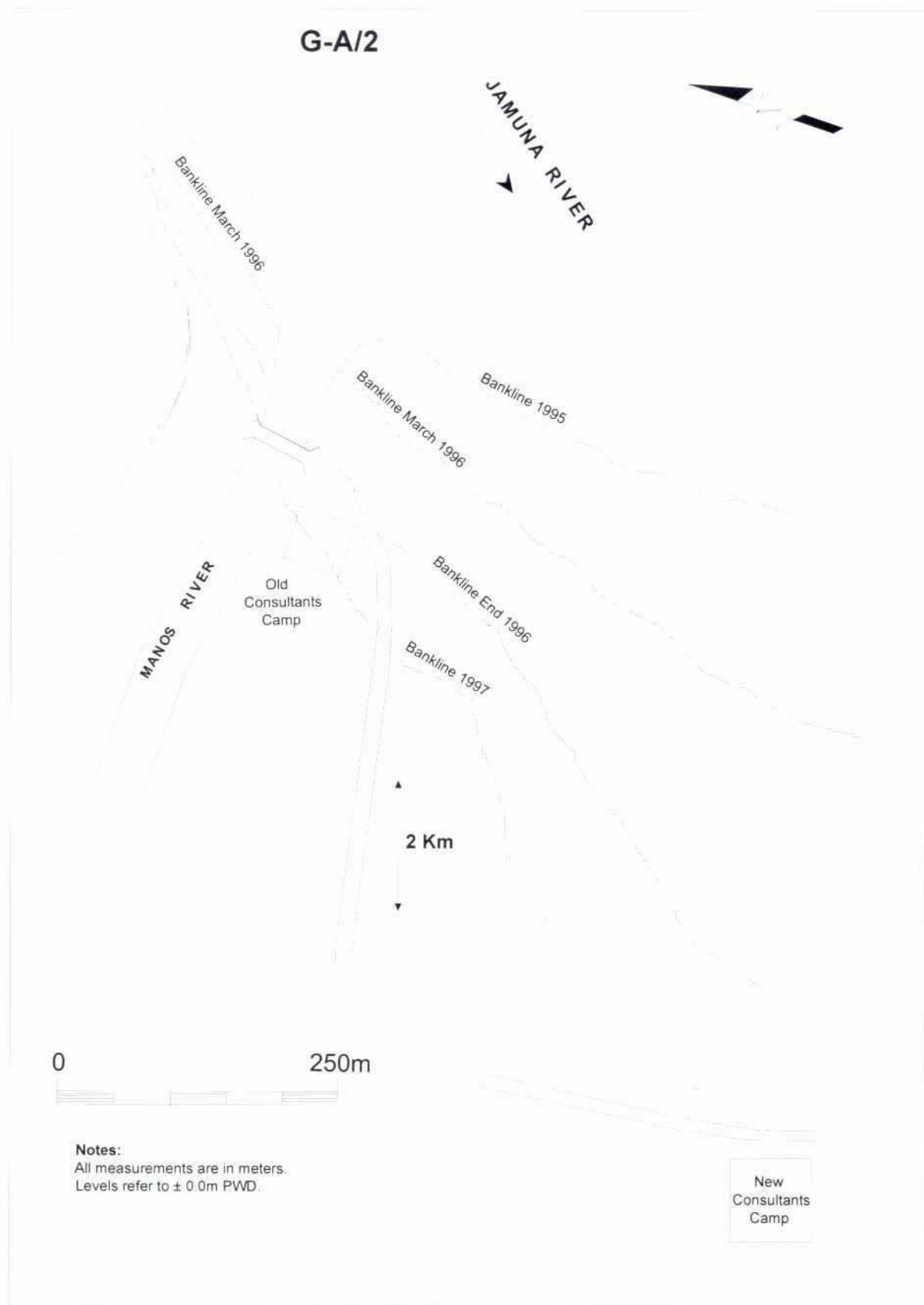
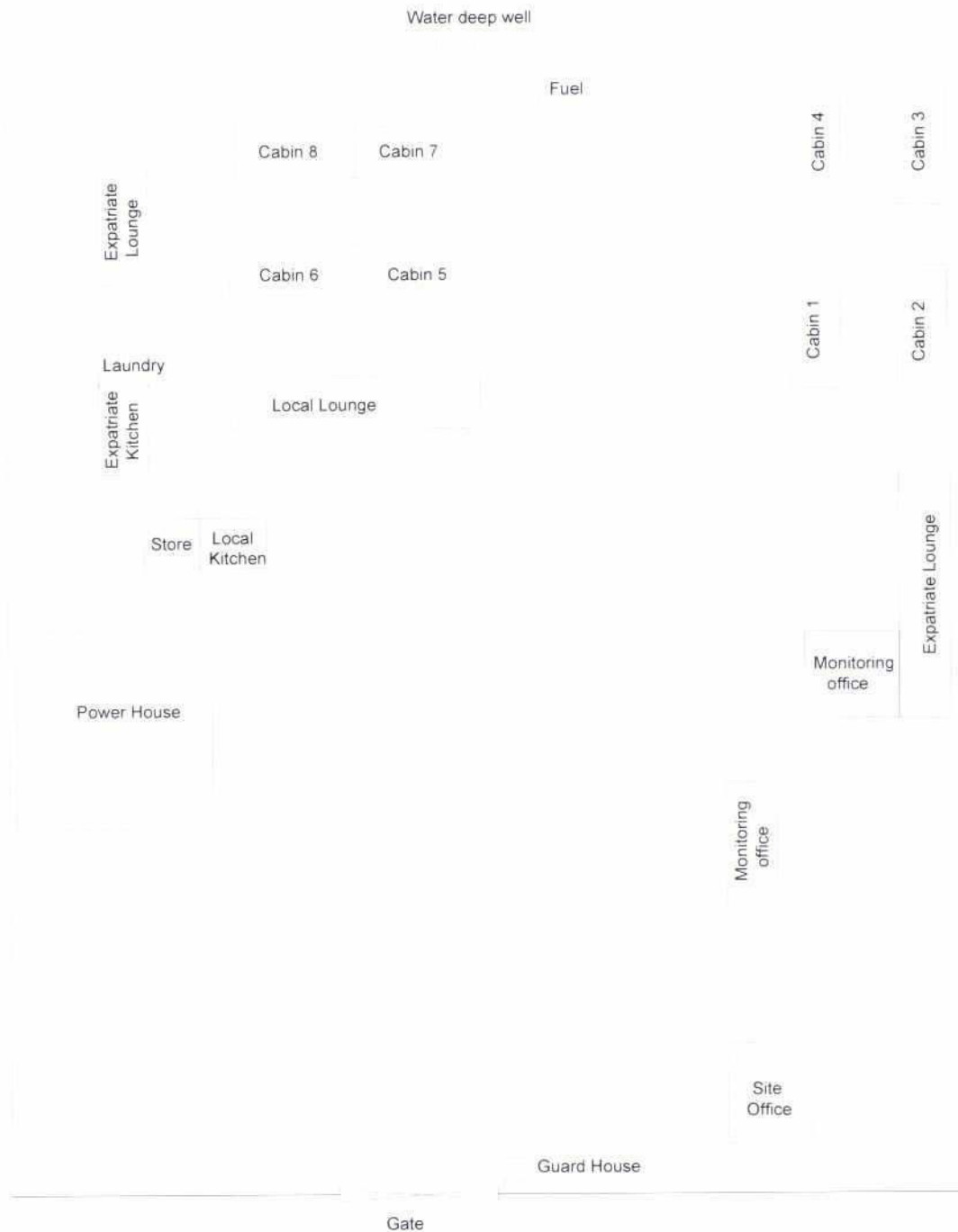


Fig. 10.17 -1: Location of Consultant/Employer's Camp before and after shifting

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Notes:
All measurements are in meters.
Levels refer to $\pm 0.0\text{m}$ PWD.

Fig. 10.17-2: General Layout Consultant/Employer's new Camp

10.18 FINANCIAL SUMMARY OF ADAPTATION WORKS

10.18.1 Summary of Overall Adaptation Cost

Details are shown in Table 10-18.1.

- Total cost of imported materials:
DM 861,871 equivalent to TK 22,408,658
- Total cost of local materials:
TK 19,886,360 equivalent to DM 733,572
- Total cost of imported (purchased) equipment:
DM 33,566 equivalent to TK 872,727
- Total cost of local equipment hire:
TK 3,020,000 equivalent to DM 110,936
- Total cost of construction works, cost of additional works, claims, etc.:
TK 107,765,941 equivalent to DM 3,990,928
- Total cost of specialists:
DM 62,078 equivalent to TK 1,614,028

Description	Amount DM	Amount Tk.
A. WORKS (Subcontract)		
Construction contract (including claims for work interruption, camp shifting and 3 years maintenance of camp and structures)	3,990,928	107,765,941
B. Local Procurement & Supply (by Employer):		
Hiring of equipment	110,936	3,020,000
Supply of boulders	611,083	16,525,238
Geo-Textile Filter Material (ordered in Bangladesh, supply from outside)	122,490	3,361,121
Port Charges & Demurrage	74,174	2,014,491
Other	136,218	3,644,185
Subtotal local	1,054,901	28,565,035
C. Non Local Procurement & Supply (by Employer)		
Tubular Steel Pile Material	861,871	22,408,648
Management/Specialist Support (accounted under Bahadurabad Test Site II)		
Other	166,237	4,322,169
Subtotal non local	1,028,108	26,730,817
Total	6,073,937	163,061,793

Table 10.18-1: Total costs of Adaptation of Test Structure

10.18.2 Analyses of Project Cost

In the following Tables 10.18.2 to 10.18-9 a breakdown of costs is given for the adaptation of 6 groynes of the test structure and the improvement of the embankment constructed by BWDB.

The tables show the net construction costs for the “as built” situation including the supply of material by the employer. General costs like site installation, consultants camp and others are shown in a separate table and are not distributed pro rata of total costs to single groynes or items. The reason is

that the general costs are extremely high (~40%) in comparison with the construction costs due to the several phases and long duration of the adaptation works. Distributing these costs would give a wrong picture of "construction costs under normal conditions", however, part of the general costs belong to the real construction costs.

The costs for emergency measures (material and works) after the occurrence of damage to the groynes like dumping of boulders, cc-blocks and earth-filled geo-textile and jute bags are mainly included in the grand total of groyne G-2 and groyne G-3, but are partly accounted too under day work and in so far figure under "General".

Analyses of Project Costs
Breakdown of total costs for adaptation of groyne no. G-1

Title		Description	Works TK	Other TK	Total TK
1		Revetment/ Bed Protection:			
	a	Revetment by stones	288,020	583,848	871,868
	b	CC-blocks, production + dumping	1,476,692		1,476,692
		Subtotal 1	1,764,712	583,848	2,348,560
		Total Groyne No. G-1	1,764,712	583,848	2,348,560
		Grand Total Groyne No.G-1		TK	2,348,560
				DM (rate 27)	86,984

Table 10.18-2: Breakdown of Adaptation Costs of GroyneG-1

Analyses of Project Costs
Breakdown of total adaptation costs for groyne no. G-2

Title		Description	Works TK	Other TK	Total TK
1		Pile Preparation and Installation:			
1.1		Steel Piles:			
	a	Procurement		6,519,230	
	b	Transport to Site	61,440		
	c	Preparation	271,620		
	d	Installation	1,053,360		
	e	Head Preparation	22,800		
		Subtotal 1.1	1,347,780	6,519,230	7,867,010
1.2		Bored Piles:			
		Bored piles Inst.	635,328		635,328
		steel tube lining	514,350		514,350
		Reinforcement	704,125		704,125
		Gangway Support	32,000		32,000
		Subtotal 1.2	1,853,803		1,885,803
		Subtotal 1	3,201,583	6,519,230	9,720,813
2		Earth works:			
	a	Excavation/Fill	392,360		
		Working Platform for Piling Rig	80,850		
		Subtotal 2	392,360		392,360
3		Revetment/ Bed Protection:			
	i	CC-blocks, production + dumping	3,707,400		
	j	Brick mattressing	430,990		430,990
		Subtotal 3	4,138,390	-	4,138,390
4		Monitoring Gangway & Platform:			
	a	Gangway, steel, supply & inst	957,000		
		Subtotal 4	957,000		957,000
		Total Groyne No. G-2, 1-4	6,835,530	6,519,230	13,354,760
		Grand Total Groyne No.G- 2		TK	13,354,760
				DM (rate 27)	494,621

Table 10.18-3: Breakdown of Adaptation Costs of GroyneG-2

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Analyses of Project Costs
Breakdown of total adaptation costs for groyne no. G-3

Title		Description	Works TK	Other TK	Total TK
1		Steel Pile Preparation and Installation:			
	a	Procurement		10,508,772	
	b	Transport to Site	56,320		56,320
	c	Preparation	450,930		450,930
	d	Installation	1,707,600		1,707,600
	e	Head/Bracing	163,350		163,350
		Subtotal 1	2,378,200	10,508,772	12,886,972
2		Earth Works:			
	a	Excavation/Fill	322,000		
		Subtotal 2	322,000	-	322,000
3		Revetment/ Bed Protection:			
	I	CC-blocks, production + dumping	7,074,155		
		Subtotal 3	7,074,155	-	7,074,155
		Total Groyne No. G-3, 1-3	9,774,355	10,508,772	20,283,127
		Grand Total Groyne No.G-3		TK	20,283,127
				DM (rate 27)	751,227

Table 10.18-4: Breakdown of Adaptation Costs of GroyneG-3

Analyses of Project Costs
Breakdown of total adaptation costs for groyne no. G-A

Title		Description	Works TK	Other TK	Total TK
1		Pile Preparation and Installation:			
1.1		New piles			
	a	Procurement		1,358,144	1,358,144
	b	Transport to Site	12,800		12,800
	c	Preparation	66,600		66,600
	d	Installation	190,680		190,680
	e	Head/Bracing	44,000		44,000
		Subtotal 1.1	270,080	1,358,144	1,628,224
1.2		Old piles dia. 711 mm + 1016 mm			
		Installation	549,820		549,820
		Subtotal 1.2	549,820		549,820
		Subtotal 1	819,900	1,358,144	2,178,044
2		Revetment/ Bed Protection:			
	a	Boulders, procurement + dumping	2,023,303	8,782,258	10,805,561
	b	CC-blocks, production + dumping	8,255,436		8,255,436
		Subtotal 2	10,278,739	8,782,258	19,060,997
		Total Groyne No. G-A, 1-3	11,098,639	10,140,402	21,239,041
		Grand Total Groyne No. G-A		TK	21,239,041
				DM (rate 27)	786,631

Table 10.18-5: Breakdown of Adaptation Costs of Groyne G-A

Analyses of Project Costs
Breakdown of total costs for new groyne no. G-A/2

Title		Description	Works TK	Other TK	Total TK
1		Pile Preparation and Installation:			
1.1		Steel Piles:			
	a	Procurement		4,023,502	4,023,502
	b	Transport to Site	37,920		37,920
	c	Preparation	166,050		166,050
	d	Installation	627,270		627,270
		Subtotal 1.1	831,240	4,023,502	4,854,742
1.2		Bored Piles:			
		Bored piles Inst.	2,124,400		2,124,400
		steel tube lining	1,015,650		1,015,650
		Reinforcement	2,161,500		2,161,500
		Subtotal 1.2	5,301,550		5,301,550
		Subtotal 1	6,132,790	4,023,502	10,156,292
2		Earth works and pavements:			
	a	Excavation/Fill/Turfing	643,425		643,425
		Subtotal 2	643,425	-	643,425
3		Revetment/ Bed Protection:			
		Procurement of geo-textile		344,665	344,665
		Filter, geo-textile	64,179		64,179
		CC-blocks, production + dumping	4,693,155		4,693,155
		Brick mattressing	938,120		938,120
		Subtotal 3	5,695,454	344,665	6,040,119
		Total Groyne No. G-A/2, 1-3	12,471,669	4,368,167	16,839,836
		Grand Total Groyne No. G-A/2		TK	16,839,836
				DM (rate 27)	623,698

Table 10.18-6: Breakdown of Costs of G-A/2

Analyses of Project Costs
Breakdown of total costs for relocation & improvement of BWDB embankment

Title		Description	Works TK	Other TK	Total TK
1		Earth works			
	a	Site Cleaning	183,270		183,270
	b	Excavation/Fill	7,799,795		7,799,795
		Subtotal 2	7,983,065	-	7,983,065
2		Revetment			
	a	Procurement of geo-textile		2,278,385	2,278,385
	b	Filter, geo-textile	424,251		424,251
	c	Filter, granular	507,500		507,500
	d	CC-blocks, production + dumping	522,161		522,161
	e	Brick mattressing	6,561,960		6,561,960
	f	Wire Mash	100,650		100,650
	g	Turfing/Vetiver	114,480		114,480
	h	Palisade	466,298		466,298
		Subtotal 3	2,278,385	2,278,385	4,556,770
		Total Re-location and	10,261,450	2,278,385	12,539,835
		Improvement BWDB Embankment			
		Grand Total Embankment		TK	12,539,835
				DM (rate 27)	464,438

Table 10.18-7: Breakdown of Adaptation Costs of the Embankment

Analyses of Project Costs
Breakdown of total general costs for adaptation of groynes

Title		Description	Works TK	Other TK	Total TK
1		General Items:			
	a	Mobilisation, Re-mobilisation	6,506,903		
	b	Preparation	8,250,000		
	c	Demobilisation	2,425,000		
	d	Salvage of Material at damage zones	562,297		
	e	Hire of Equipment	3,020,000		
	f	Construction of Steel Mooring Float	1,000,000		
	g	Fuel for Monitoring	2,251,831		
	h	Camp Maintenance, 43 month	11,937,883		
	i	Camp Shifting and re-installation	3,112,232		
	j	Rent of Land for Site Installation	905,490		
	k	Boat Rent For Groyne Cleaning	277,147		
		Subtotal	40,248,783		
	l	Daywork and other non specified	18,329,803		
		Total	58,578,586	-	58,578,586
		Grand Total General Items		TK	58,578,586
				DM (rate 27)	2,198,076

Table 10.18-8: Breakdown of the General Costs for the Adaptation of the Groynes



Analyses of Project Costs
Summary of total costs for adaptation of groynes

Title	Description	Works & Procurement	
		TK	DM
1	Groyne G-1	2,348,560	86,984
2	Groyne G-2	13,354,760	494,621
3	Groyne G-3	20,283,127	751,227
4	Groyne G-A	21,239,041	786,631
5	Groyne G-A/2	16,839,836	623,698
6	Embankment	12,539,835	464,438
	Subtotal 1	86,605,159	3,207,598
7	General	58,578,586	2,198,076
	Subtotal 2	145,183,745	5,405,674
8	Surplus Boulder	7,159,132	264,736.32
9	Surplus Geotextile	738,071	26,897.63
10	Port & Demurrage Charges	2,014,491	74,174
11	Other	7,966,354	302,455
	Subtotal 3	17,878,048	668,263
	Grand Total Adaptation Groynes	163,061,793	6,073,937

Table 10.18-9: Summary of total costs for adaptation of the groynes

REFERENCES

- [1] FAP 21 – Final Report Planning Study (June 1993)
- [2] FAP 21 – Procurement and Construction Report, Test Site 1 – Kamarjani (January 1994)
- [3] FAP 21 – Final Project Evaluation Report (2000)
- [4] FAP 21/22 – Morphological Predictions for Test Areas – November 1993
- [5] FAP 21 - Construction of Permeable Groynes, Tender Evaluation (1994)

Attachment 1
Selection of Design
and Construction Drawings
(Implementation)

260

ATTACHMENT 1

List of Drawings

Drawing No. KA-003/1:	General Layout of Groynes
Drawing No. KA-008:	General Installation Method, Steel Sheet Pile Cofferdam
Drawing No. KA-009:	General Installation Method, Tubular Steel Piles
Drawing No. KA-015:	General Installation Method, Bored Concrete Piles
Drawing No. KA-101:	Groyne G-1, General Arrangement of Plan, Elevation
Drawing No. KA-201:	Groyne G-2, General Arrangement of Plan, Elevation
Drawing No. KA-301:	Groyne G-3, General Arrangement of Plan, Elevation
Drawing No. KA-401:	Groyne G-A, General Arrangement of Plan, Elevation
Drawing No. KA-402/1:	Groyne G-A, In-situ Concrete Pile Dia. 914 (36"), (Bored Pile)
Drawing No. KA-403:	Groyne G-A, Prestressed Spun Concrete Pile dia. 500 x 100 mm
Drawing No. KA-405:	Groyne G-A, Reinforced Concrete Sheet Piling
Drawing No. KA-501/1:	Groyne G-B/1 General Arrangement of Plan, Elevation

J A M U N A R I V E R

N

Existing ponds (previous borrow pits) was drained completely, mud from pond bottom to be removed and pile to be filled in layers with suitable sandy soil, including compaction of each layer to $D = 0.75$ (Spec. Subsection 935 (b))

Existing embankment to be removed. Material to be used for layerwise refilling of ponds



CONTROL POINT SCHEDULE			
ID No. of Control Point	Coordinates BTM		Elevation (m PWD)
	Easting	Northing	
FAP 21.04	481 935.686	804 621.661	+ 20.677
FAP 21.05	490 435.320	802 817.795	+ 21.357
FAP 21.16	481 763.470	804 952.676	+ 21.436

LEGEND

- Road Pavca, Kacha, Footpath
- High Water Bank
- High Water Bank & Trashed
- Training Station & Piller
- Horn Station
- Mission

Notes

- All measurements in meters
- Levels refer to 5.0m PWD
- Flood plain levels are determined at site within the area of permanent
- Reference Drawings

- KA-002 General Layout of Implementation Proposal
- KA-001 Groyne G-1, General Arrangement
- KA-001 Groyne G-2, General Arrangement
- KA-001 Groyne G-3, General Arrangement
- KA-001 Groyne G-A, General Arrangement
- KA-501/1 Groyne G-B/1, General Arrangement
- KA-501/2 Groyne G-B/2, General Arrangement

0 50 100 150 200 m

STEP IV

Direction of Pile Installation

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

3.0 m

1.5 m

0.5 m

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SECTION A-A

- Remove shift and rain stall pile installation template, starter end of template fixed to pile ⑤
- Pitch piles ⑦ to ⑫

Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

3.0 m

1.5 m

0.5 m

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STEP V

Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

3.0 m

1.5 m

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- Drive piles carefully in the sequence ⑫ - ⑩ - ⑧ by about 3.5m

- Remove upper horizontal beams of template;
- Continue pile driving in the sequence ⑪ - ⑨ - ⑩ - ⑧ - ⑥ to designed pile head level;
- Pile ⑫ serves as starter pile for pitching the next panel of 6 double piles, i.e. STEPS IV, V, VI to be repeated analogously.

STEP VI

Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

3.0 m

1.5 m

0.5 m

0.5 m

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Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

3.0 m

1.5 m

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Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

3.0 m

1.5 m

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Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

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3.0 m

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Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

4.5 m

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Direction of Pile Driving

12 11 10 9 8 7

6 5 4 3 2 1

Design head level

8.0 m

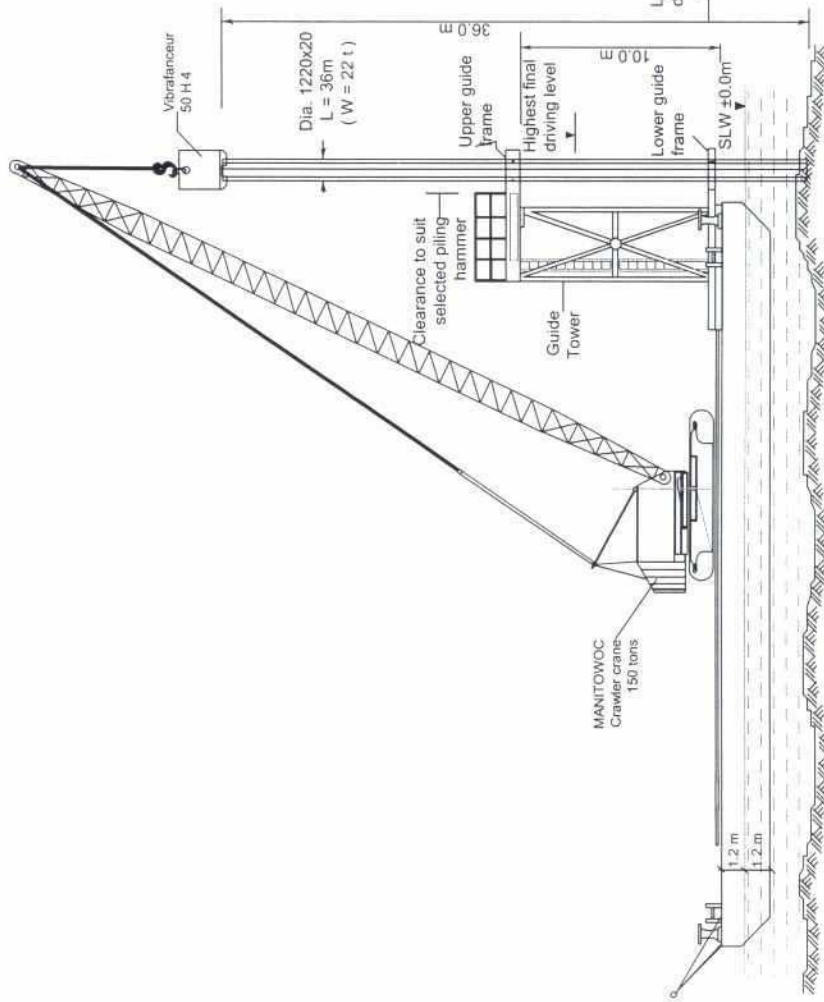
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3.0 m

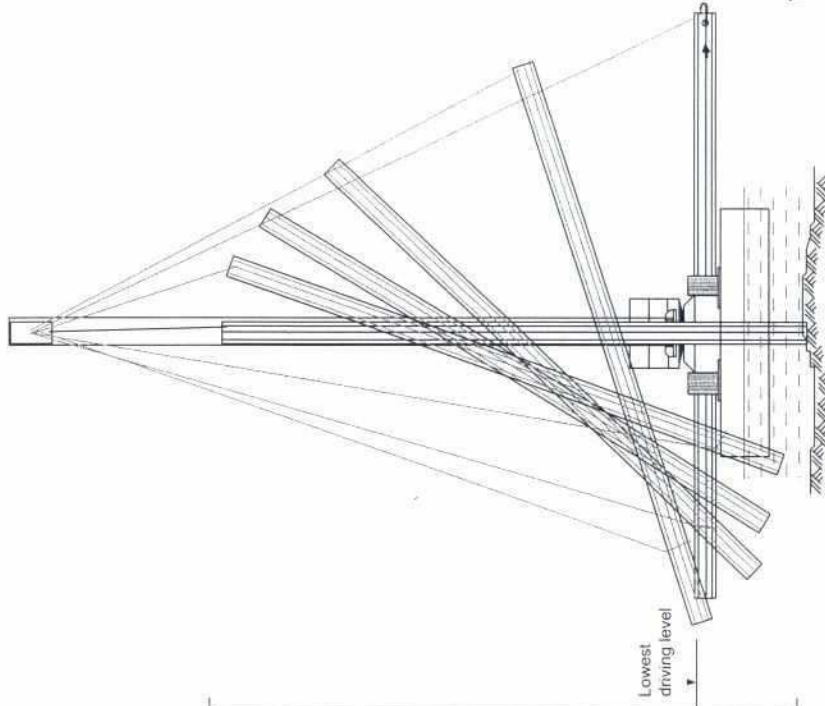
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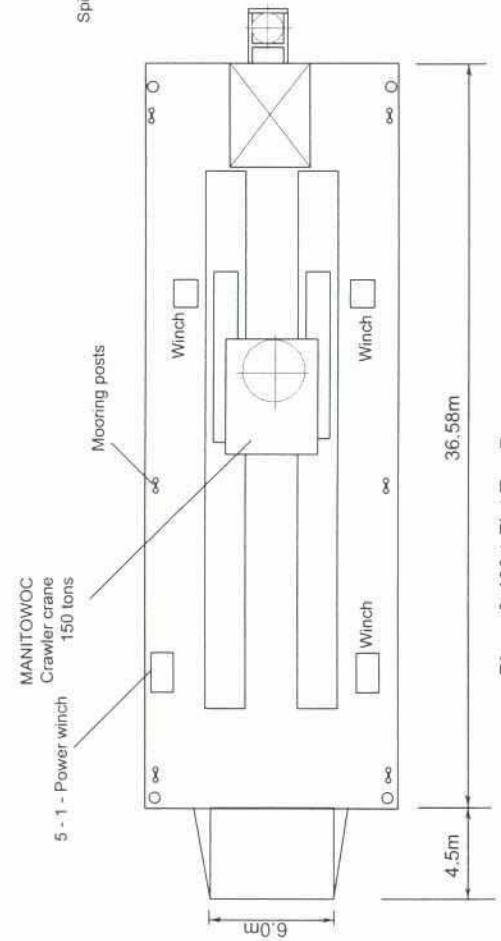


400-t Flat Top Barge With 150-t Crawler Crane And Pile Guide tower

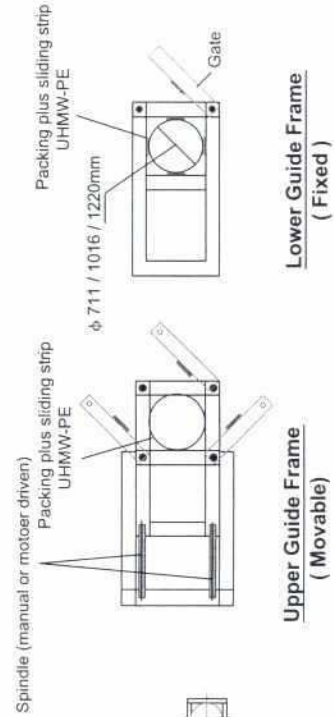


60-t to 100-t Crawler Crane with 100-ft-Boom and piling Leader (L > 36m)

Stages of Pile Lifting Off - Shore (analogously to be applied on-shore)



Plan of 400-t Flat Top Barge



Lower Guide Frame (Fixed)

Upper Guide Frame (Movable)

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JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE CONSULTING ENGINEERS & ARCHITECTS 100, RAJSHAHI ROAD, RAJSHAHI DHAKA-1000, BANGLADESH COMP. NO. 100/RAJSHAHI ROAD, RAJSHAHI DHAKA-1000, BANGLADESH		TEST SITE I - KAMARJANI	
GENERAL INSTALLATION METHOD TUBULAR STEEL PILES		SCALE 1:2000	
DRAWING NO KA - 009		REVISION 0	
APPROVED	CHECKED	DATE	NAME
		25.10.2006	AMARJANI

1

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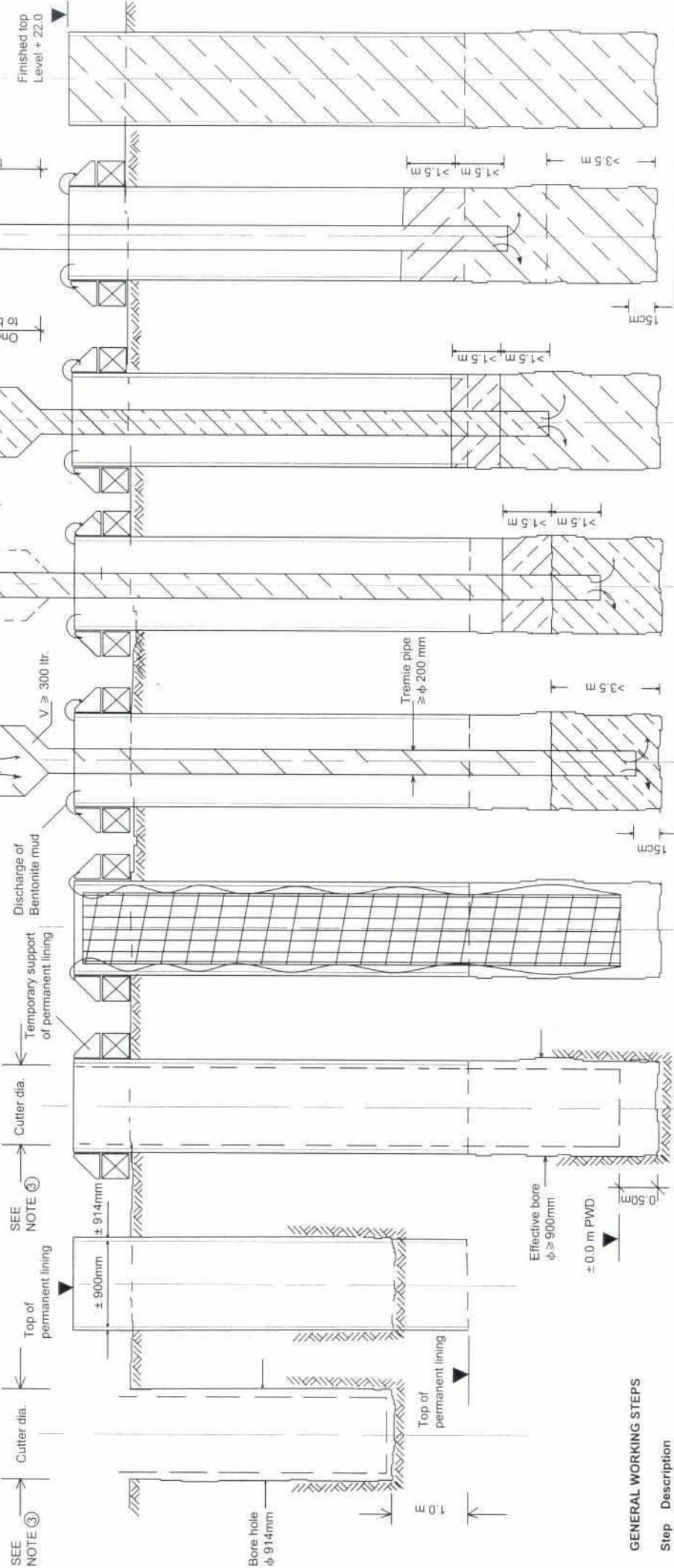
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6

7

18

19



GENERAL WORKING STEPS

Step	Description
(1)	Pre-boring
(2)	Install permanent steel tube lining composed of individual tube sections of -3.6 m length, but welded at place in vertical position; Steel tube to be inserted to design level by rotating and pushing the tube.
(3)	Continue boring the hole to final depth. Make sure to select the cutter head diameter - to ensure design pile diameter, - to ensure safe insertion and extraction of the cutter head through casing tube.
(4)	Prior to inserting the reinforcing cage, Bentonite mud has to be replaced completely and bore hole depth confirmed. Secure reinforcing cage at design level.
(5)	Insert tremie pipe with a disposable plug (to avoid segregation of concrete) and place initial concrete pour as per Specification.

BORING METHOD AND EQUIPMENT

Preferably rotary drill rigs with direct mud circulation method shall be used.

Reverse circulation rotary rigs are likely to produce suction and may destabilize the bore hole to unacceptable extent.

All conditions of Specifications, section 1200 are to be adhered to.

Detailed boring method and working steps are subject to approval by the Employer

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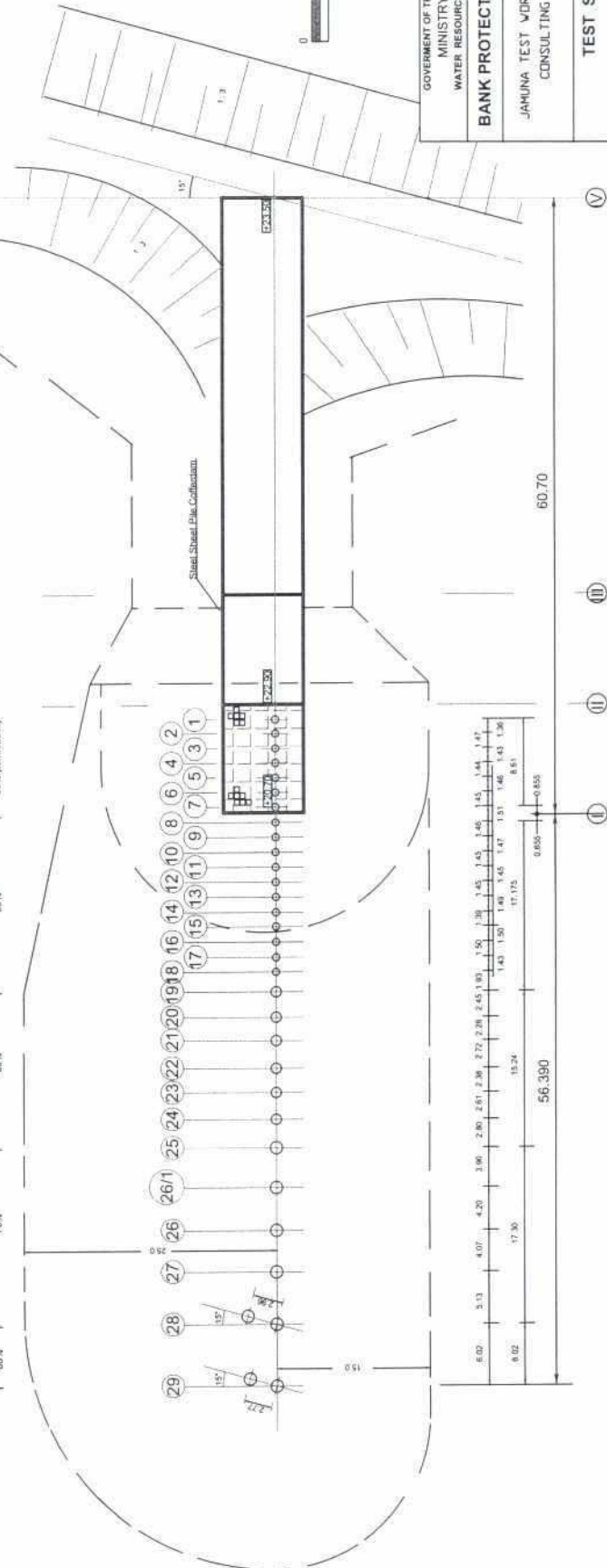
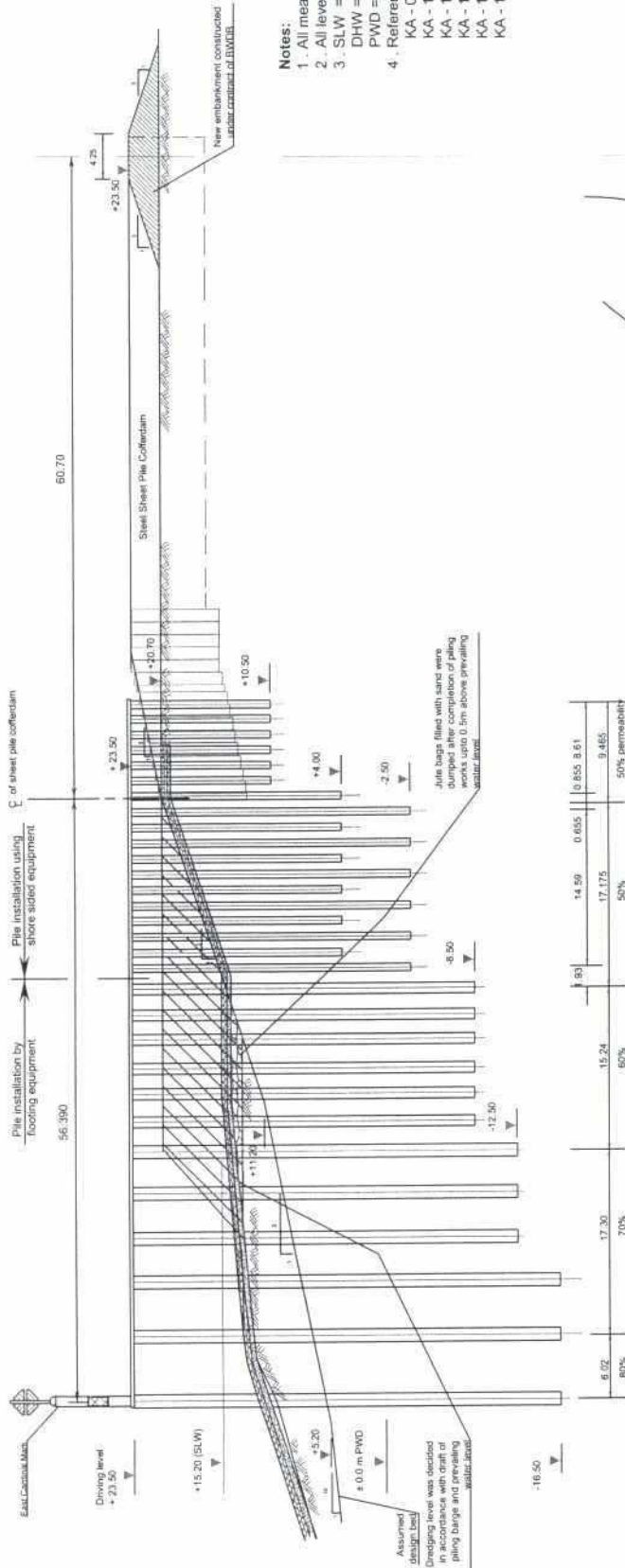
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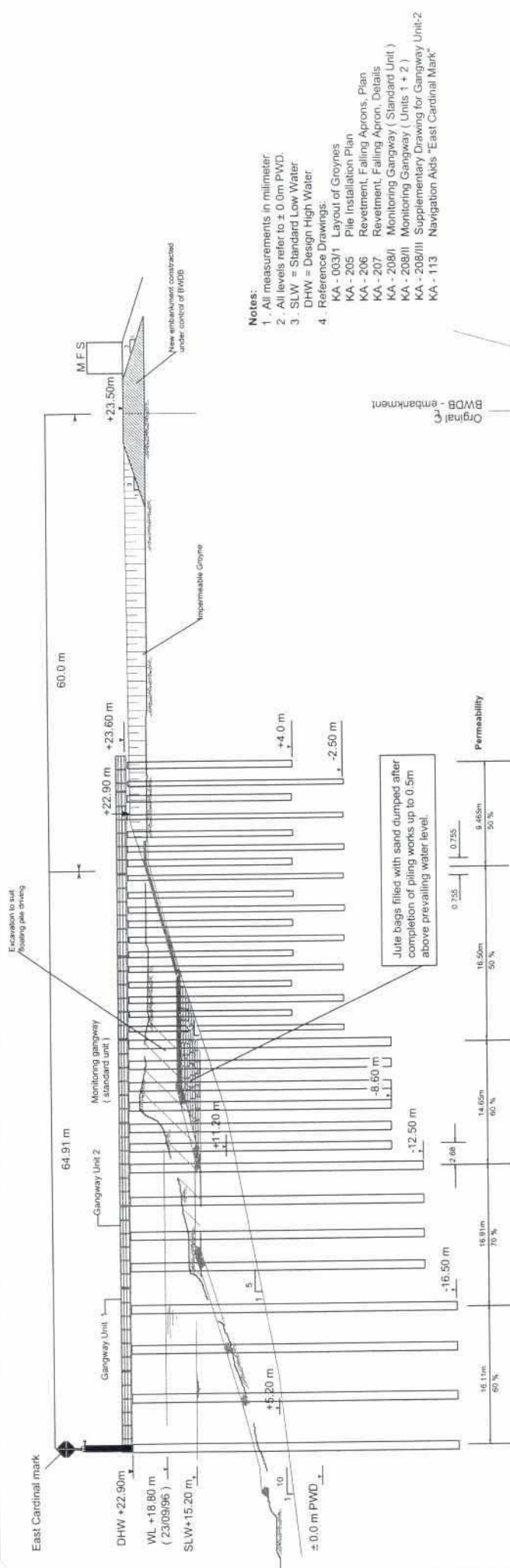
TEST SITE I - KAMARJANI

**GENERAL INSTALLATION METHOD
BORED CONCRETE PILES**

SCALE
DRAWING NO
REVISION
KA - 015
0

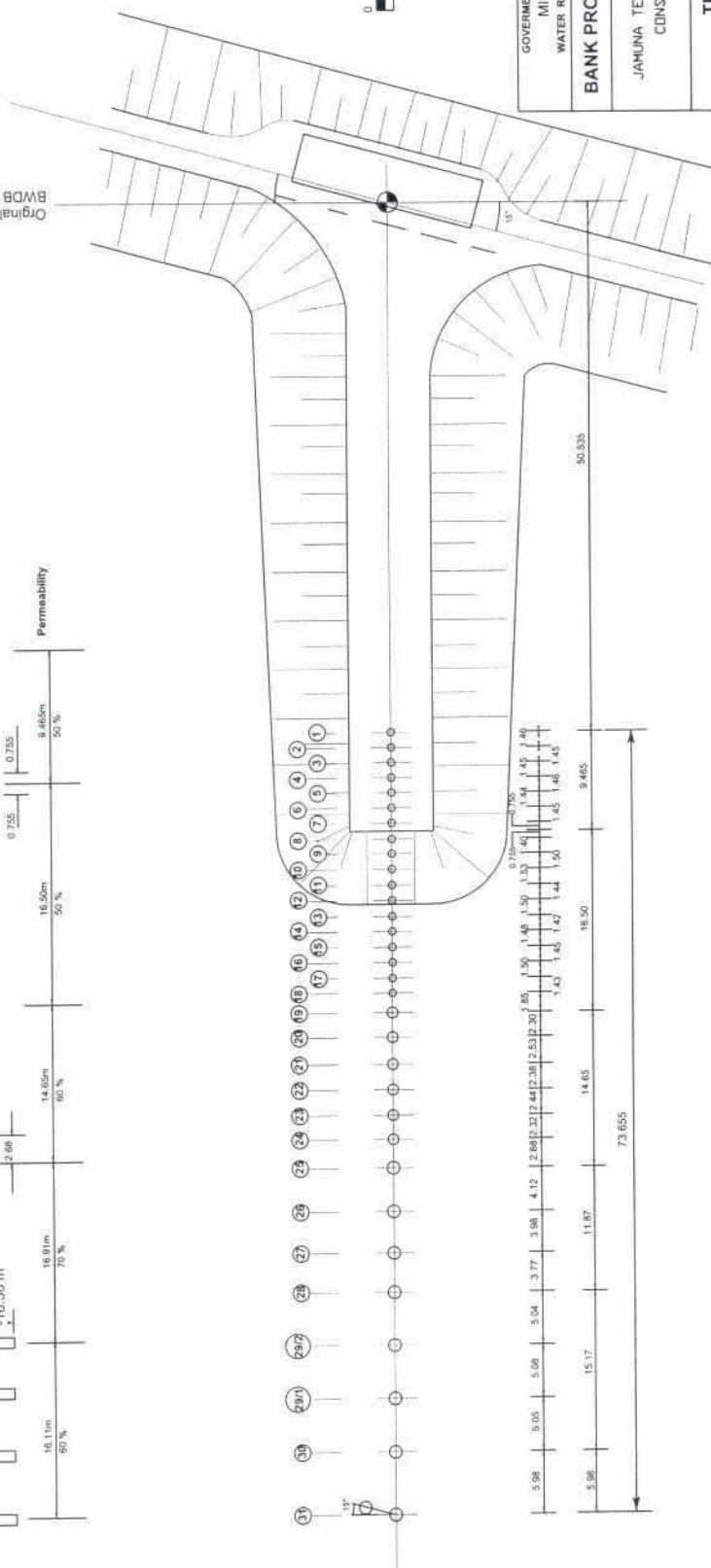


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BANK PROTECTION PILOT PROJECT FAP - 21			
JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE			
CONSULTING CONSORTIUM FAP-21/22			
TEST SITE I - KAMARJANI			
GROYNE G - 1			
GENERAL ARRANGEMENT OF PLAN - ELEVATION			
SCALE			
NAME	DATE	DRAWING NO	REVISION
ANOWAR	25-03-2001	KA - 101	0
DESIGNED			
CHECKED			
APPROVED			



Notes:

1. All measurements in millimeter;
2. All levels refer to ± 0 Om PWD;
3. SLW = Standard Low Water;
DHW = Design High Water;
4. Reference Drawings:
KA - 003/I Layout of Groynes
KA - 205 Pile Installation Plan
KA - 206 Revetment, Felling Aprons, Plan
KA - 207 Revetment, Felling Apron, Details
KA - 208/I Monitoring Gangway (Standard Unit)
KA - 208/II Monitoring Gangway (Units 1 + 2)
KA - 208/III Supplementary Drawing for Gangway Unit-2
KA - 118 Navigation Aids "East Cardinal Mark"



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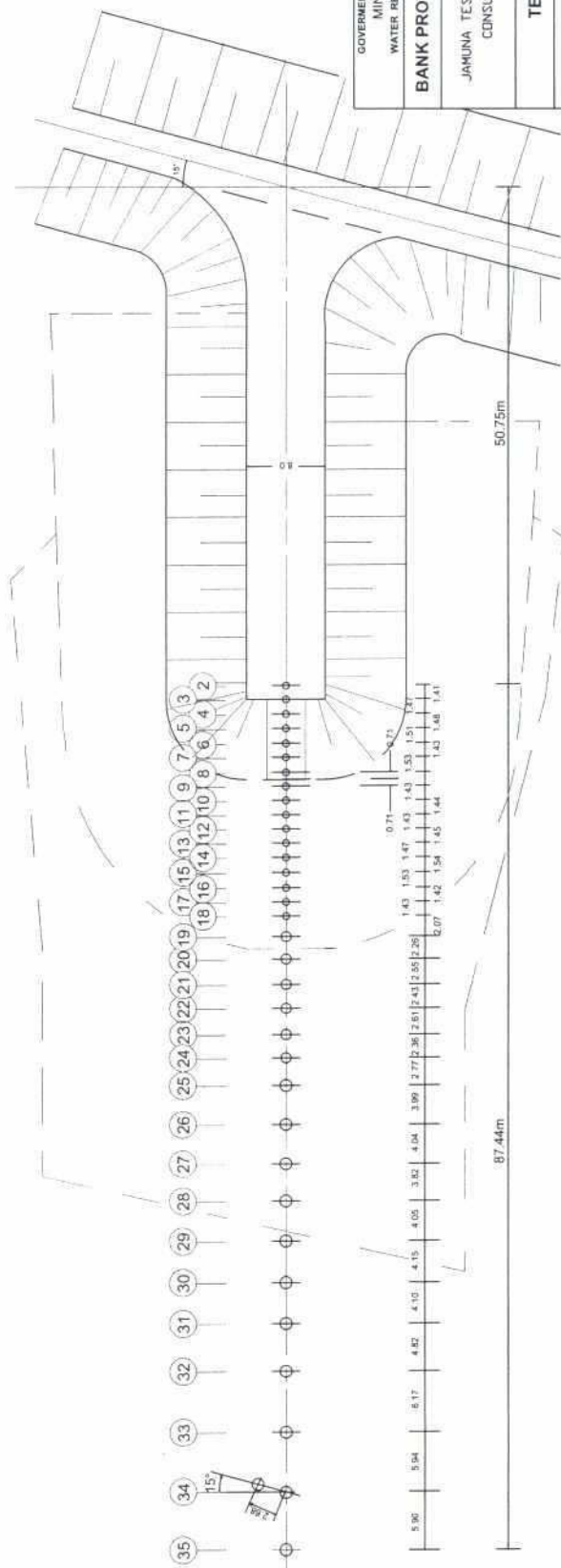
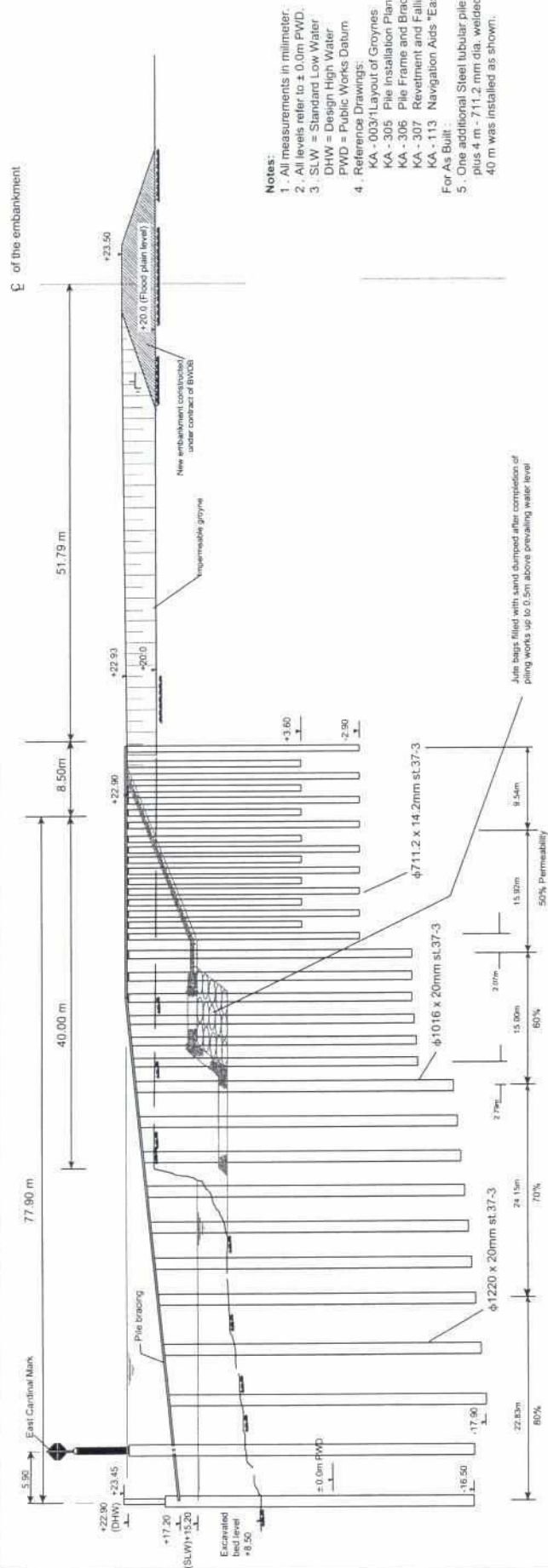
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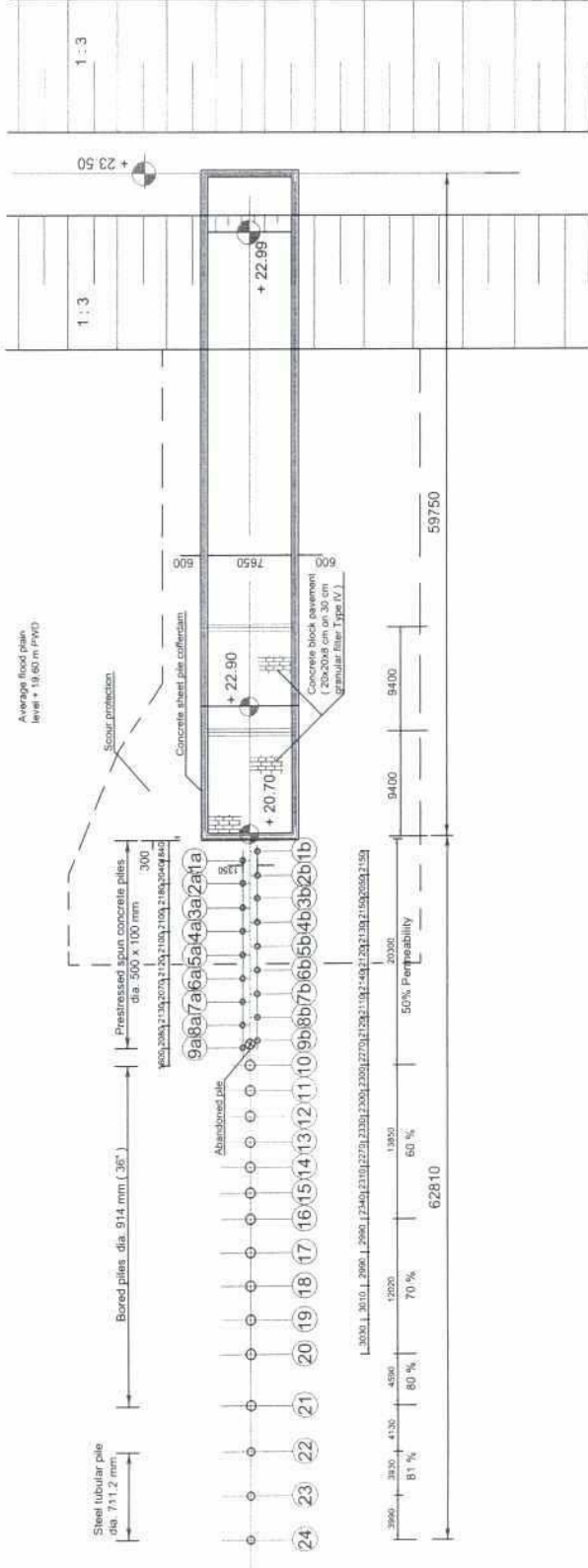
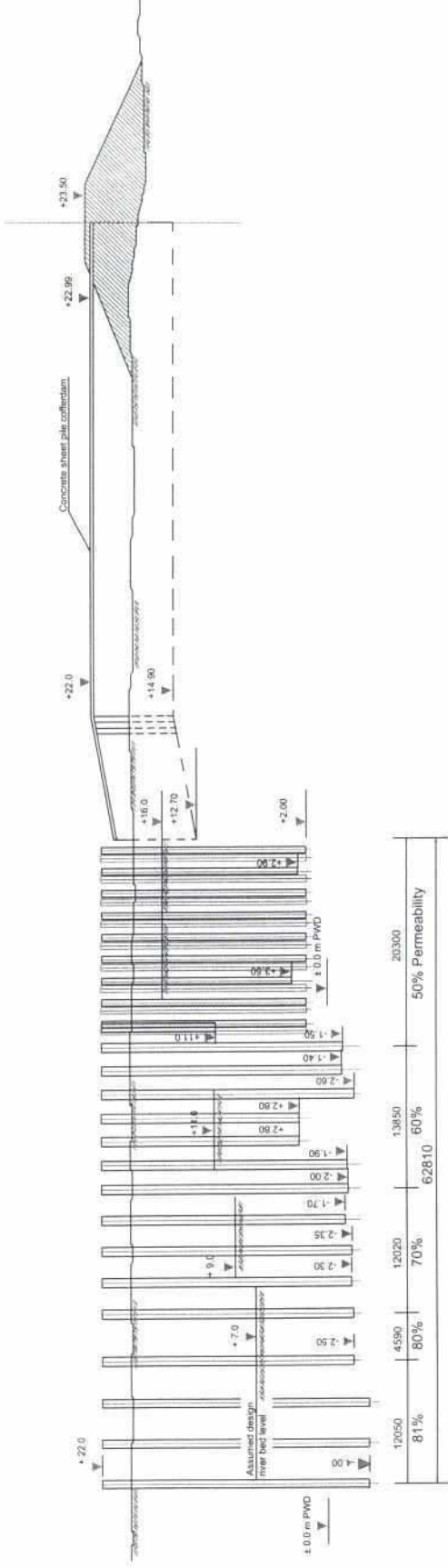
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CONSULTING CONSORTIUM FAP-21/22

TEST SITE I - KAMARJANI

GROYNE G - 2
GENERAL ARRANGEMENT OF
PLAN, ELEVATION

	NAME	DATE	SCALE
DRAWN	ANYWAY	25-03-2001	
CHECKED			DRAWING NO
APPROVED			KA-201
			REVISION
			0





- Notes:**
1. All measurements in millimeter.
 2. All levels refer to ± 0.0 m PWD.
 3. Reference Drawings:
 - KA - 404 Pile Installation Plan
 - KA - 406 Concrete Sheet Pile Cofferdam
 - KA - 003/ Layout of Groynes
 - KA - 414 Details of Scour Protection
 4. Bored pile No. 9 abandoned due to collapse of bore hole during installation.
 5. Two additional spun pile installed.
 6. Three additional steel tubular pile installed.
 7. Two spun piles could not be driven to design depth. Undriven part was cut off and repaired.



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BANK PROTECTION PILOT PROJECT FAP - 21			
JAMUNA TEST WORKS CONSULTANTS JOINT VENTURE CONSULTING CONSORTIUM FAP-21/22			
TEST SITE I - KAMARJANI			
GROYNE G - A			
GENERAL ARRANGEMENT OF PLAN - ELEVATION			
NAME	DATE	SCALE	
DRAWN	21-03-2001		
CHECKED			
APPROVED			
		DRAWING NO	REVISION
		KA - 401	0

REINFORCEMENT BAR SCHEDULE (Steel Grade 60)				
ITEM No.	BAR DIA. (mm)	BAR SHAPE (in meter)	Nos.	CUTTING LENGTH (m)
1	25 (1")	—	24	5.50
2	25 (1")	—	24	10.50
3	25 (1")	—	24	11.00
4	φ 9.5 (φ 3/8")	0.20-1" Spiral		310.00
5	50 x 4 mm	0.80 x 0.053	64	0.80
Total Weight Per Pile				2747.23
TOTAL WEIGHT FOR 12 Nos. BORED PILES				32,966.76' kg.

MATERIAL SCHEDULE - STEEL TUBE LINING Dia. 914 x 6mm, Steel grade St.37				
LOCATION PILE No.	LENGTH OF STEEL TUBE LINING (m)	No.	WEIGHT / No. (kg)	TOTAL WEIGHT (kg)
10 to 15	11.00	6	1,476	8,856
16 to 19	13.00	4	1,747	6,988
20, 21	15.00	2	2,015	4,030
9	8.30	1	1,115.21	1,115.21
TOTAL				21001.21

SHEET PILE SUPPLY SCHEDULE				
Type	Supplied Nos.	Total Weight of Reinforcement Steel (Kg.)	Remark	
① Starter Pile	6	1,502.34	Used	6
② Starter Pile	310	86,093.20	Surplus	11
TOTAL		87,595.54		

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH

MINISTRY OF WATER RESOURCES

WATER RESOURCES PLANNING ORGANIZATION (WARPO)

BANK PROTECTION PILOT PROJECT FAP - 21

ADDITIONAL TEST REPORT CONTRACT NO. 001/2010

CONSULTANT: CH2M HILL

DESIGNER: CH2M HILL

CONTRACTOR: CH2M HILL

TESTER: CH2M HILL

DATE: 01/01/2010

TEST SITE I - KAMARJANI

- ADAPTION WORKS -

GROUPE TYPE G - A

IN - SITU CONCRETE PILE DIA. 914mm(36")

(BORED PILE)

NAME

DATE

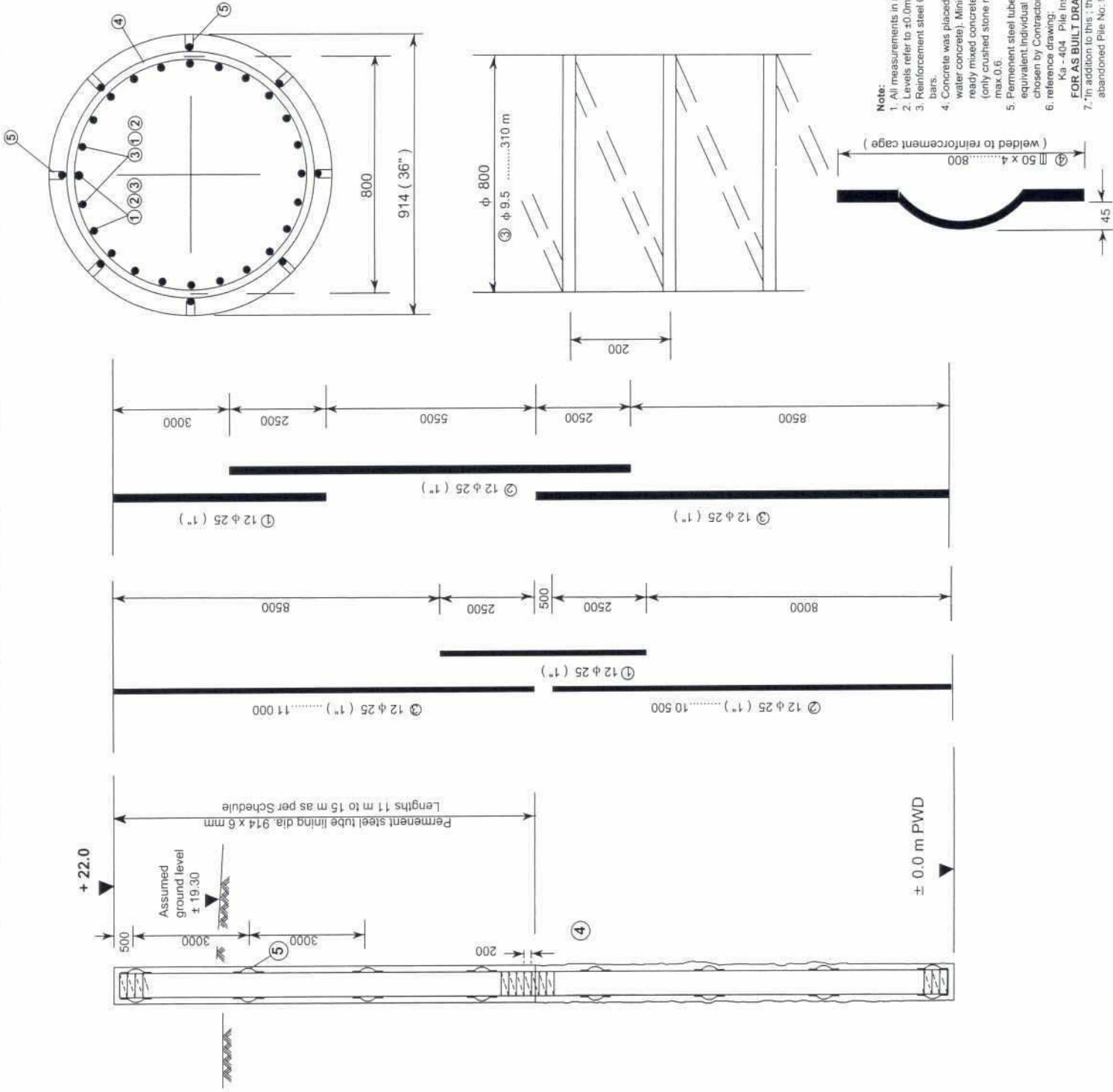
SCALE

DRAWING NO

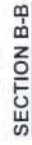
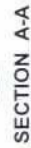
REVISION

KA - 402 / I

1

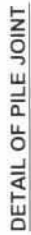


Bottom pile section
10,000

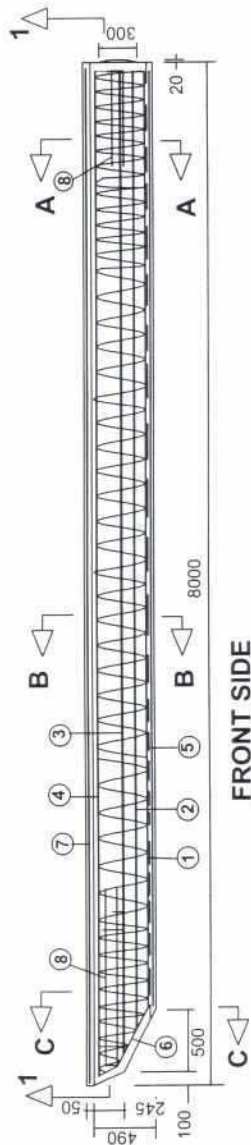


1. Prestressed spun concrete piles are produced by GEMCON Ltd, Panchagar.
2. Concrete quality grade B 45, DIN 1045.
3. Prestressing wire $>1600 \text{ N/mm}^2$ tensile strength.
4. Piles were not handled/lifted till the concrete has reached compressive strength of atleast 35 N/mm^2 .

Site weld during pile

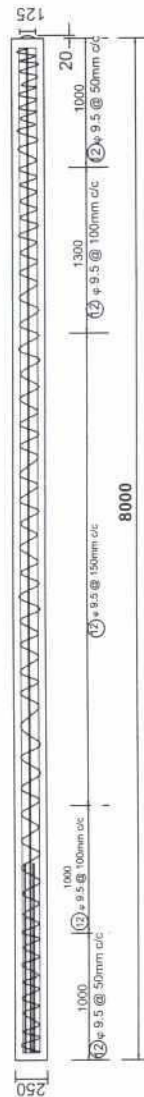


GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF WATER RESOURCES WATER RESOURCES PLANNING ORGANIZATION (WARPO)		JAMUNA TRIPLEX LIMITED, 301/1, 301/2, 301/3 COMPLEX, COMPLETION F.A. 1/201 HINTER HOUSE NO. 10/1 BARI, SOUTH SIDE OF RAJSHAHI ROAD, RAJSHAHI, DISTRICT RAJSHAHI, DIVISION RAJSHAHI (DIPLOMA IN CIVIL ENGINEERING) (DIPLOMA IN CIVIL ENGINEERING) (DIPLOMA IN CIVIL ENGINEERING)		REVISION 0	
BANK PROTECTION PILOT PROJECT FAP - 21		TEST SITE I - KAMARJANI - ADAPTATION WORKS - GROYNES TYPE G - A PRESTRESSED SPUN CONCRETE PILE DIA. 500 x 1000 mm		DRAWING NO KA- 403	
NAME	DATE	SCALE			
DESIGNED	APPROVED	BY 10/2016			
CHECKED	APPROVED				



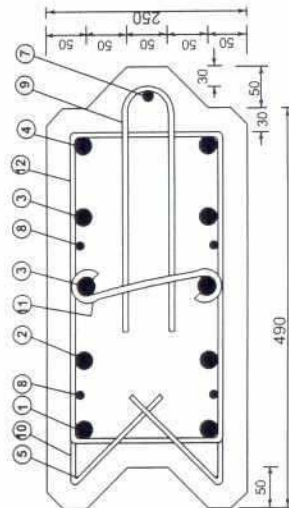
FRONT SIDE

Scale 1 : 400



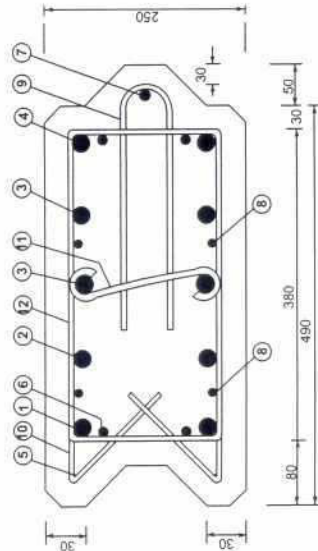
LONGITUDINAL SECTION 1-1

Scale 1 : 400



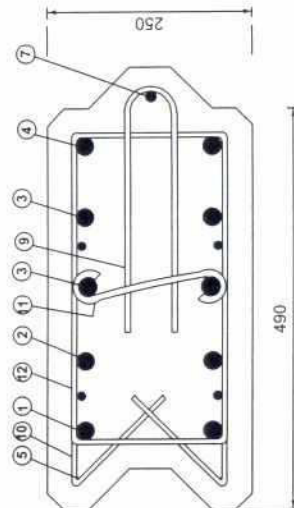
SECTION A-A

Scale 1 : 40



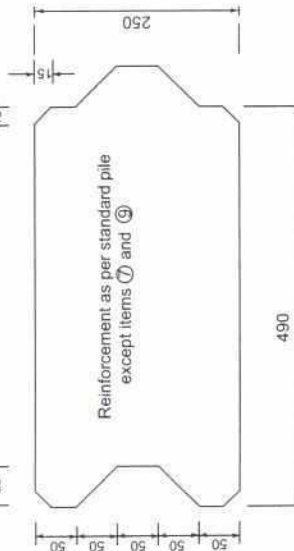
SECTION C-C

Scale 1 : 40



SECTION B-B

Scale 1 : 40



SECTION OF STARTER PILE

Scale 1 : 40

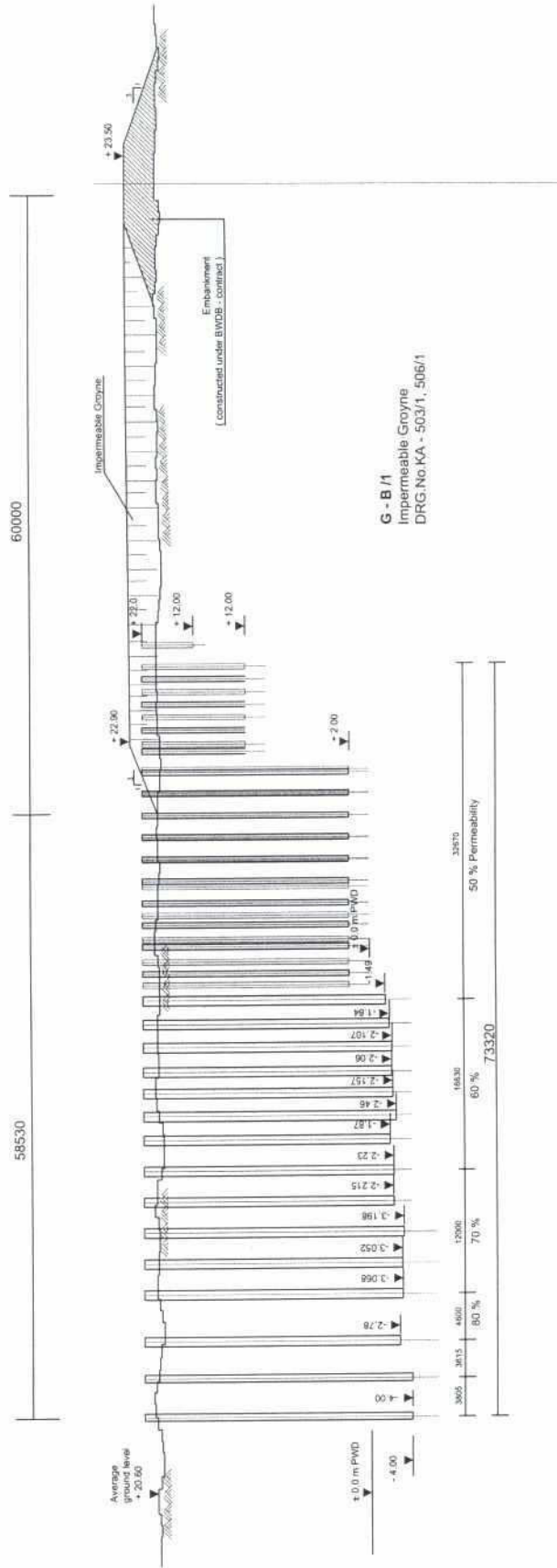
- Note:**
1. All measurements in millimeter.
 2. Reinforcement steel, deformed bars, Grade 60 ASTM A-615
 3. Concrete class B 45, DIN 1045.
 4. Lifting and handling of precast sheet piles were done after concrete compressive strength had reached 35 N/mm²
 5. Reference Drawing : KA - 406 Concrete Sheet Pile Cofferdam Pile Installation Plan, Details

BAR SCHEDULE FOR ONE SHEET PILE				
ITEM No.	BAR DIA.	BAR SHAPE (in mm)	No.	WEIGHT (kg)
1	15.9 (15.9)	7500	2	24.33
2	15.9 (15.9)	7500	2	24.17
3	15.9 (15.9)	7500	4	49.61
4	15.9 (15.9)	7500	2	25.60
5	9.5 (9.5)	6350	2	7.84
6	9.5 (9.5)	6350	2	3.70
7	9.5 (9.5)	7500	1	4.87
8	9.5 (9.5)	1500	4	7.90
9	9.5 (9.5)	1500	52	22.46
10	9.5 (9.5)	1500	96	29.62
11	9.5 (9.5)	1500	52	12.83
12	9.5 (9.5)	1500	Spiral 105	64.79
Total Weight for one Standard Pile				277.72
Total Weight for one Standard Pile (7 and 8 deleted)				250.39

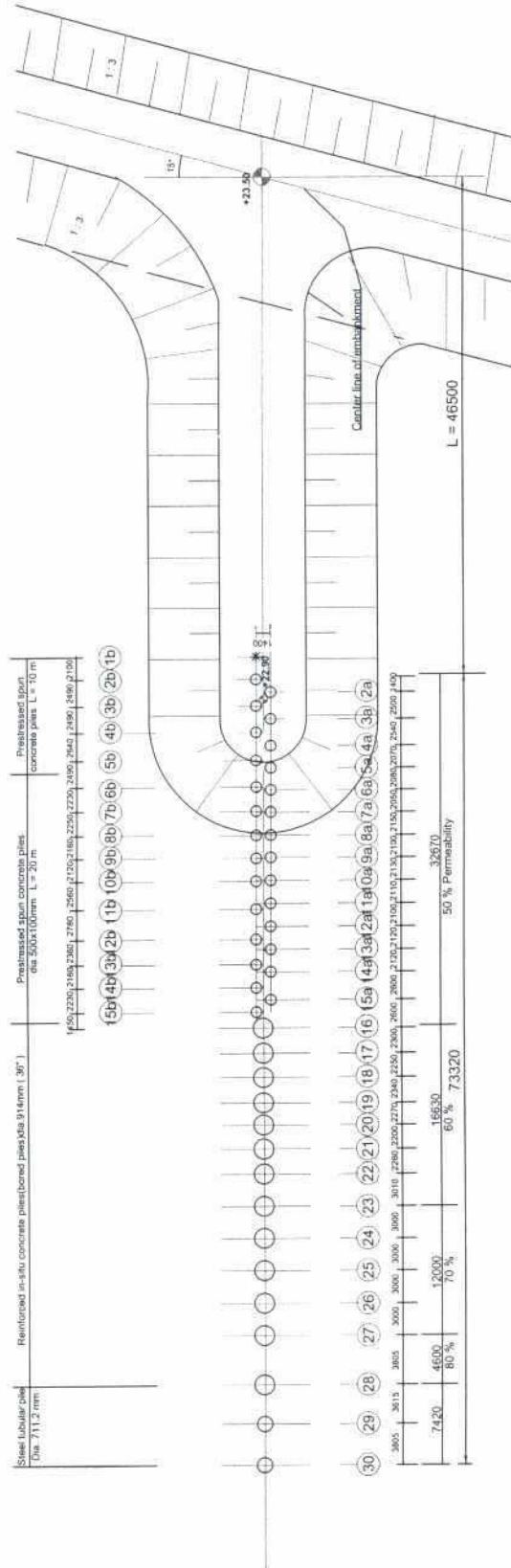
SHEET PILE SUPPLY SCHEDULE

Type	Supplied Nos.	Total Weight of Reinforcement Steel (kg)	Remark
1 Starter Pile	6	1,502.34	Used
2 Starter Pile	310	86,093.20	Surplus
TOTAL		87,595.54	11

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH	
MINISTRY OF WATER RESOURCES	
WATER RESOURCES PLANNING ORGANIZATION (WARPO)	
BANK PROTECTION PILOT PROJECT FAP - 21	
JAKHIA 1517 PILE COFFERDAM, WEST VENTURE	
CONSULTING ENGINEER FAP - 103	
DESIGNER: JAKHIA 1517 PILE COFFERDAM, WEST VENTURE	
CHECKED: JAKHIA 1517 PILE COFFERDAM, WEST VENTURE	
APPROVED: JAKHIA 1517 PILE COFFERDAM, WEST VENTURE	
TEST SITE 1 - KAMARJANI	
- ADAPTAION WORKS -	
GROYNE TYPE G - A	
REINFORCED CONCRETE SHEET PILING	
NAME	SCALE
DATE	10-10-2000
DRAWN	APPROVED
CHECKED	APPROVED
DRAWING NO	KA-405
REVISION	1



- Notes:**
1. All measurements in millimeter.
 2. All levels refer to ± 0.0m PWD.
 3. Reference Drawings:
KA - 504/1 Pile Installation Plan



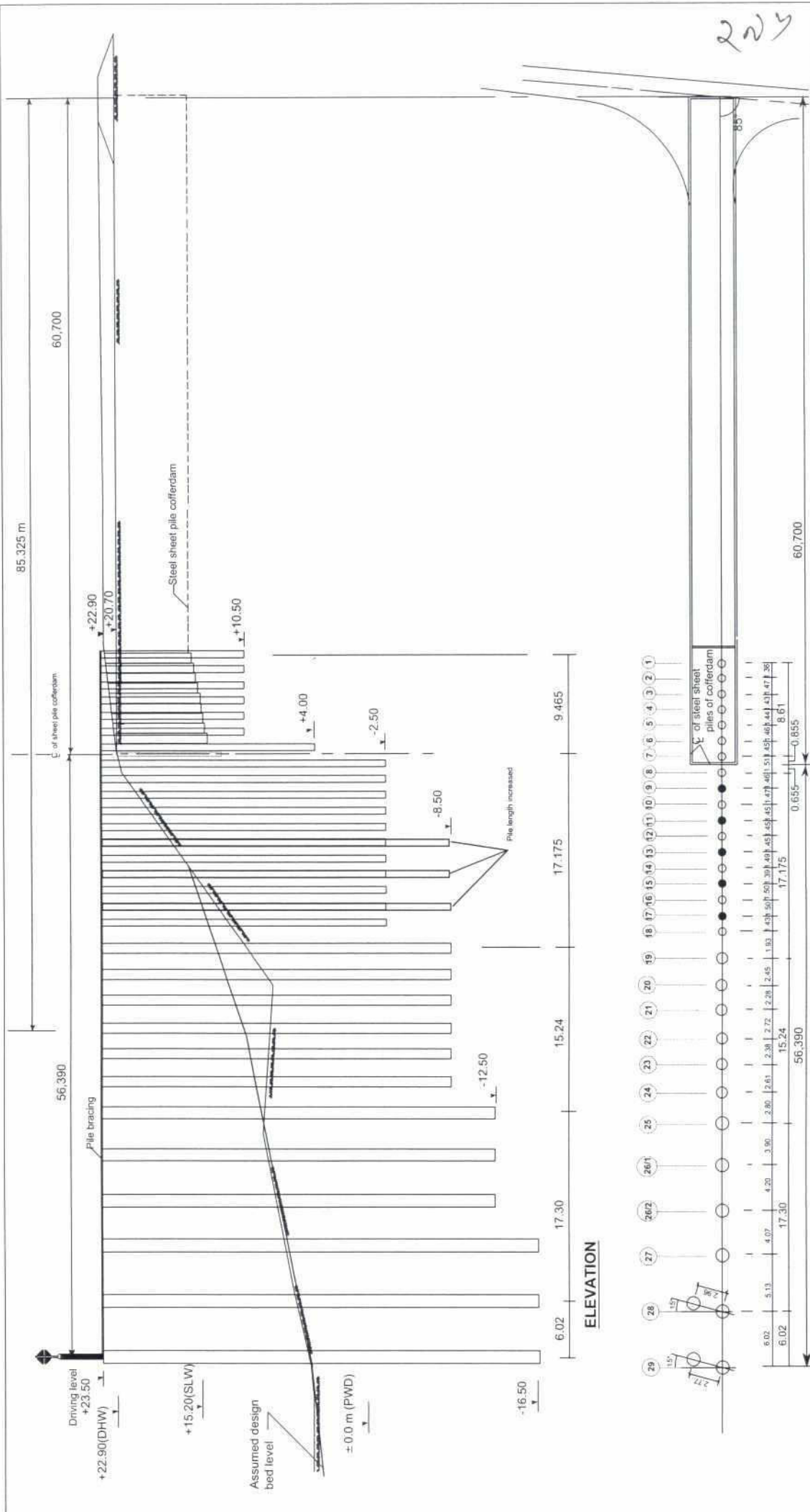
GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF WATER RESOURCES WATER RESOURCES PLANNING ORGANIZATION (WARPO)			
BANK PROTECTION PILOT PROJECT FAP - 21			
JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE CONSULTING CONSORTIUM FAP-21/22			
TEST SITE I - KAMARJANI			
GROYNE G - B / 1			
GENERAL ARRANGEMENT OF PLAN, ELEVATION			
NAME	DATE	SCALE	REVISION
APPROVED	APPROVED	APPROVED	APPROVED
CHECKED	CHECKED	CHECKED	CHECKED
DRAWN	DRAWN	DRAWN	DRAWN
DRAWING NO. KA - 501/1			0

Attachment 2
Selection of Design
and Construction Drawings
(Adaptation)

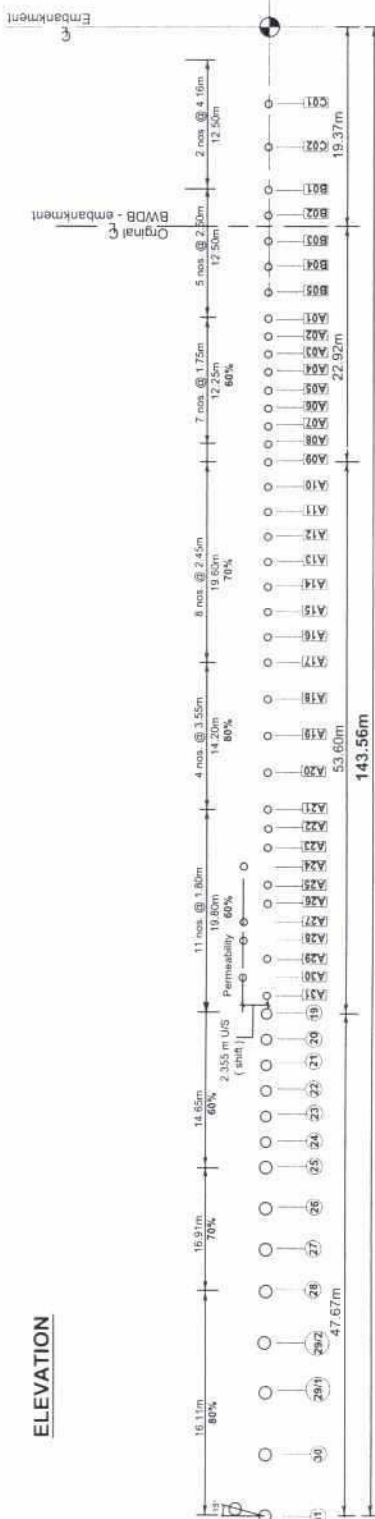
ATTACHMENT 2

List of Drawings

Drawing No. AD-KA-001:	Adaptation of Test Structures, General Project Layout
Drawing No. AD-KA-011:	Groyne Type G-1, Pile Layout Plan
Drawing No. AD-KA-021:	Groyne Type G-2, Pile Layout Plan
Drawing No. AD-KA-031:	Groyne Type G-3, Pile Layout Plan
Drawing No. AD-KA-041:	Groyne Type G-A, Pile Layout Plan, Plan & Elevation
Drawing No. AD-KA-051:	Groyne Type GA-2, General Arrangement of Plan, Elevation



GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF WATER RESOURCES WATER RESOURCES PLANNING ORGANIZATION (WARPO)			
BANK PROTECTION PILOT PROJECT FAP - 21			
JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE CONSULTING CONSORTIUM FAP-21/22			
TEST SITE I - KAMARJANI - ADAPTAION WORKS -			
GROYNE TYPE G - 1 PILE LAYOUT PLAN			
NAME	DATE	SCALE	
DESIGNED			
CHECKED			
APPROVED			
DRAWING NO			REVISION
AD-KA-011			0



ELEVATION

Notes:

1. All measurements in meter.
2. All levels refer to $\pm 0.0m$ PWD
3. S L W = Standard Low Water
D H W = Design High Water

PLAN OF G2

ADAPTATION 97/98						
PILE INSTALLATION SCHEDULE						
PILE TYPE	LOCATION No.	Nos.	PILE HEAD		PILE POINT LEVEL m PWD	TOTAL PILE LENGTH (m)
			TOP LEVEL AFTER INSTALLATION m PWD			
BORED PILE φ 750 mm	A 1 - A7 B 1 - B5	12	+22.90		- 1.10	24.00
	C 1 - C2	2				

GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH
MINISTRY OF WATER RESOURCES
WATER RESOURCES PLANNING ORGANIZATION (WARPO)

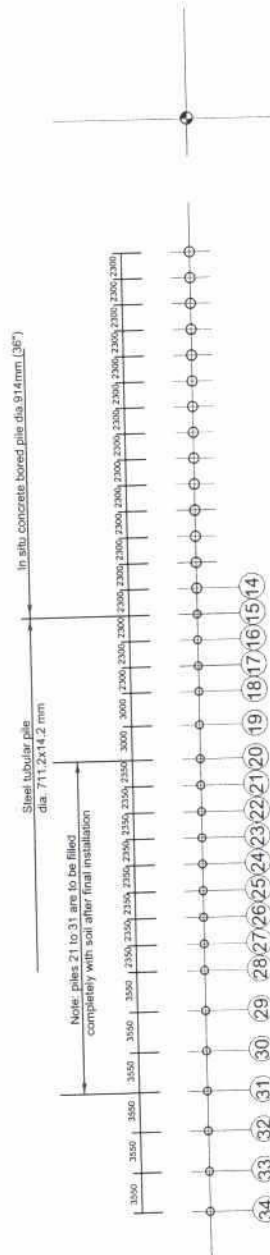
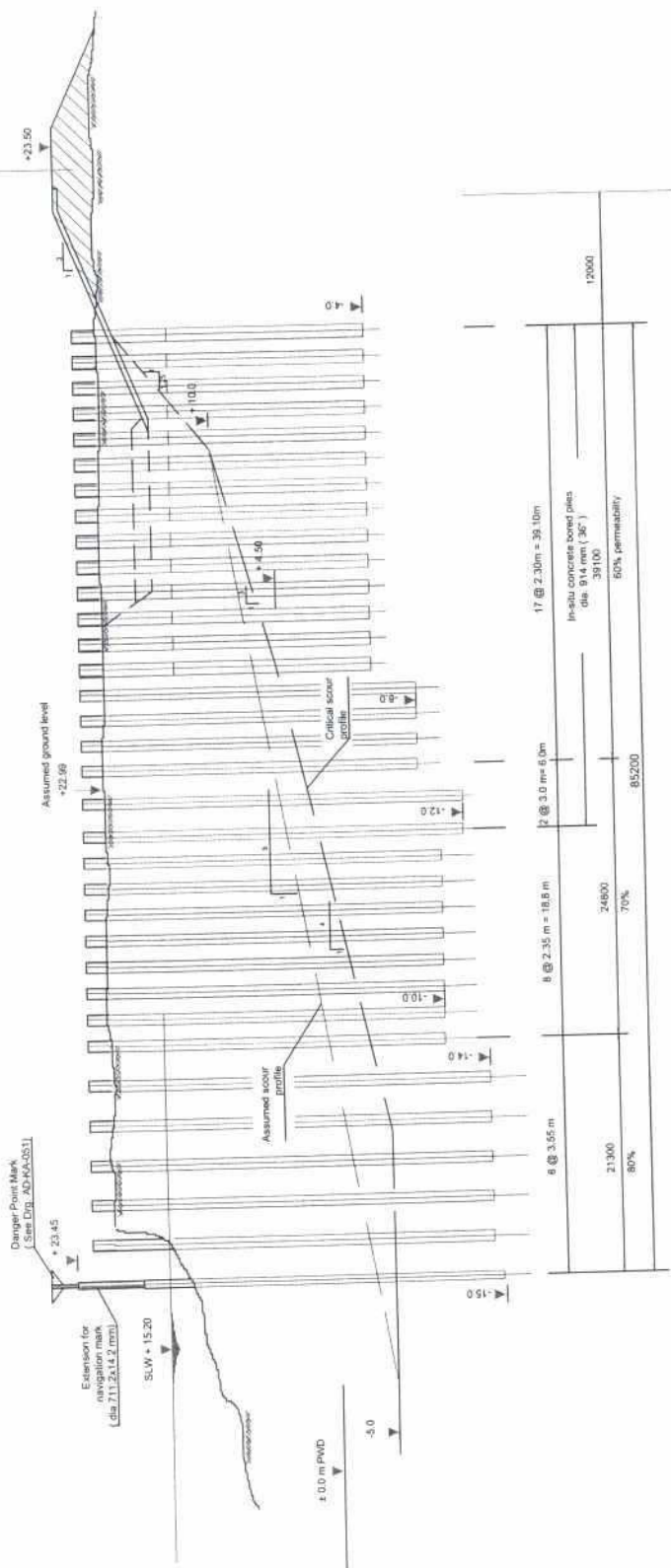
BANK PROTECTION PILOT PROJECT FAP - 21

JAMUNA TEST WORKS CONSULTANTS, JOINT VENTURE
CONSULTING CONSORTIUM FAP-21/22

TEST SITE I - KAMARJANI
- ADAPTAION WORKS -

GROYNE TYPE G-2
PILE LAYOUT PLAN

	NAME	DATE	SCALE	
	DESIGNER	18-10-2008		
	CHECKED		DRAWING NO	REVISION
	APPROVED		AD-KA-021	0



Notes:
 1. All measurements in millimeter.
 2. All levels refer to ± 0.00 PWD.
 3. Reference Drawings:
 AD-KA-001 Adaptation of Test Structures
 General Project Layout
 AD-KA-050 General Layout Plan
 AD-KA-052 Steel Pile Dia 711.02 x 140.2mm
 Pile Schedule.

600



GOVERNMENT OF THE PEOPLE'S REPUBLIC OF BANGLADESH MINISTRY OF WATER RESOURCES WATER RESOURCES PLANNING ORGANIZATION (WARPO)			
BANK PROTECTION PILOT PROJECT FAP - 21			
JAHANA TEST WORKS CONSULTANTS JOINT VENTURE CONSULTING CONSORTIUM FAP-21/22			
TEST SITE I - KAMARJANI - ADAPTATION WORKS -			
GROYNE G - A/2 GENERAL ARRANGEMENT OF PLAN, ELEVATION			
SCALE			
NAME	DATE	DRAWING NO	REVISION
ANOWAR	21-03-2001	AD-KA-051	0
CHECKED			
APPROVED			

