DURBAN HARBOUR ENTRANCE

WIDENING AND DEEPENING PROJECT

APPENDICES

MASTER PROJECT CT4061, GROUP CF44

FINAL REPORT, AUGUST 2005

PRDW

Faculty of Civil Engineering and Geosciences

Delft University of Technology

APPENDICES

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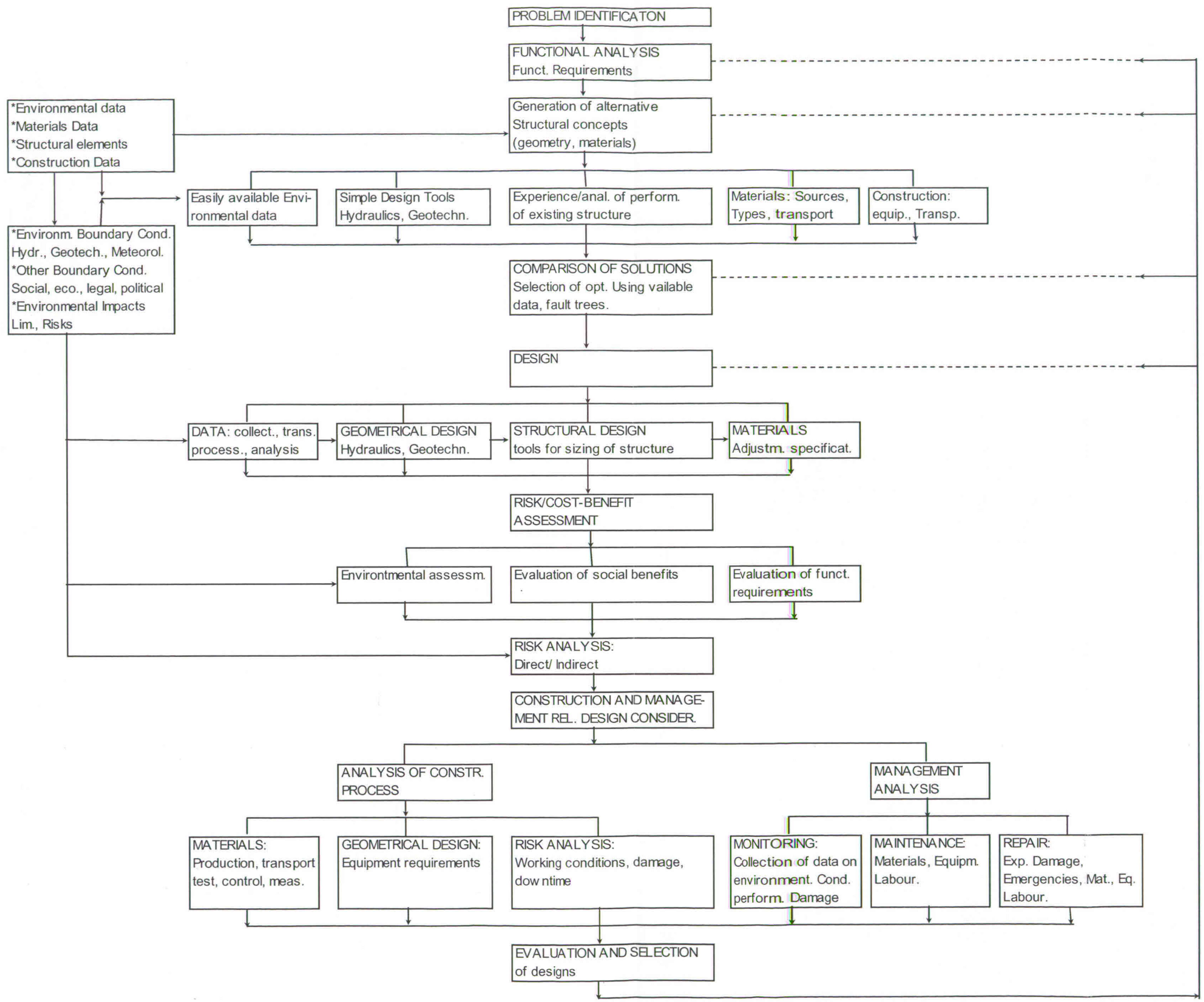
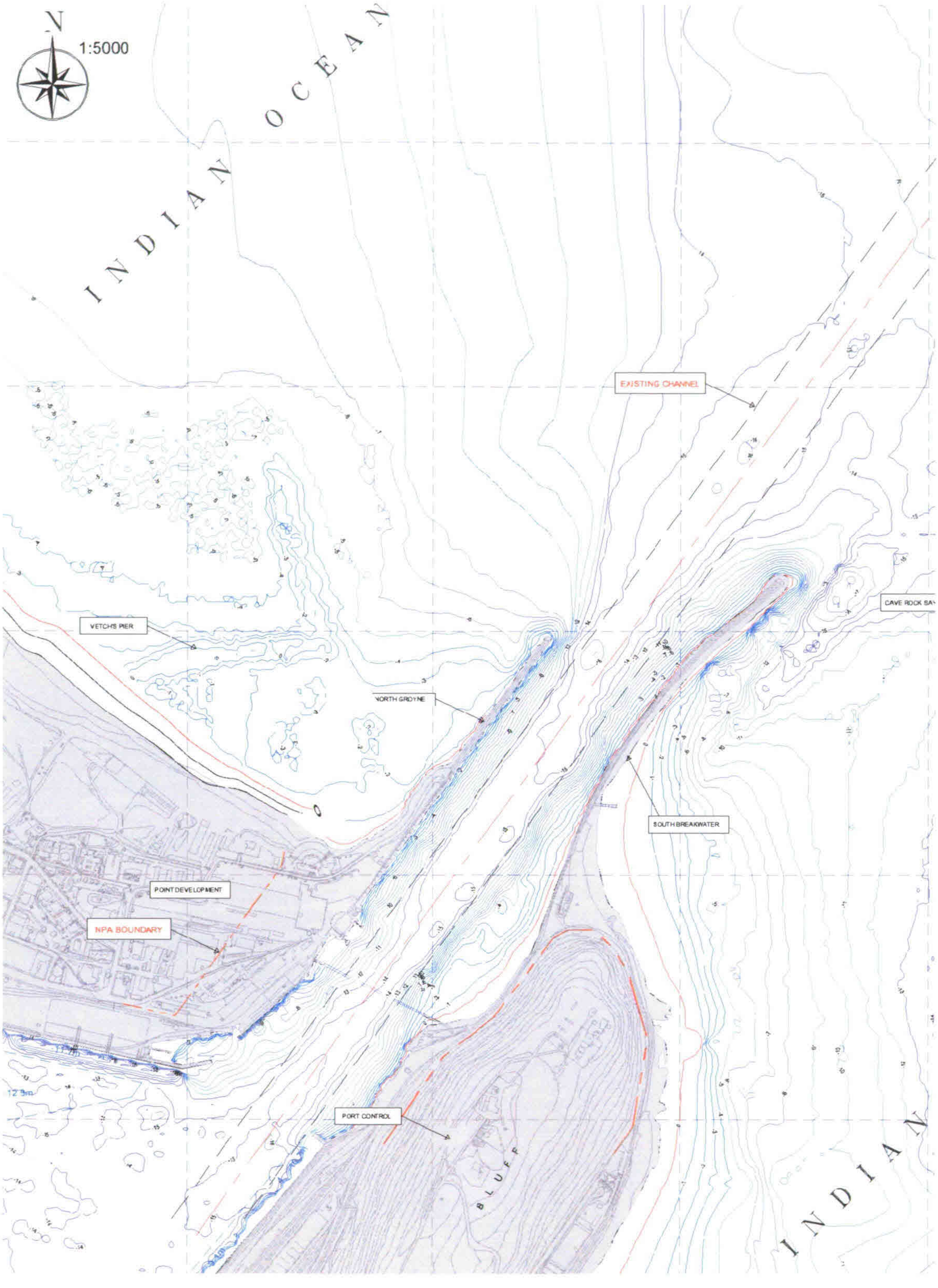
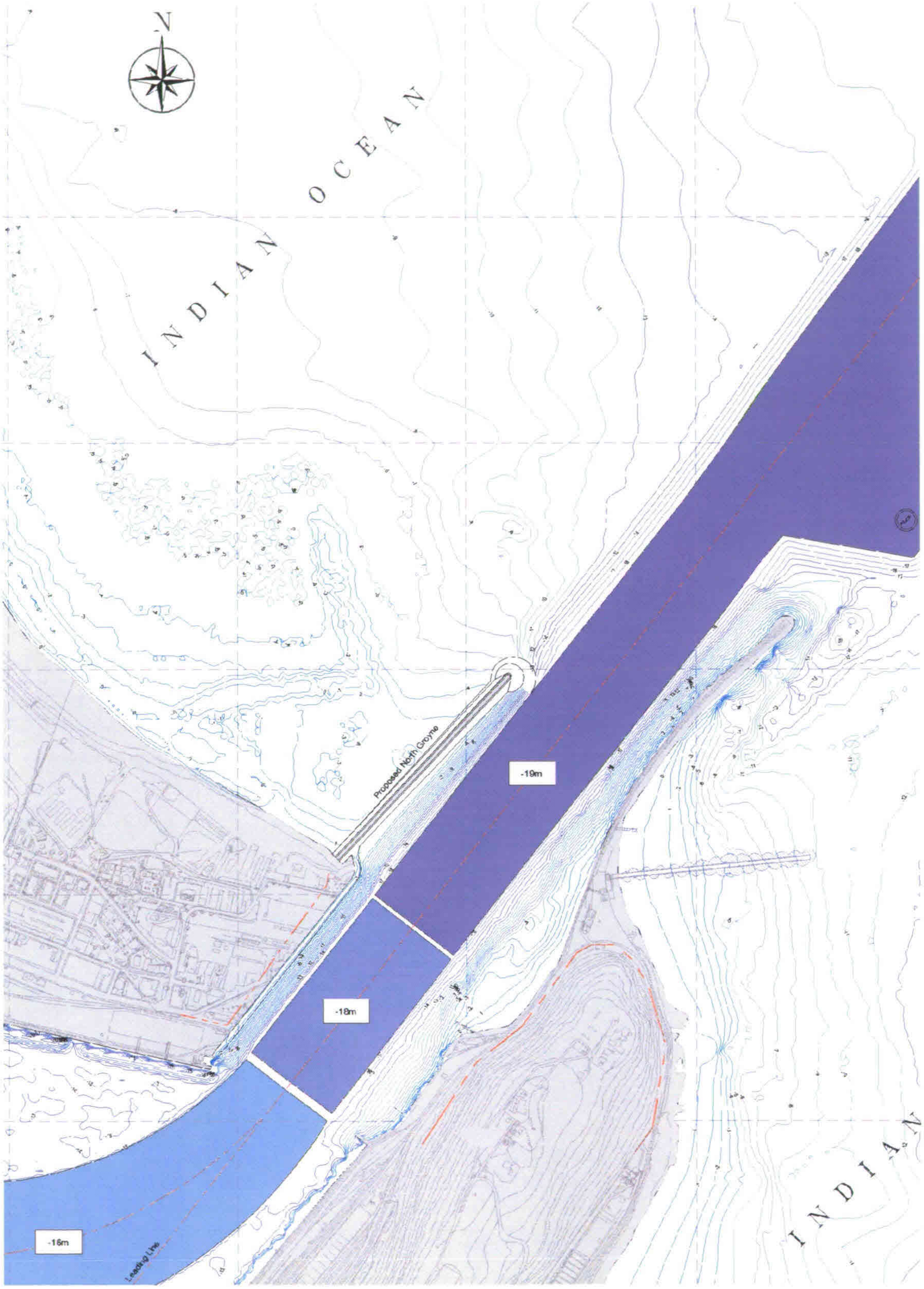

PROBLEM IDENTIFICATON
FUNCTIONAL ANALYSIS
Funct. Requirements
Generation of alternative
Structural concepts
(geometry, materials)
*Environmental data
*Materials Data
*Structural elements
*Construction Data
Easily available Envi-
ronmental data
Simple Design Tools
Hydraulics, Geotechn.
Experience/anal. of perform.
of existing structure
Materials: Sources,
Types, transport
Construction:
equip., Transp.
*Environm. Boundary Cond.
Hydr., Geotech., Meteorol.
*Other Boundary Cond.
Social, eco., legal, political
*Environmental Impacts
Lim., Risks
COMPARISON OF SOLUTIONS
Selection of opt. Using vailable
data, fault trees.
DESIGN
DATA: collect., trans.
process., analysis
GEOMETRICAL DESIGN
Hydraulics, Geotechn.
STRUCTURAL DESIGN
tools for sizing of structure
MATERIALS
Adjustm. specificat.
RISK/COST-BENEFIT
ASSESSMENT
Environtmental assessm.
Evaluation of social benefits
Evaluation of funct.
requirements
RISK ANALYSIS:
Direct/ Indirect
CONSTRUCTION AND MANAGE-
MENT REL. DESIGN CONSIDER.
ANALYSIS OF CONSTR.
PROCESS
MANAGEMENT
ANALYSIS
MATERIALS:
Production, transport
test, control, meas.
GEOMETRICAL DESIGN:
Equipment requirements
RISK ANALYSIS:
Working conditions, damage,
down time
MONITORING:
Collection of data on
environment. Cond.
perform. Damage
MAINTENANCE:
Materials, Equipm.
Labour.
REPAIR:
Exp. Damage,
Emergencies, Mat., Eq.
Labour.
EVALUATION AND SELECTION
of designs



APPENDIX C: EXISTING CHANNEL ALIGMENT



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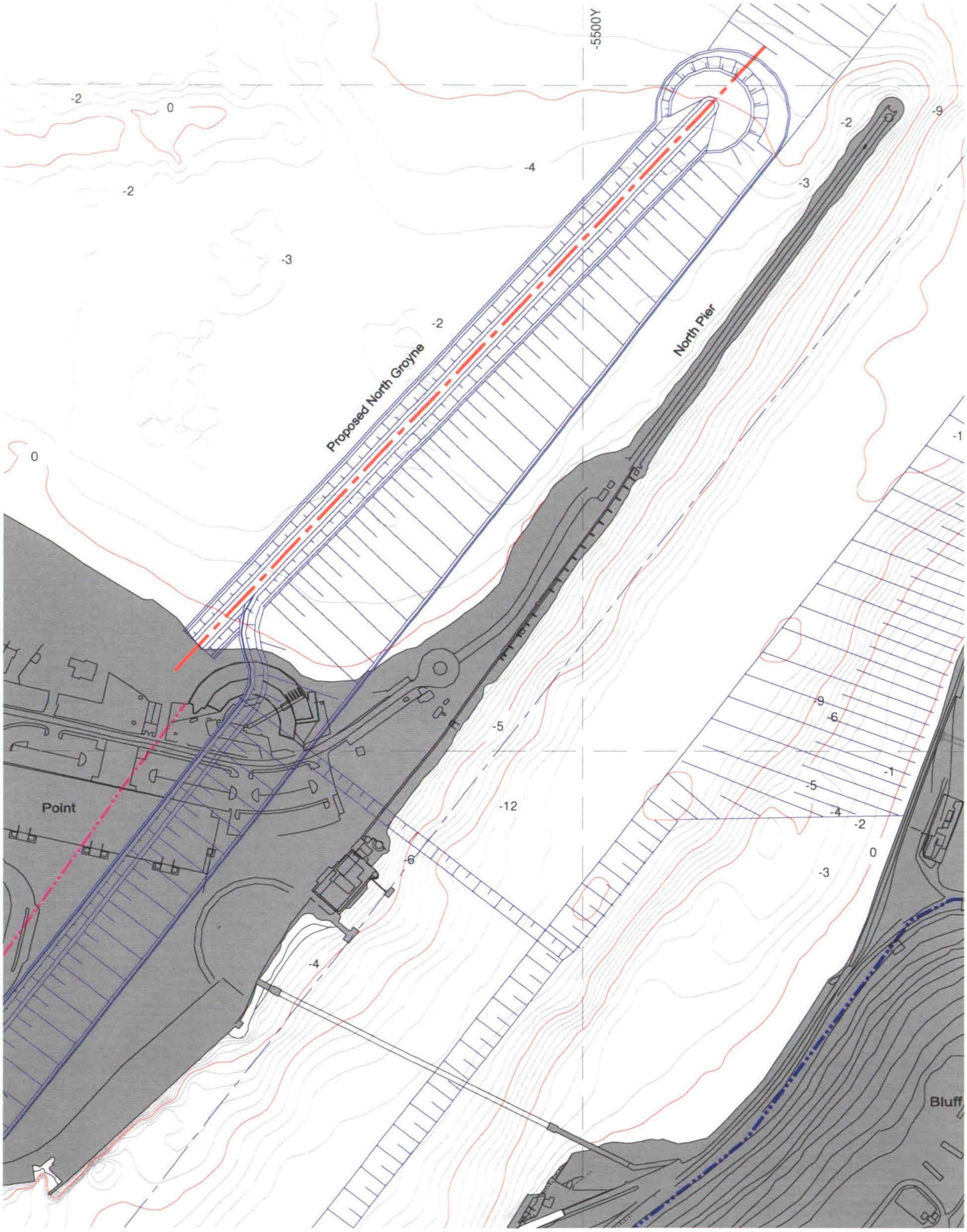
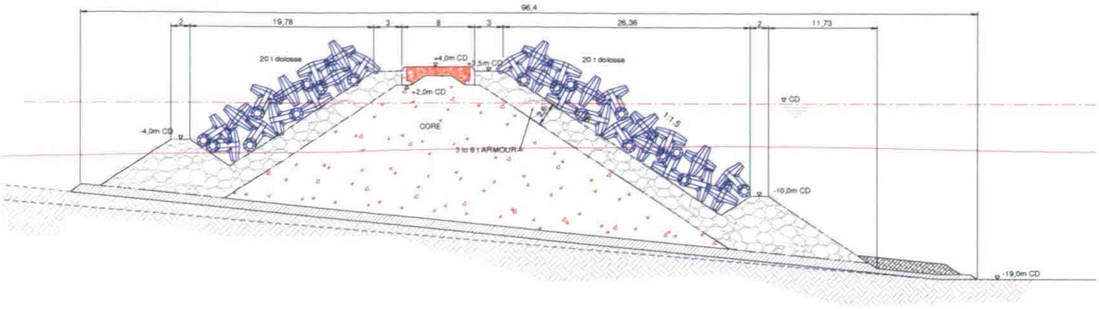
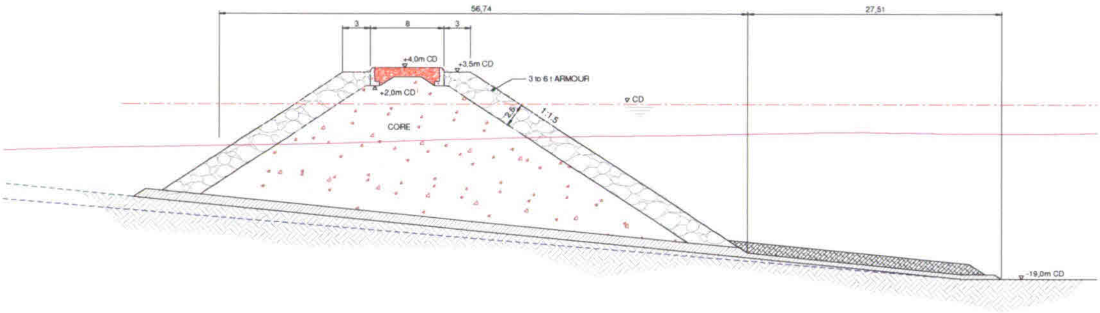


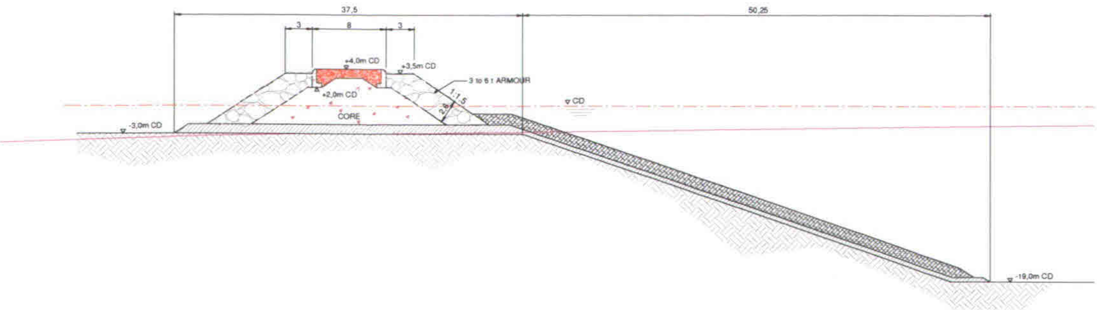
APPENDIX E: PRELIMINARY DESIGN NEW NORTH GROYNE



Head of the groyne

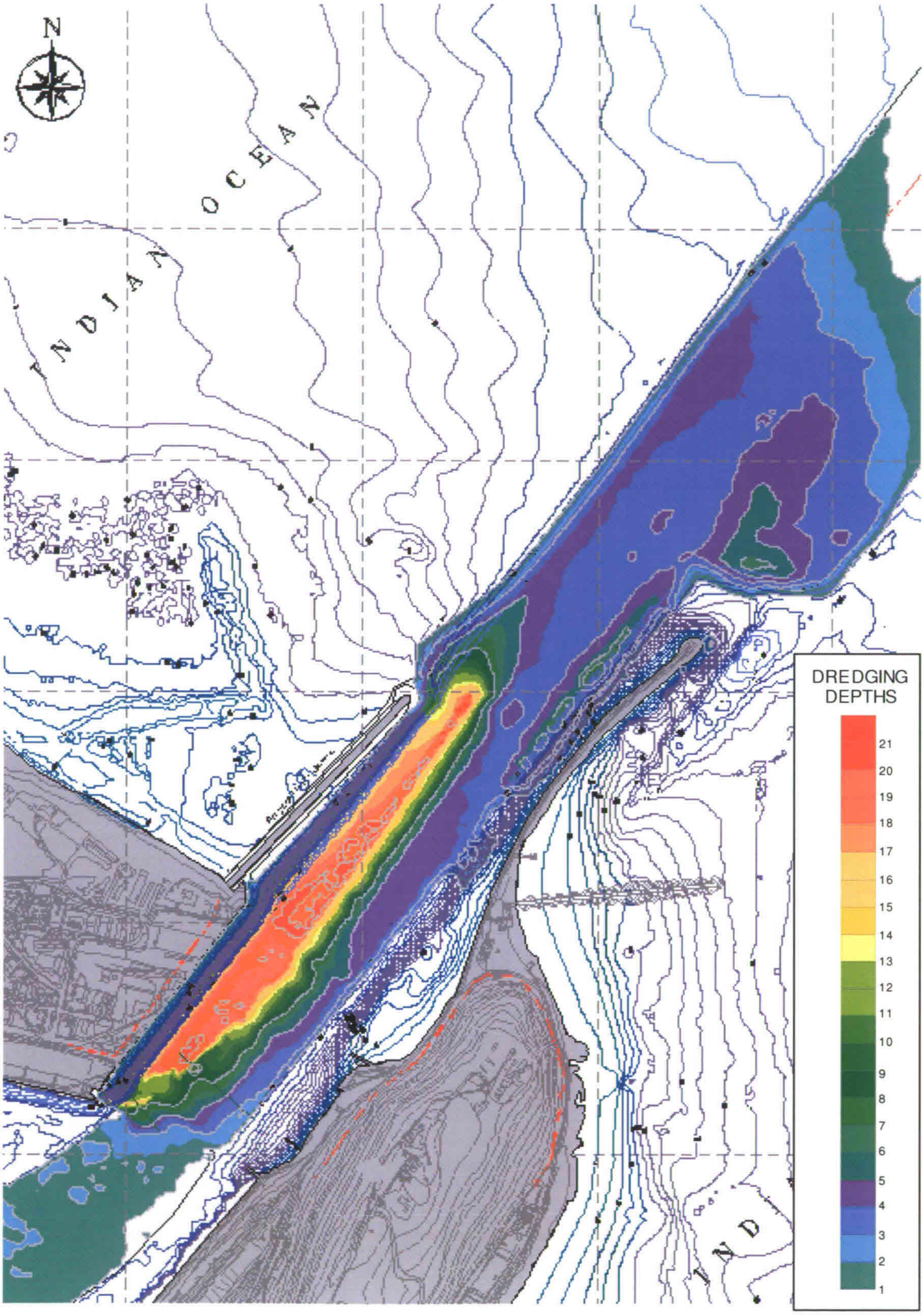


Trunk of the groyne

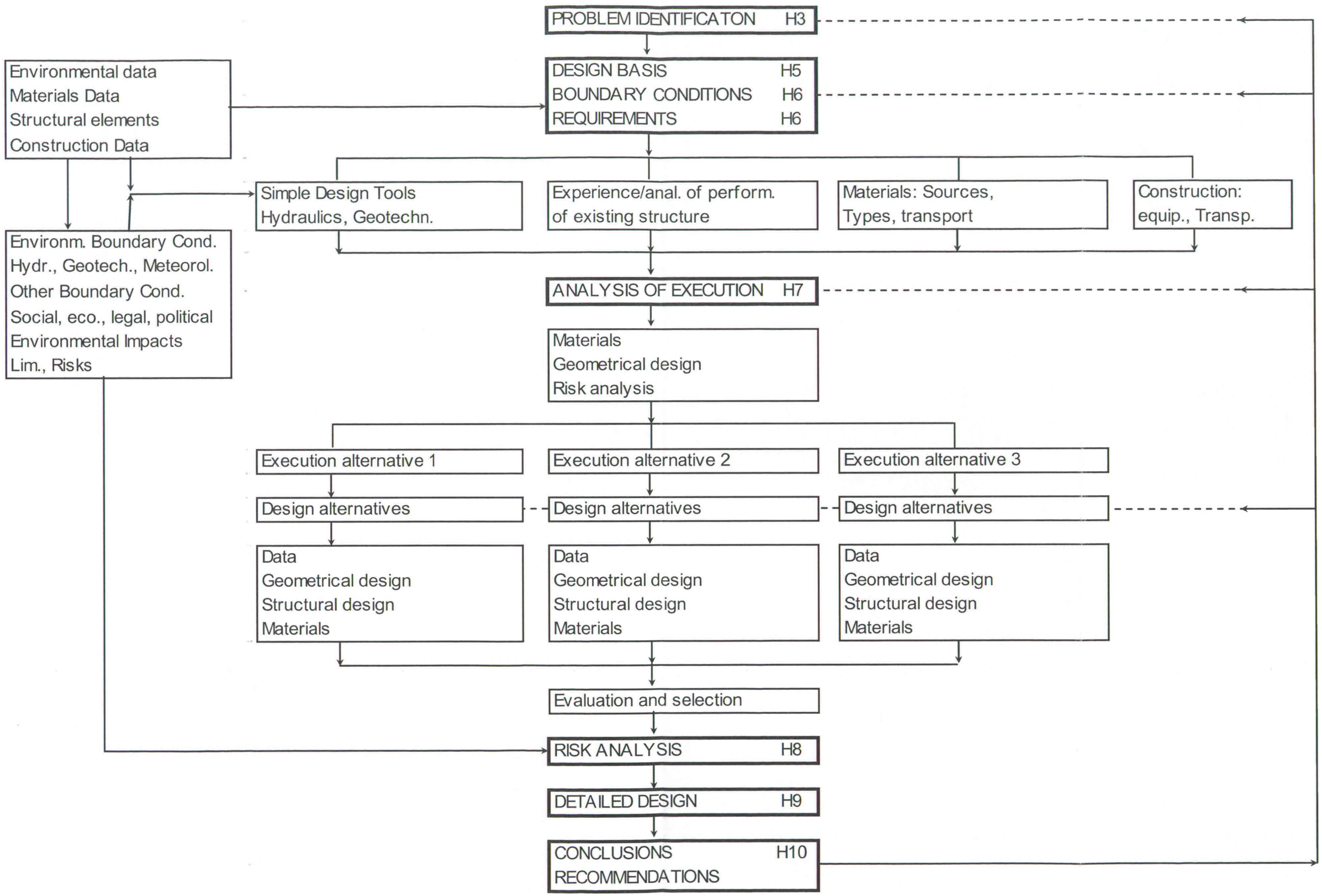


North Revetment

APPENDIX F: TYPICAL SECTIONS PROPOSED NEW NORTH GROYNE



APPENDIX G DREDGING ISOPACHES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Environmental data
Materials Data
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DESIGN BASIS H5
BOUNDARY CONDITIONS H6
REQUIREMENTS H6
Simple Design Tools
Hydraulics, Geotechn.
Experience/anal. of perform.
of existing structure
Materials: Sources,
Types, transport
Construction:
equip., Transp.
Environm. Boundary Cond.
Hydr., Geotech., Meteorol.
Other Boundary Cond.
Social, eco., legal, political
Environmental Impacts
Lim., Risks
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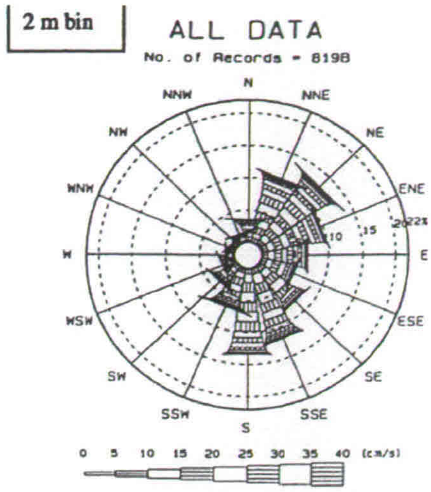
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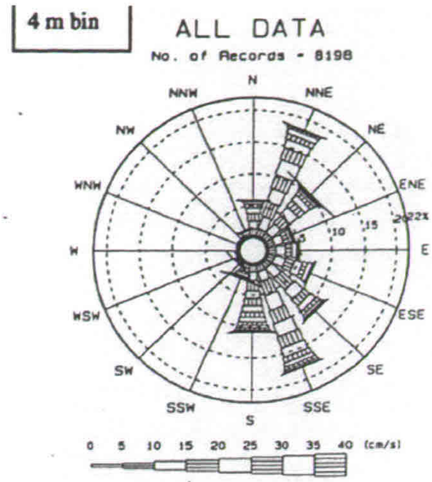
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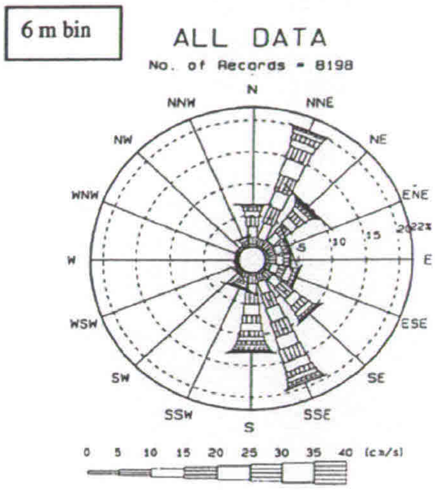
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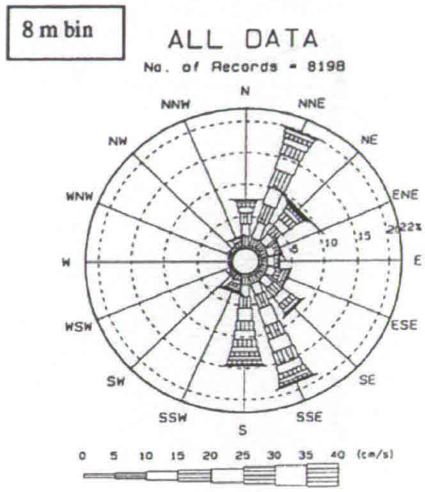
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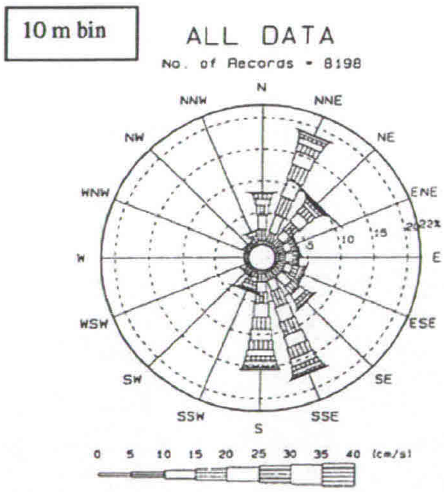
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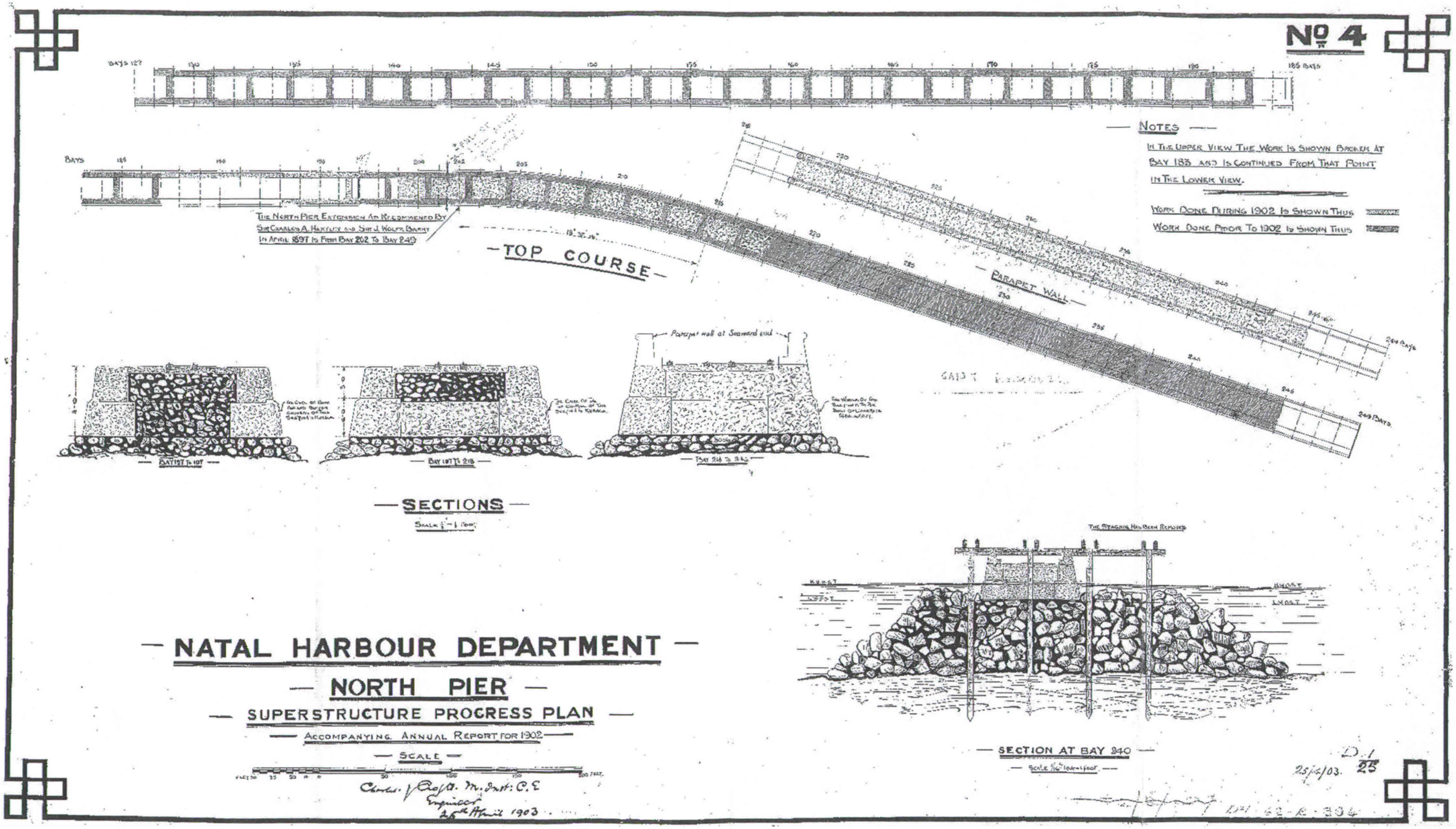
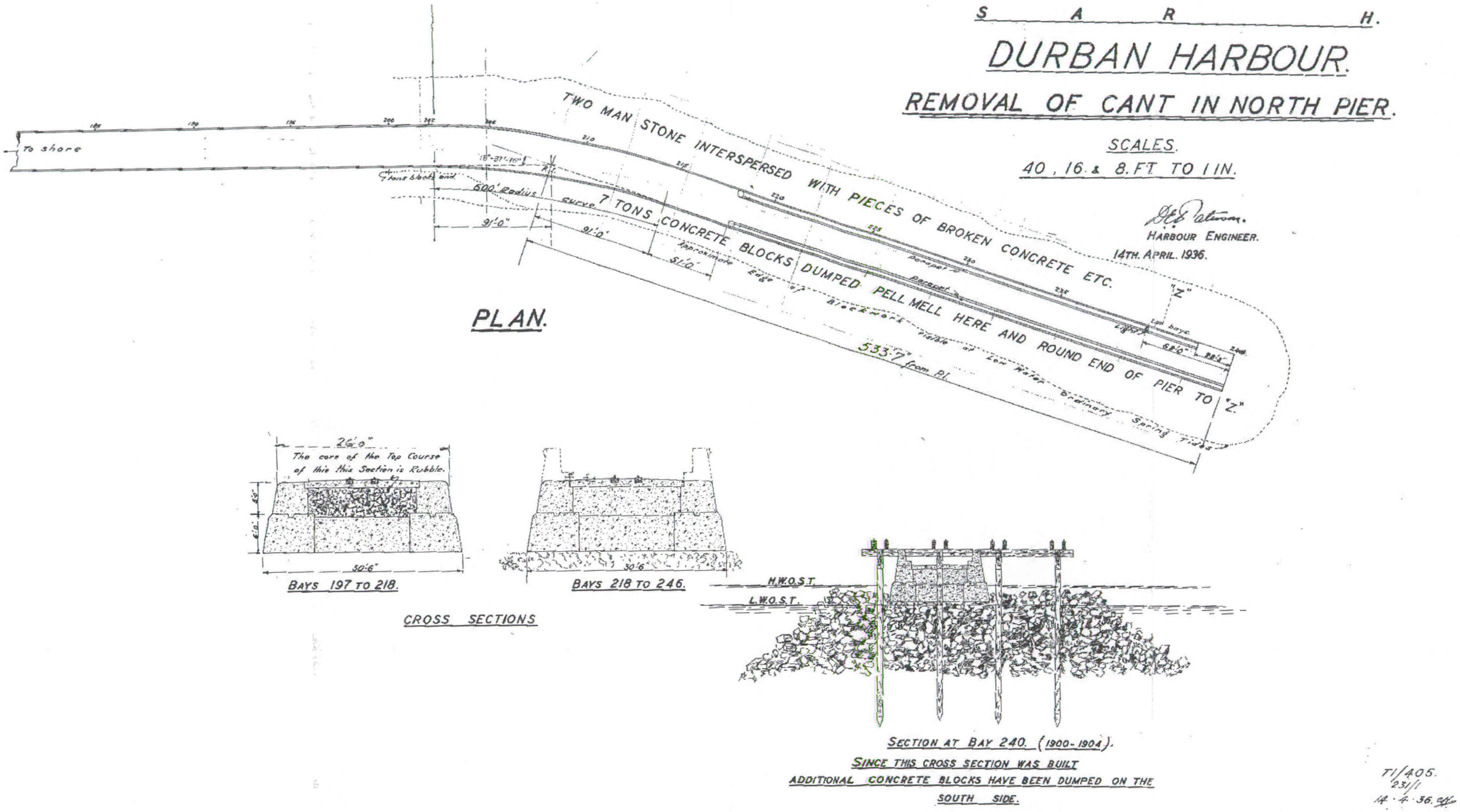


Nº 4
NOTES
IN THE UPPER VIEW THE WORK IS SHOWN BROKEN AT BAY 183 AND IS CONTINUED FROM THAT POINT IN THE LOWER VIEW.
WORK DONE DURING 1902 IS SHOWN THUS
WORK DONE PRIOR TO 1902 IS SHOWN THUS
THE NORTH PIER EXTENSION AS RECOMMENDED BY SIR CHARLES A. HARTLEY AND SIR J. WOLFE BARRY IN APRIL 1897 IS FROM BAY 202 TO BAY 249
TOP COURSE
PARAPET WALL
Parapet wall at Seaward end
SECTIONS
NATAL HARBOUR DEPARTMENT
NORTH PIER
SUPERSTRUCTURE PROGRESS PLAN
ACCOMPANYING ANNUAL REPORT FOR 1902
SCALE
SECTION AT BAY 240
THE STAGING HAS BEEN REMOVED
25th April 1903


S A R H.
DURBAN HARBOUR.
REMOVAL OF CANT IN NORTH PIER.
SCALES.
40, 16 & 8 FT TO 1 IN.
HARBOUR ENGINEER.
14TH. APRIL. 1936.
To shore
TWO MAN STONE INTERSPERSED WITH PIECES OF BROKEN CONCRETE ETC.
7 TONS CONCRETE BLOCKS DUMPED PELL MELL HERE AND ROUND END OF PIER TO "Z"
"Z"
600' Radius curve
91'0"
51'0"
533'7" from P.I.
PLAN.
26'0"
The core of the Top Course of this Section is Rubble.
30'6"
BAYS 197 TO 218.
BAYS 218 TO 246.
CROSS SECTIONS
H.W.O.S.T.
L.W.O.S.T.
SECTION AT BAY 240. (1900-1904).
SINCE THIS CROSS SECTION WAS BUILT ADDITIONAL CONCRETE BLOCKS HAVE BEEN DUMPED ON THE SOUTH SIDE.
T1/405.
23/1
14.4.36.

WAVE DIRECTION ANALYSIS

Introduction

The purpose of this appendix is to evaluate the effect of the wave climate on the construction method. An analysis will be made to determine in which season the largest waves occur. Furthermore an analysis of the expected will be made. As the trunk of the New North Groyne will be unprotected for a long period of time in certain construction alternatives it is interesting to know the chance of damage to this less protected structure. Finally the downtime of the dredging equipment during construction will be considered.

Seasonal analysis

The data of the ADCP for the period of January 2002 to December 2003 is used. In this data every 2 hours the significant wave height is determined. CSIR model studies (ref.I8) have determined that, while the biggest wave come from the this direction, the south western waves have less impact on the North Groyne due to the protection offered by the South Breakwater. In this analysis only the wave from the north western and north eastern direction (between 80° and 300°) are considered. The wave data is divided per season and plotted in table A 1and figure A 1.

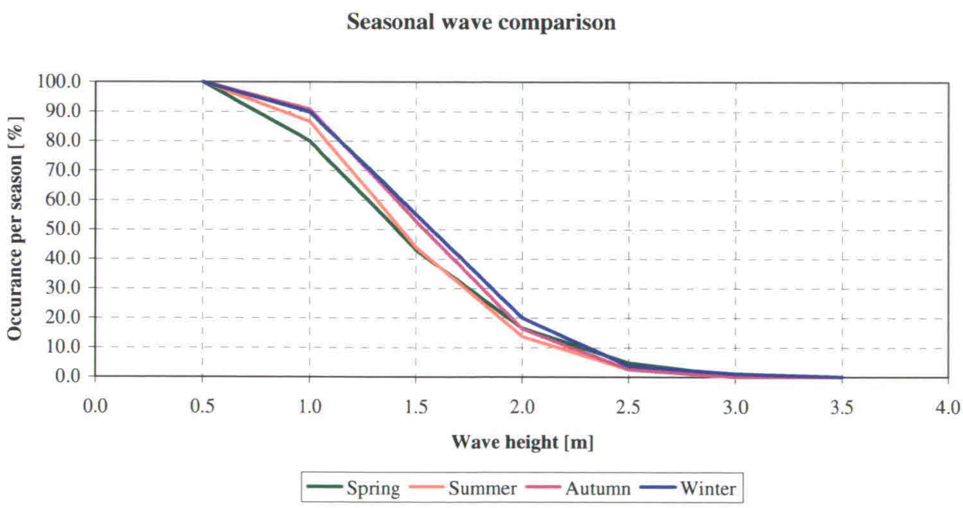


figure A 1: wave height comparison per season;

Wave height	Occurrence in spring	Occurrence in summer	Occurrence in autumn	Occurrence in winter
>0.5 m	100 %	100 %	100 %	100 %
>1.0 m	80 %	86 %	90 %	90 %
>1.5 m	43 %	44 %	53 %	55 %
>2.0 m	16 %	14 %	16 %	20 %
>2.5 m	5 %	3 %	2 %	4 %
>3.0 m	0.5 %	0 %	0 %	1 %
>3.5 m	0 %	0 %	0 %	0 %

table A 1: wave height occurrence per season;

It can be seen that in the maximum wave height occurring is approximately the same in each season. In spring and summer approximately 45% of the waves are higher than 1.5 m while in autumn and winter this value is approximately 55%. The best construction period is therefore determined to be the spring and summer period.

Damage and downtime

For the preliminary damage and downtime calculations it is assumed that the wave height occurrence as shown in table A 2 (TR 8) is also valid for the partially completed New North Groyne.

Return Interval	North Groyne
1	2.5
5	2.9
10	3.0
20	3.2
50	3.3
100	3.5

table A 2: single omni-directional wave heights: North Groyne;

It is not yet known for which wave height unacceptable damage will occur to the partially constructed New North Groyne. The chance of occurrence is therefore calculated for wave heights of 1.5 m, 2.0 m and 2.5 m. The results are shown in table A 3.

Wave Height	Occurrences per year
1.5	130
2.0	12
2.5	1

table A 3: wave height occurrences;

It should be noted that table A 3 is only valid at the head of the New North Groyne, for unprotected trunk parts in the shallower water close to the land; less high waves can be expected. Furthermore the extent of the damage, caused by the occurrence of a wave larger than the design wave of the unprotected trunk, is not known. The displacement of several rocks might be acceptable during construction although scattered rocks might cause problems during dredging. It can be concluded that long periods (months to years) of an unprotected trunk head, especially close to the proposed groyne head, will cause unacceptable loss of parts of the trunk. Placing a temporary extra armour layer at the exposed head might prevent this damage. It is assumed the dredging equipment will not be operational when waves of more than 1.5 m occur. In table A 3 it can be seen that 1.5 m waves occur approximately 130 times a year. Some downtime of dredging equipment will therefore be expected.

SURF ZONE EXISTING NORTH GROYNE

A rough estimation will be made of the length of the surf zone next to the existing North Groyne. This length is needed to determine which part of the existing North Groyne can be removed and which part cannot be removed to maintain its function to prevent the ingress of sand into the channel. In this case a simple relation will be used to determine the breaking depth,

$$H_s \approx 0.5h,$$

Where,

H_s	Significant wave height	[m],
h	Water depth	[m].

From the bathymetry, given in chapter 5.2, the water depth next to the New North Groyne can be approximated as indicated in table A 4. The wave conditions next to the North Groyne are obtained from a study by CSIR. The two significant wave directions are indicated in table A 4. Together with the breaking depth relation, the surf zone can be determined. The length of the surf zone is approximately 330 meters as illustrated in figure A 2.

Groyne length [m]	Depth [m]	H_s, 43° [m]	H_s, 87° [m]
0	0	0	0
200	1.6	1	0.8
250	2	1.2	1.2
300	2.6	1.4	1.2
400	3.2	1.4	1.2
550	4	1.6	1.2
600	6	1.8	1.2

table A 4: wave conditions next to the New North Groyne;

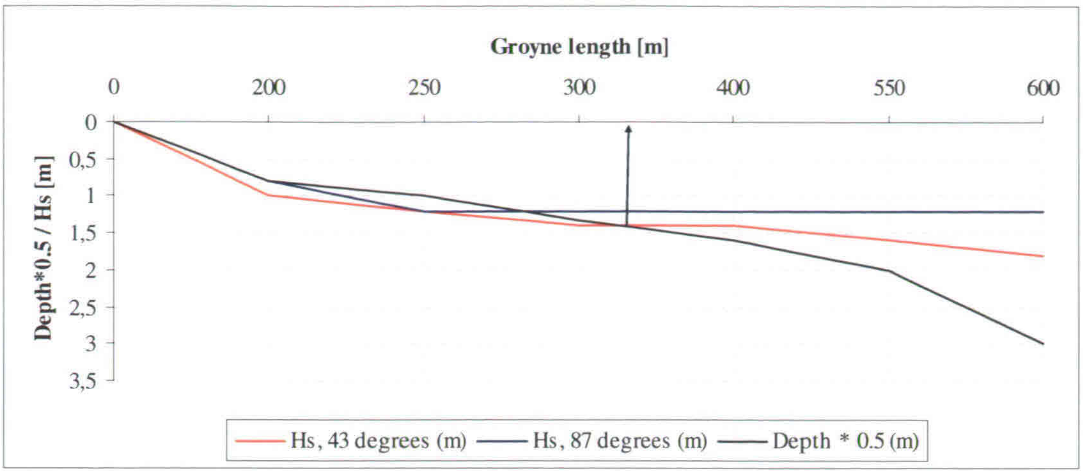


figure A 2: surf zone determination;

TYPICAL SECTIONS EXISTING NORTH GROYNE

For demolishing and recycling of the existing North Groyne it is useful to know the typical cross section of the original construction.

The total length of the North Groyne is approximately 550 meter. The groyne can be divided into two parts. The first part between 0 and 230 meter has an approximated width of 20 meter. The second part of the groyne, between 230 and 550 meter, has a width of 8 meter; this is illustrated in figure A 3. An overview of the existing North Groyne is given in figure A 4.

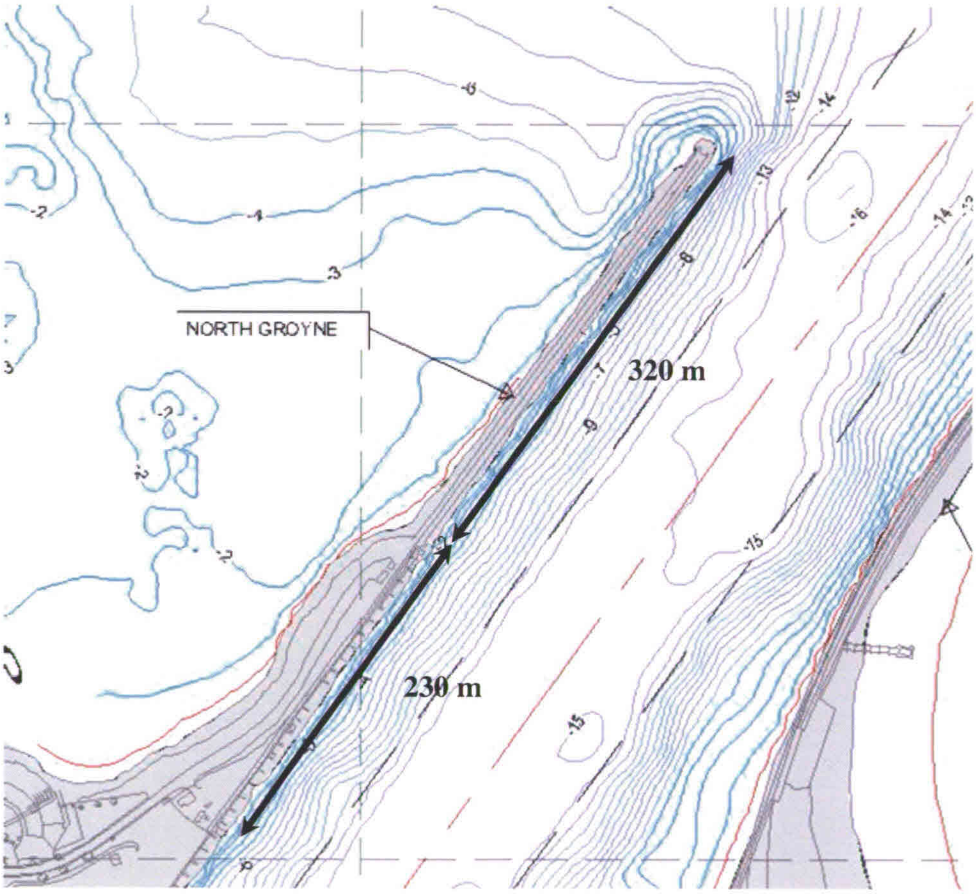


figure A 3: existing North Groyne parts;

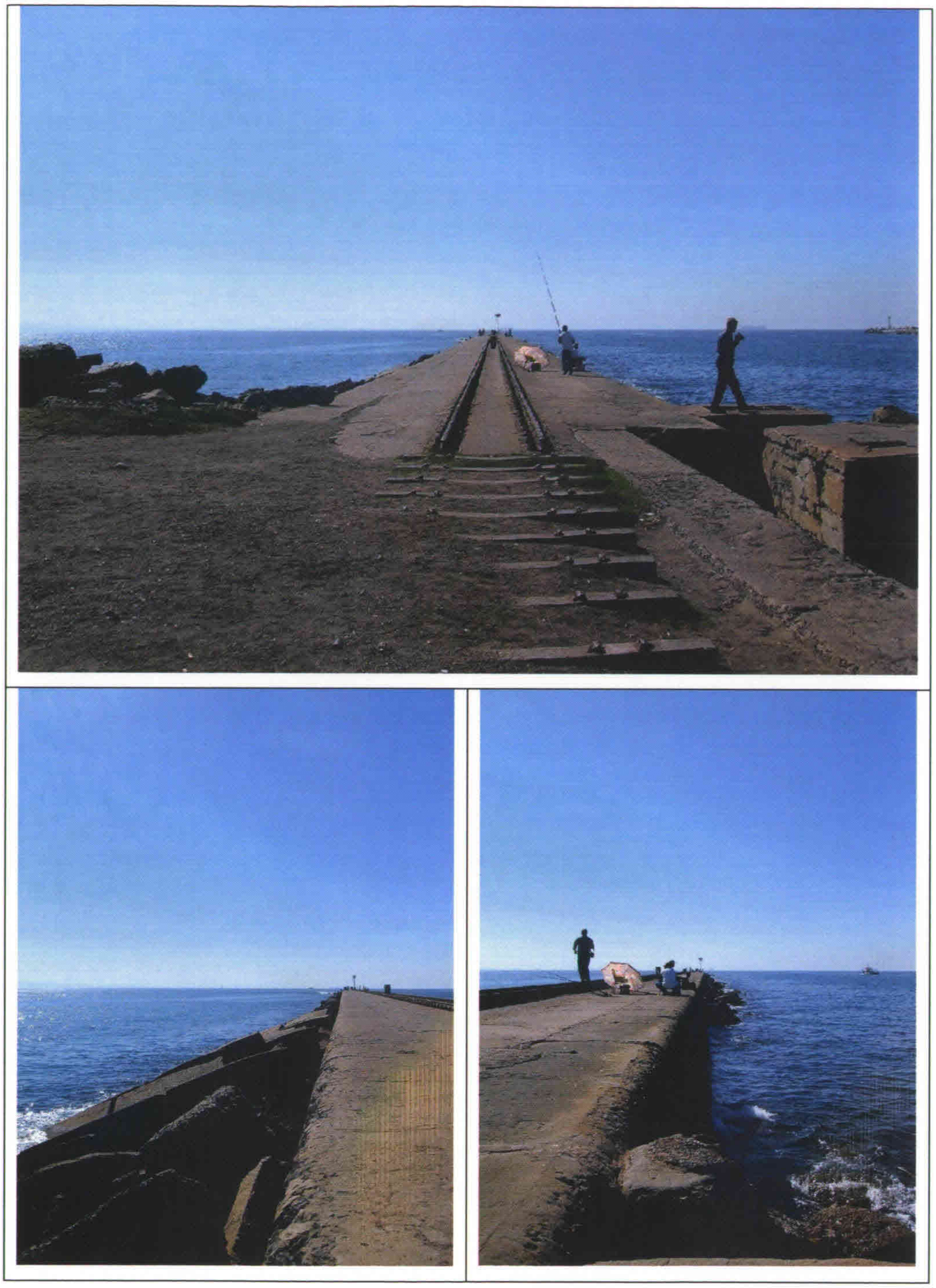


figure A 4: overview existing North Groyne;

From the historical drawings (APPENDICES J and K), the cross-section of this part of the groyne can be determined. The cross-section between bay 127 and 197, which is the final part of the groyne, is illustrated in figure A 5. The length of this part of the groyne is 320 meter; the length of 5 bays is approximately 23 meter. The height of the top course is 11 feet or 3.4 meter. The core of both top and bottom courses is rubble.

Assumed is the top level of +4.0 m CD, the slope of the foundation is assumed to be 1:2 (A 12). The width of the groyne in this section is approximately 8 meter.

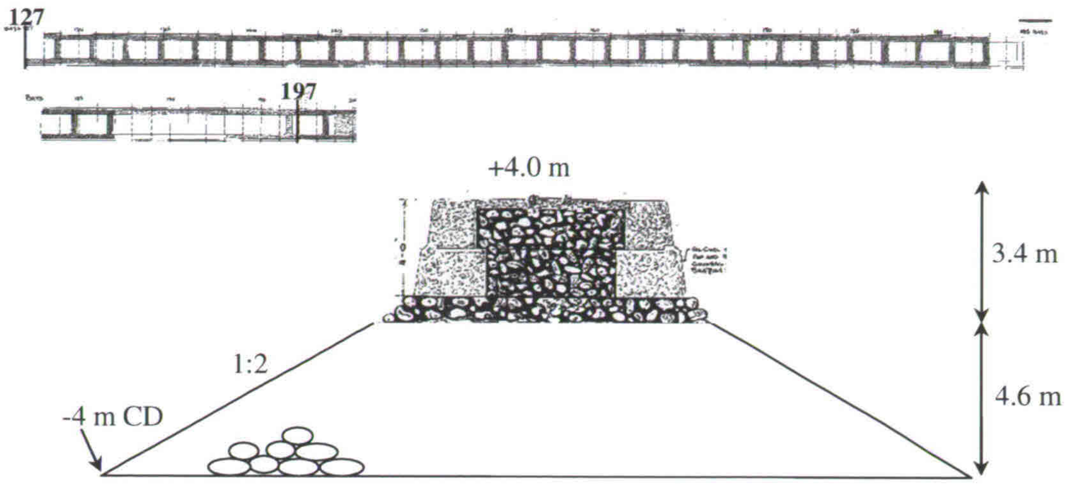


figure A 5: cross-section North Groyne;

MATERIAL VOLUME CALCULATION NORTH GROYNE

In this appendix the volume of the removed material from the existing North Groyne used as core material for the New North Groyne will be calculated. Removal of the existing North Groyne in the surf zone to +1 m CD and for the remaining part to -4 m CD provides the following volume of material (table A 5).

Section	Volume [m^3]
0-230 m	11,000
230-280 m	1,000
280-550 m	28,700
Total	40,700

table A 5: volume of material per section;

The three sections are illustrated in figure A 6.

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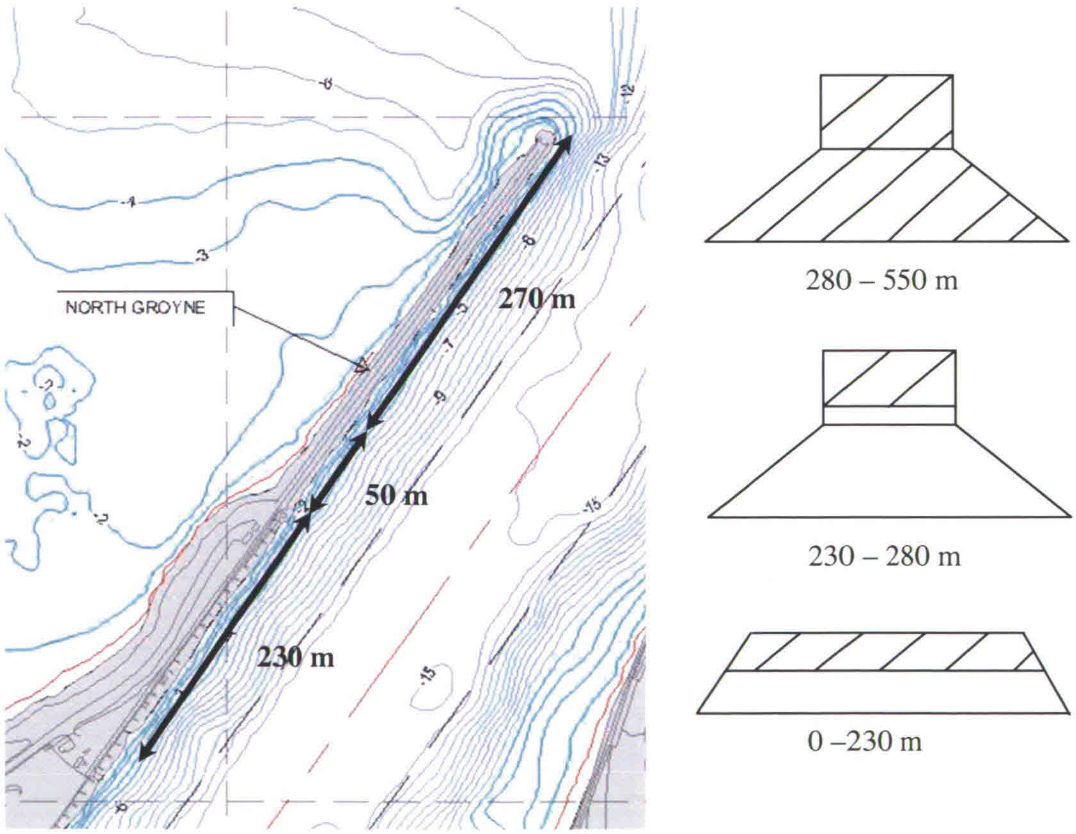


figure A 6: esisting North Groyne sections to be removed;

MATERIAL VOLUME COMPARISON CONSTRUCTION ALTERNATIVES

A rough estimation has been made of the expected volume of material required for the different construction alternatives. A graphical representation of the construction depths of different alternatives is given in figure A 7.

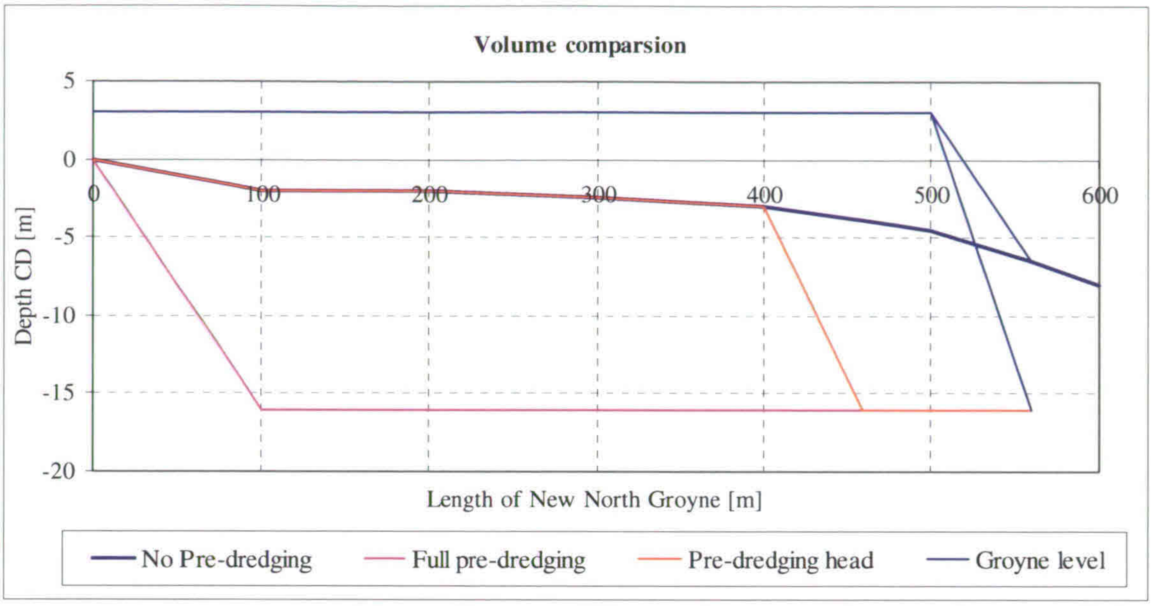
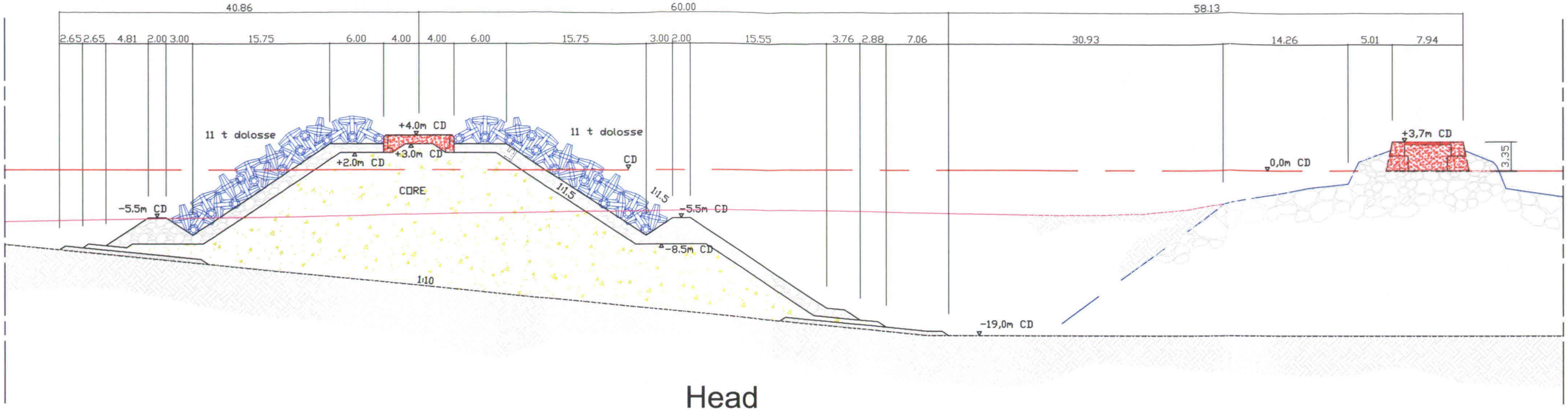


figure A 7: construction depths;

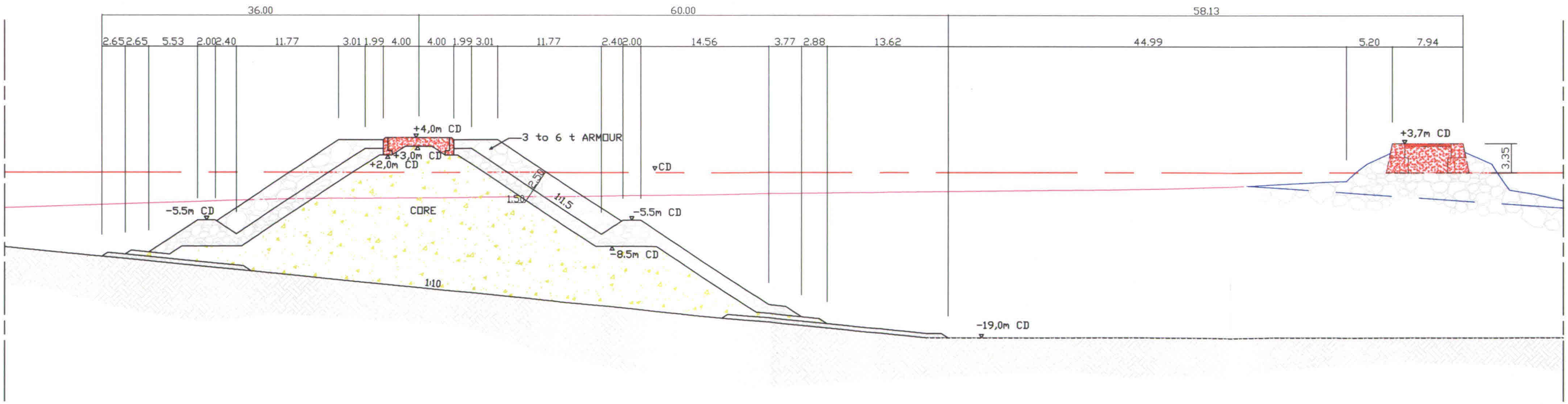
No pre-dredging will be done for alternatives 4 and 5. Pre-dredging of the head will be done for alternatives 2 and 3. Full pre-dredging will be done for alternative 1. This results in the required material volume approximations as represented in table A 6.

Construction method (alternative)	Volume of material required [m^3]
Full pre-dredging (1)	370,000
Pre-dredging head (2&3)	115,000
No pre-dredging (4&5)	65,000

table A 6: required material volume;



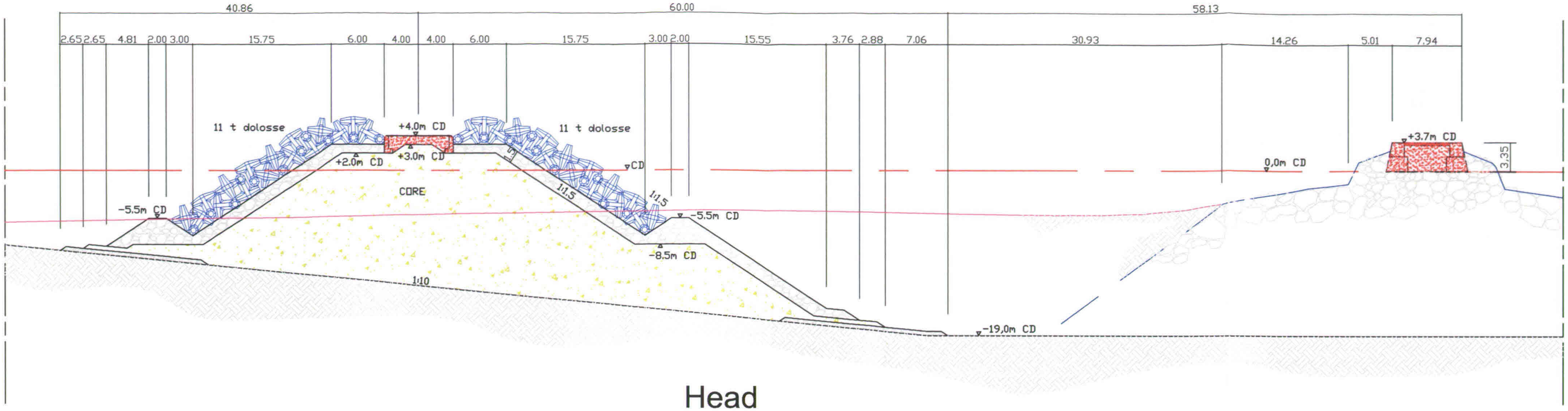
Head



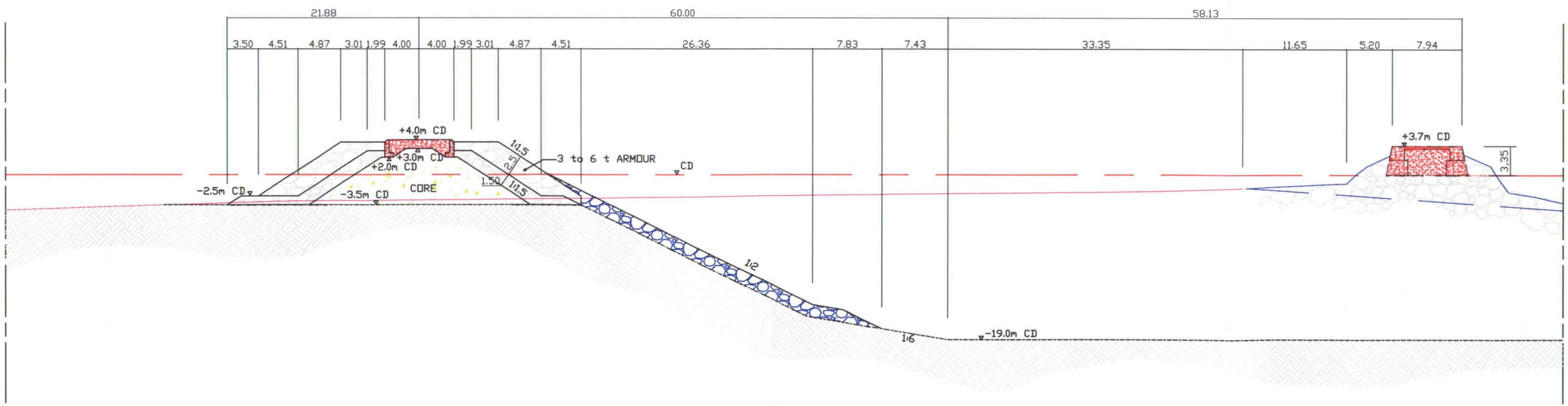
Trunk

3 to 6 ton rock	Quarry run
0.3 to 1 ton rock	Sea gravel
Sand	Concrete

Title: Appendix Q Alternative 1		Scale: 1:500	Format: A3
TUDelft Civil Engineering and Technical Geosciences	PRDW Consulting Coastal, Ocean and Environmental Engineers	GROUP CF44	Date: 11-8-2005 CT4061



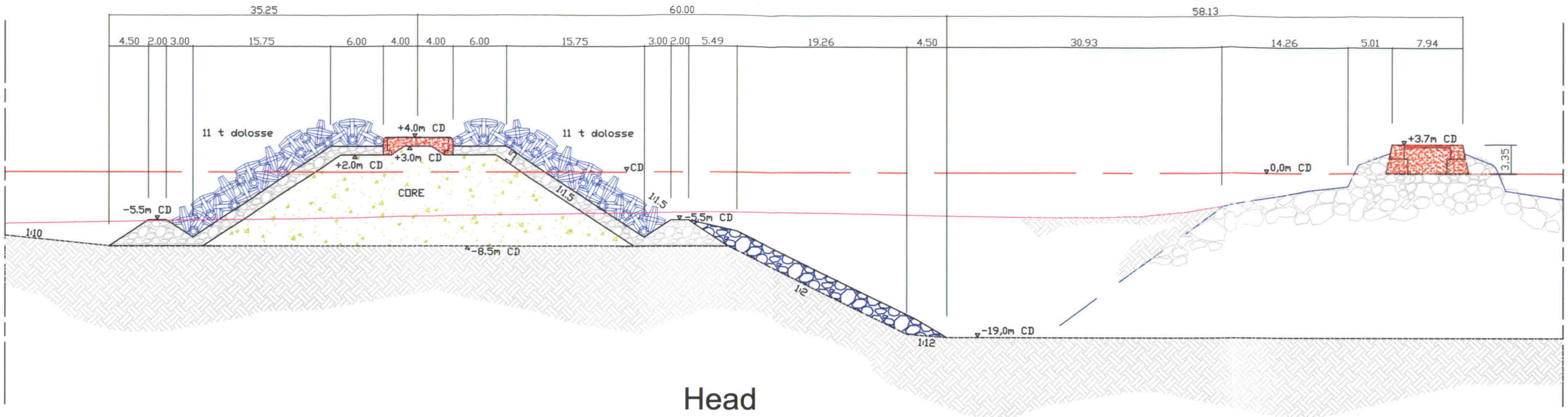
Head



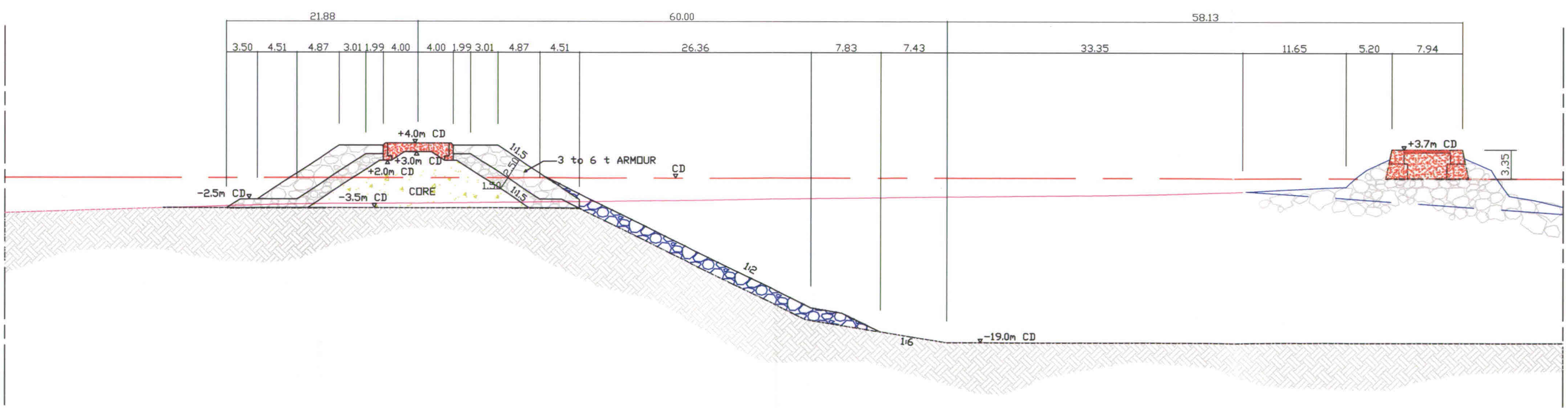
Trunk

3 to 6 ton rock	Quarry run
0.3 to 1 ton rock	Sea gravel
Sand	Falling Apron
Concrete	

Title: Appendix R Alternative 3		Scale: 1:500	Format: A3
TUDelft Civil Engineering and Technical Geosciences	PRDW Consulting Coastal, Ocean and Environmental Engineers	GROUP CF44	Date: 11-8-2005 CT4061



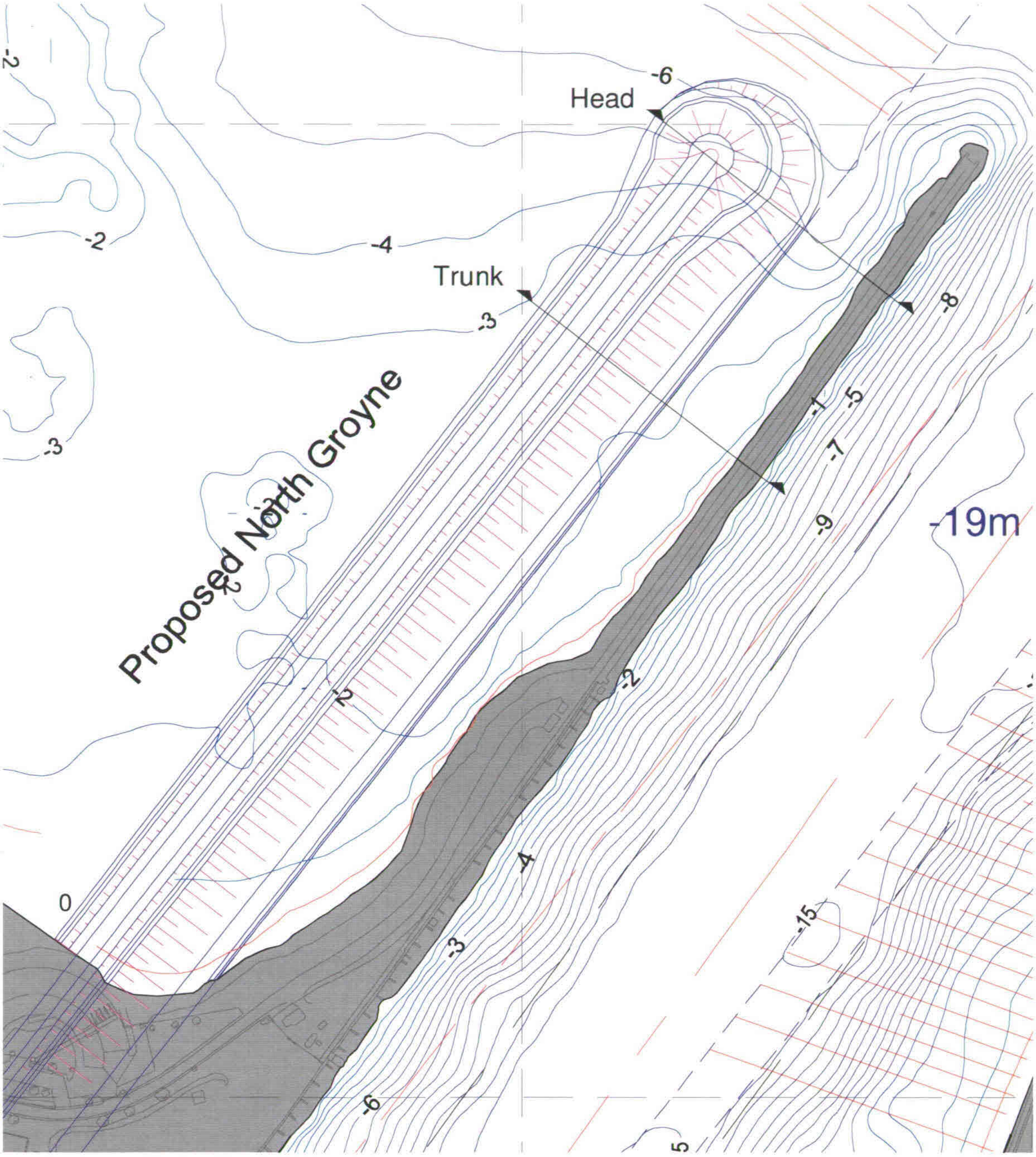
Head



Trunk

3 to 6 ton rock	0.3 to 1 ton rock
Quarry run	Sea gravel
Sand	Concrete
Falling Apron	

Title: Appendix S Alternative 4		Scale: 1:500	Format: A3
TUDelft Civil Engineering and Technical Geosciences	PRDW Consulting Coastal, Ocean and Environmental Engineers	GROUP CF44	Date: 11-8-2005 CT4061



APPENDIX T: OVERVIEW DRAWING ALTERNATIVE 4

SLOPE EQUILIBRIUM CALCULATION

A calculation based on the threshold of motion formula will be executed to determine the equilibrium dredged side slope of the channel. It is assumed that the occurring currents caused by waves, tides and ships govern the final side slope of the channel. With the Shields and Van der Meer equations and the reduction factor for threshold of motion on slopes an estimation of the possible slope is made.

The calculation of a grain diameter to based on an extreme value of velocity is normally done using the following equation:

$$d_{n50} = \frac{\bar{u}_c^{2}}{\Psi_c \Delta C^2 K_\alpha},$$

where,

d_{n50}	Median nominal diameter	[m]
$\bar{u}_c$	Depth averaged velocity	[m/s]
Ψ_c	Shields stability parameter	[-]
Δ	Relative density	[-]
C	Chezy coefficient	[m$^{1/2}$/s]
K_α	Reduction coefficient	[-]

The determination of these parameters is given in Appendix V.

The reduction coefficient K_α is based on the angle of the side slope of the channel and the friction angle of the slope material. Grains lying on a slope will have a lower threshold of movement than grains on a flat bed, see figure A 8. The reduction factor is calculated as the difference between the friction of a flat bed and the friction of bed with slope β:

$$K_\alpha = \frac{F_{friction}(\beta)}{F_{friction}(0)} = \sqrt{1 - \frac{\sin^2 \beta}{\sin^2 \phi}},$$

where,

ϕ	Friction angle of slope material	[°],
β	Slope angle	[°].

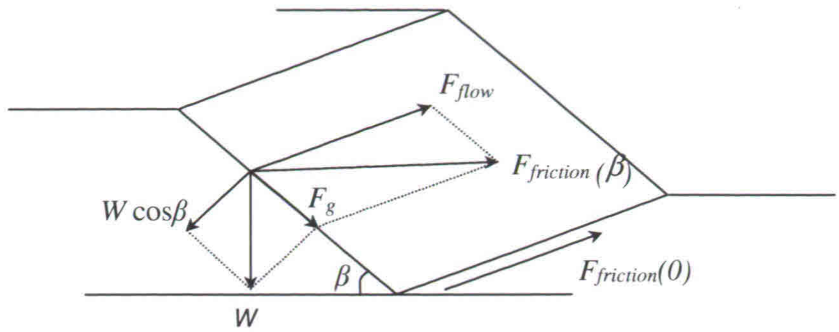


figure A 8: reduction to threshold of motion caused by slope;

With a known flow velocity and grain diameter the maximum reduction factor K_α and thus the maximum slope can be calculated. It is assumed that for a certain value of the Shields stability factor the slope will be stable. For different values of the Shields stability factor the allowable slope has been plotted against the depth averaged velocity in figure A 9.

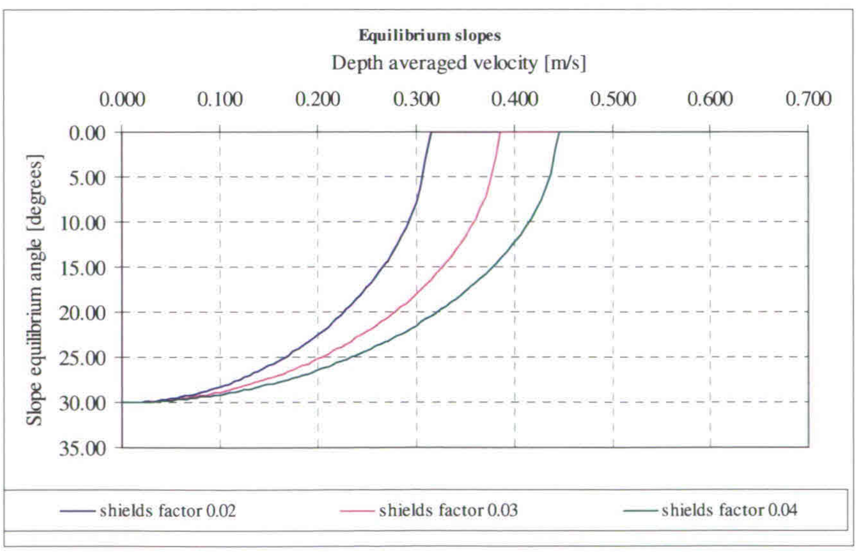


figure A 9: equilibrium slopes for different stability factors;

It can be seen that when the depth-averaged velocity approaches the critical velocity for each Shields factor the slope steepness approaches 1:infinity very quickly. NB: If the depth-averaged velocity is larger than the critical velocity there is even motion on a flat bed. Because the equilibrium slope using this method is highly dependent on the Shields stability factor, it is important to determine the exact factor where the grain movement will be large enough to flatten the slope.

To determine the Shields stability factor with the available data, a few assumptions will be made:

- In the existing situation the slope at the end of the channel is in equilibrium;
- The Shields stability factor required for equilibrium slopes is constant over the entrance channel length;
- The required stability factor of the new situation is the same as the one required for the old situation;
- At the end of the existing and New North Groyne the current at the bottom of the channel is only caused by tidal fluctuations. Penetrating long waves and passing ships have no effect.

From these assumptions it can be concluded that the calculated equilibrium slope is only a rough estimation.

The tidal current in the existing situation is approximately $\bar{u}_{tide}$ = 0.25 m/s. The slope at the end of the existing North Groyne has an angle of 10 degrees. Using above calculation method in reverse, with these parameters and the existing bathymetry, this gives:

$$\Psi_c = \frac{\bar{u}_{tide}^2}{d_{n50}\Delta C^2 K_\alpha} \approx 0.015 .$$

This value for Ψ_c is assumed to be the stability factor belonging to a stable slope in all situations. Using this factor the stable slope in the new situation can be determined. The tidal current $\bar{u}_{tide}$ in the new situation is approximated at $\bar{u}_{tide}$ = 0.13 m/s. The stable slope at the end of the New North Groyne will then be 26 degrees or 1:2. The depth averaged critical velocity is $\bar{u}_c$ = 0.27 m/s. In figure A 10 the curve for the calculated stability factor is shown.

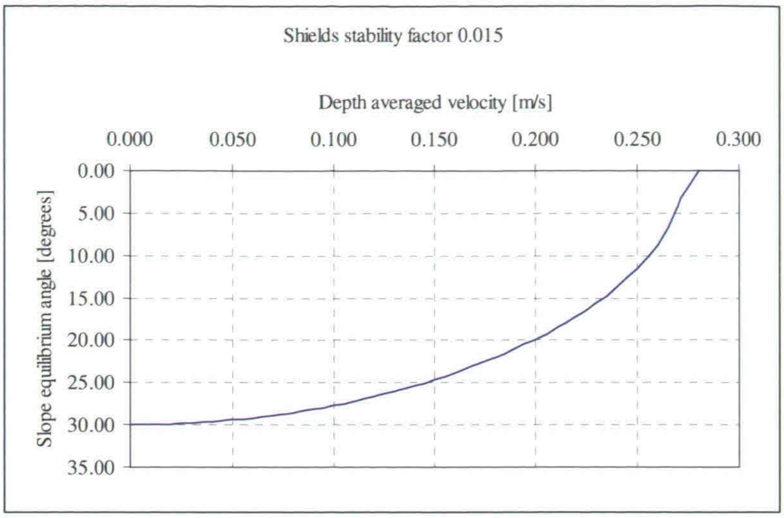


figure A 10: velocity-slope for stability factor 0.015;

To determine the equilibrium slope at the head of the New North Groyne, the current at that location needs to be determined. It is recommended to acquire more current data, both for the new and existing situation, in and outside the channel to validate these values. The ADCP current measurements can be used to calibrate the input parameters of the equations used. Furthermore the model only calculates the threshold of motion at the toe of the slope on the channel depth. Via the Chezy coefficient, the water depth has an influence on the threshold of motion. At the moment this influence is not taken into account but should be considered in final calculations.

At the moment it is considered unsafe to use the calculated value of 26 degrees as the equilibrium slope in the channel. Too many factors are not taken into account for a trustworthy result. It is therefore decided to use the stable slope of the current channel of 10 degrees, or 1:6, for calculation purposes.

SLOPE EQUILIBRIUM CALCULATION ELABORATION.

$$d_{n50} = \frac{\overline{u_c}^{2}}{\Psi_c \Delta C^2 K_\alpha},$$

where,

d_{n50}	Median nominal diameter	[m],
$\overline{u_c}$	Depth-averaged velocity	[m/s],
Ψ_c	Shields stability parameter	[-],
Δ	Relative density	[-],
C	Chezy coefficient	[$m^{1/2}$/s],
K_α	Reduction coefficient	[-].

The median nominal diameter d_{n50} is taken equal to the mean diameter of the Facies A and B materials.

The depth-averaged velocity $\overline{u_c}$ is for purpose of this method assumed as the tidal velocity and calculated with the tidal flow through the channel.

The Shields stability parameter Ψ_c is calculated with the d_* as used by Van Rijn (1984),

$$d_* = d_{n50}\left(\frac{\Delta \cdot g}{v^2}\right)^{\frac{1}{3}},$$

where,

g	Gravitational constant	[m/s],
Δ	Relative density	[-],
	$\Delta = \frac{\gamma_s - \gamma_w}{\gamma_w} = 1.59$	
v	Dynamic viscosity	[m^2/s],
	$v = 10^{-6}(1.14 - 0.031(T-15) + 0.0068(T-15)^2)$	

for $T = 20$ °C $\quad \nu = 1.0 \cdot 10^{-6}$ m^2/s

The Chezy Coefficient is dependant on the hydraulic radius and the bottom roughness,

$$C = 18 \log\left(\frac{12R}{k_r}\right) = 87.54,$$

where,

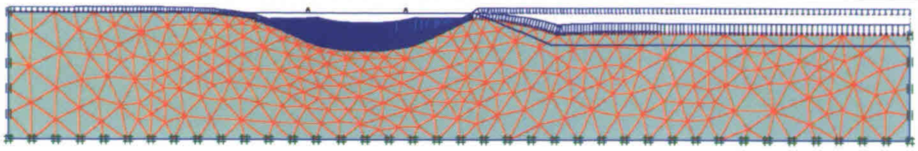
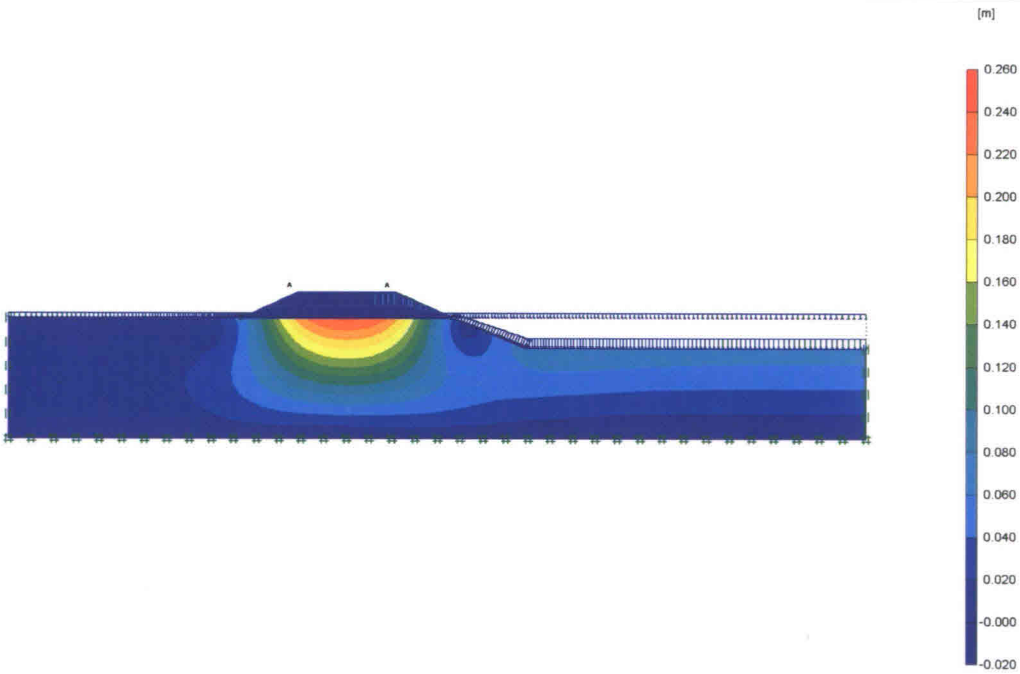
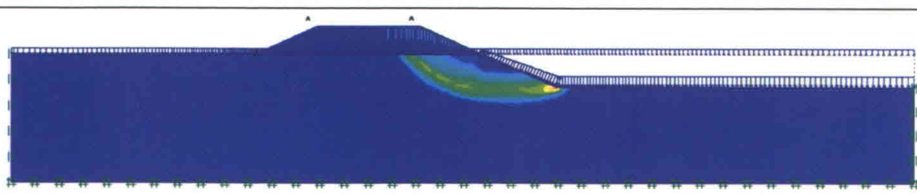
k_r	Bottom roughness	[-],
	$k_r \approx 6d_{n50} = 0.00258$	
R	Hydraulic radius	[m].
	$R = 15.7$ m	

In the Spreadsheet shown on the following page the equilibrium slope angle based on the parameters can be calculated.

Invoer parameters	blue = input, white = calculation fields			
Constant of Van Karreman	kappa	0.4	[-]	
Water Temperature	T	20	[degrees C]	
Kinematic viscoisty	nu	1.00E-06	[m2/s]	
Grain diameter (D_n50)	d	0.00043	[m]	
Sediment density	rho_s	2650	[kg/m3]	silicate
Water density	rho_w	1025	[kg/m3]	salt water
Relative density	delta	1.59	[-]	
Gravitation constant	g	9.81	[m2/s]	
Dimensionless particle diameter	dstar	10.72	[-]	Van Rijn
Shields stability parameter	Tau_c	0.032	[-]	Van Rijn approximation of Shields curve
Channel side slope length		50	[m]	50 for new, 70 for old situation
Channel bottom width		220	[m]	220 for new, 105 for old situation
Channel depth		19	[m]	19 for new, 15 for old situation
Channel cross section	A	5130	[m2]	
Channel wet perimeter	p_w	327	[m]	
Hydraulic radius	R_h	15.7	[m]	
Equivalent roughness	k_r	0.00258	[-]	6 times dn50
Chezy coefficient	Ch	87.54	[-]	Based on Hydraulic radius
Depth averaged velocity	u	0.131	[m/s]	for tidal current
Critical velocity	ustar	0.015	[m/s]	Based on Shields Parameter
Max depth av. Velocity	umax	0.406	[m/s]	For flat bed
Calculated reduction factor	K_alpha	0.32	[-]	
Friction angle	phi	30.0	[degrees]	coarse sand = 30 degrees
		0.52	[rad]	
Final slope angle	beta	28.2	[degrees]	
slope		1: 1.86		

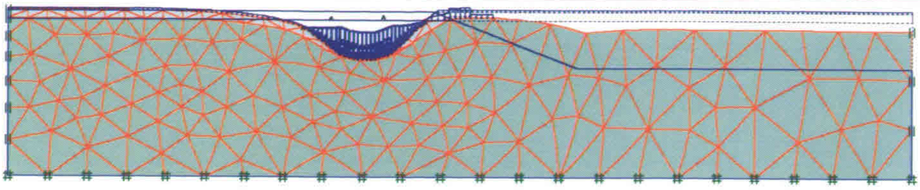
PLAXIS RESULTS

Calculation 1

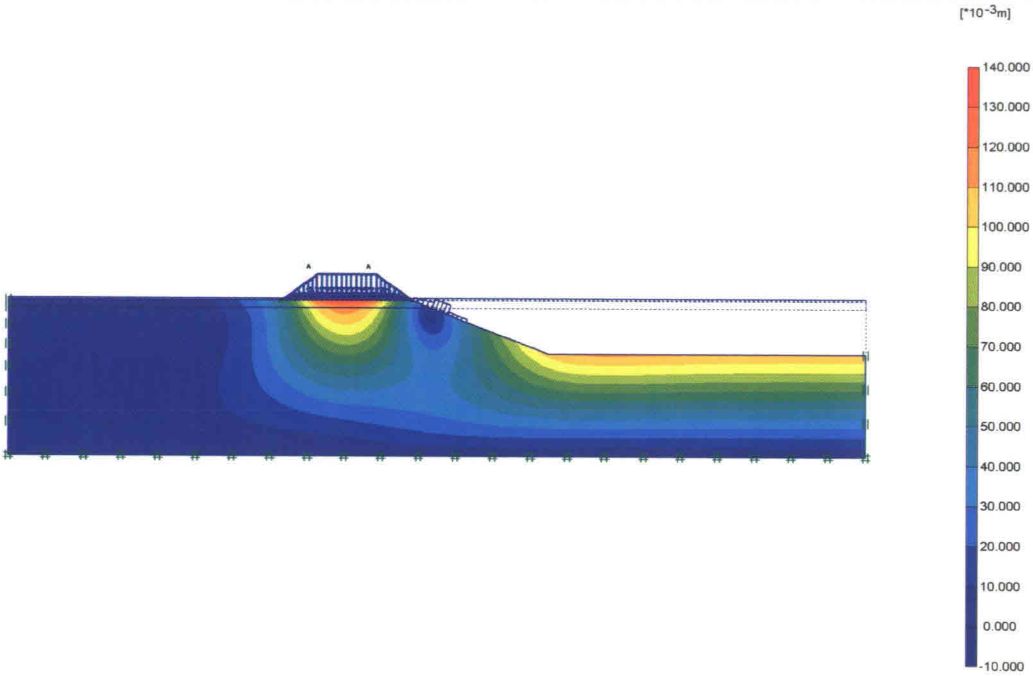
Section 1: Head (–8.5 m CD)
Safety Factor = 1,89
 **Deformed Mesh (scaled up 100x)**
 **Displacements after consolidation** ($U_{tot.\max} = 244.91 \cdot 10^{-3}$ m)
 **Incremental shear strains (slip circle)**

Section 2: Trunk (-3 m CD)

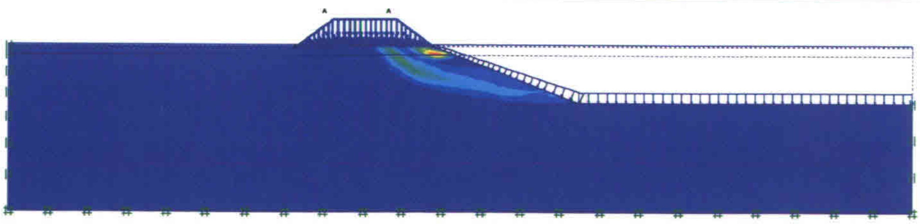
Safety Factor = 1,84



Deformed Mesh (scaled up 100x)



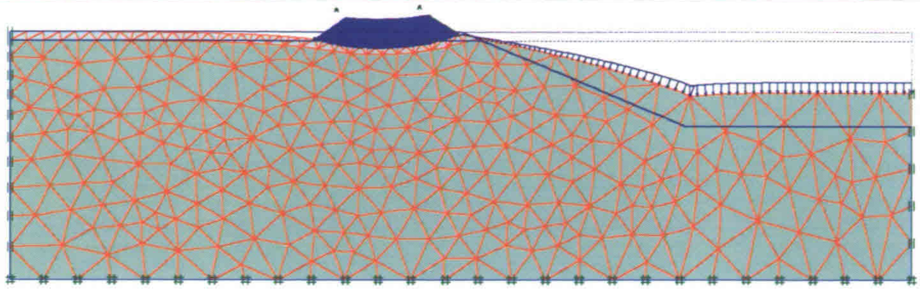
Displacements after consolidation ($U_{tot.\max} = 135.93 \cdot 10^{-3}$ m) (heave)



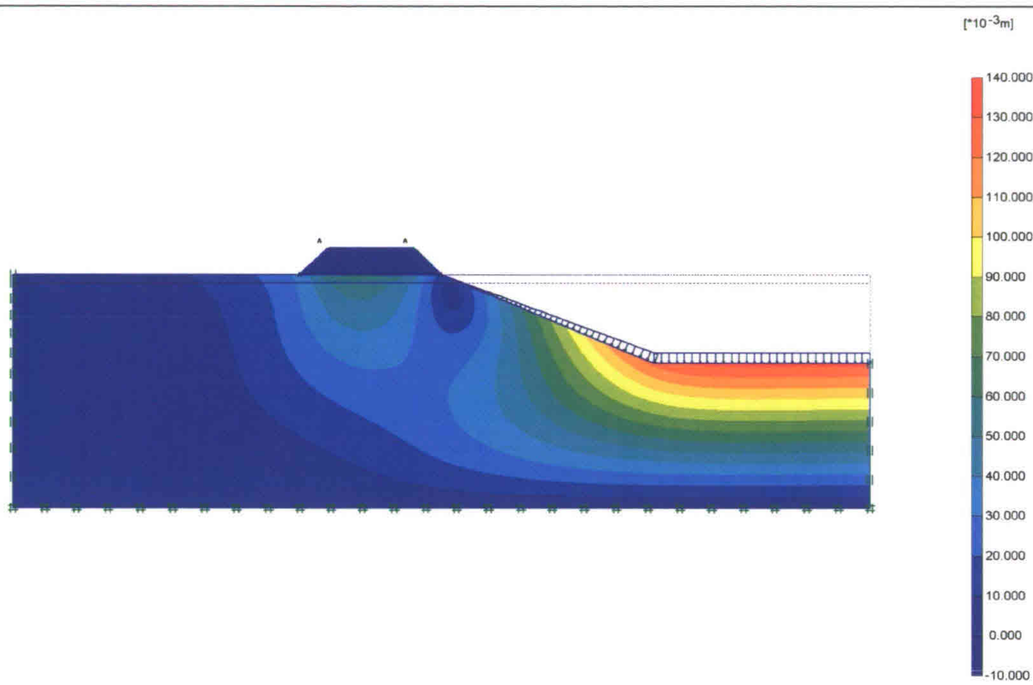
Incremental shear strains (slip circle)

Section 3: Trunk (+0.0 m CD)

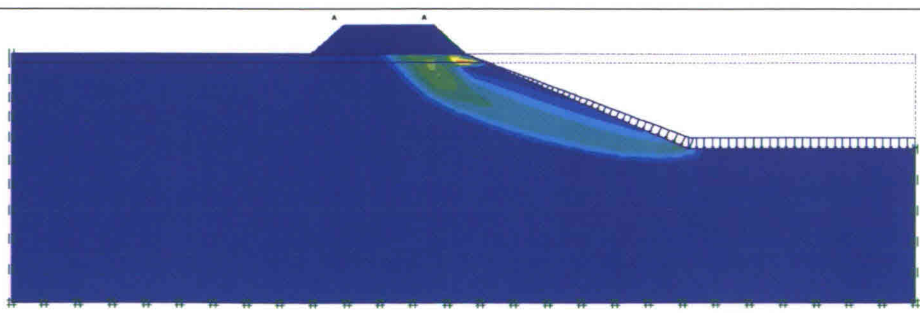
Safety Factor = 1,87



Deformed Mesh (scaled up 100x)



Displacements after dredging ($U_{tot.\max} = 133.77 \cdot 10^{-3}$ m) (heave)



Incremental shear strains (slip circle)

Calculation 2

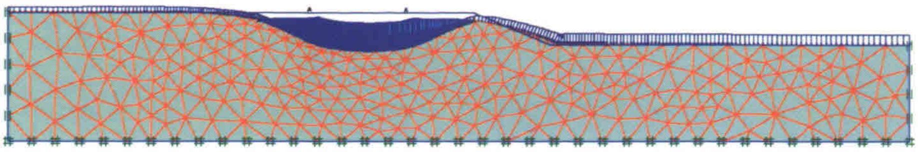
Section 1: Head (–8.5 m CD)

Safety Factors:

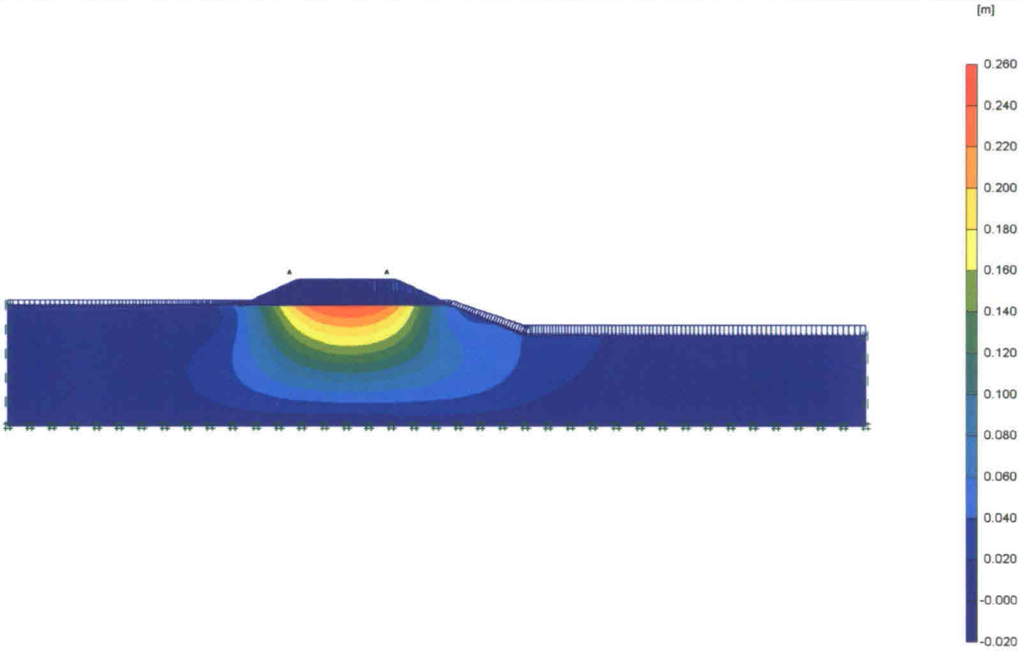
Initial = 2.24

Construction = 1.87

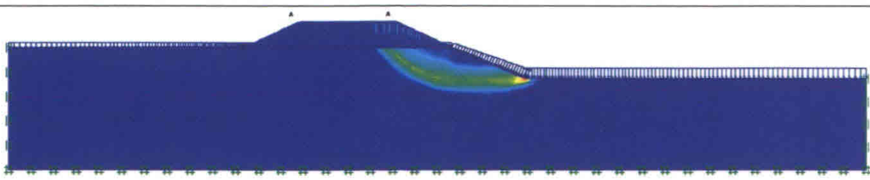
Consolidation = 1.88



Deformed Mesh (scaled up 100x)



Displacements after consolidation ($U_{tot.\max} = 245.72 \cdot 10^{-3}$)



Incremental shear strains (slip circle)

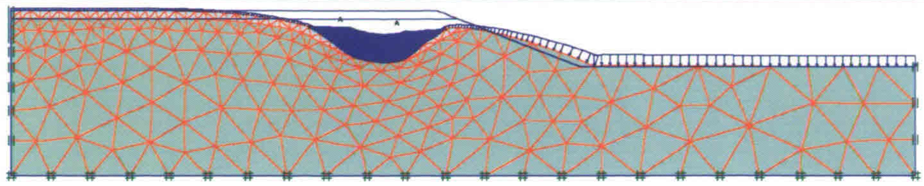
Section 2: Trunk (+0 m CD)

Safety Factors:

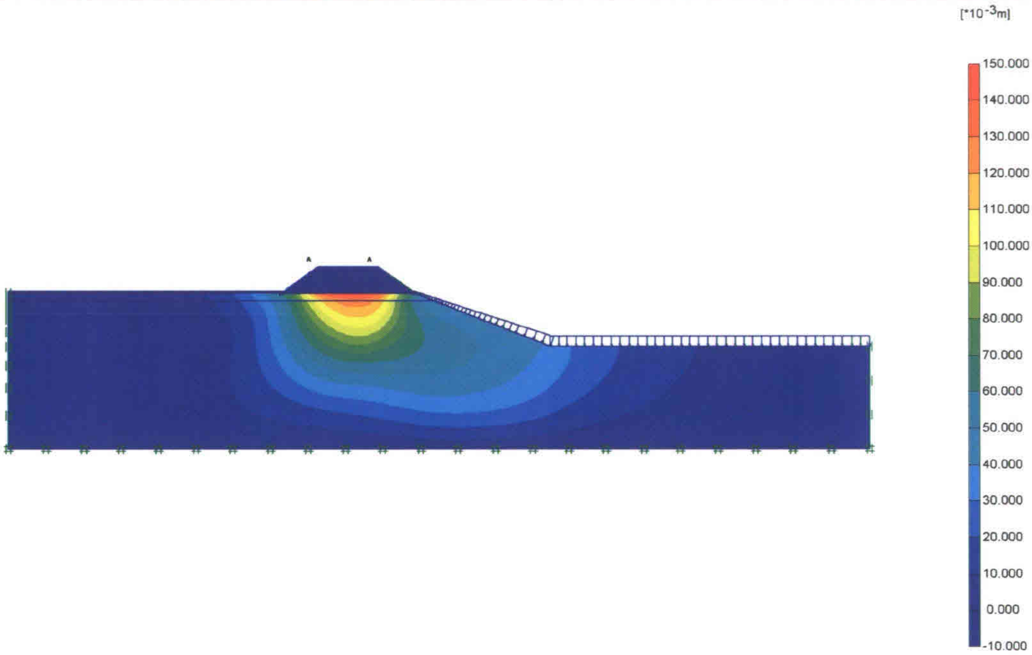
Initial = 2.17

Construction = 1.89

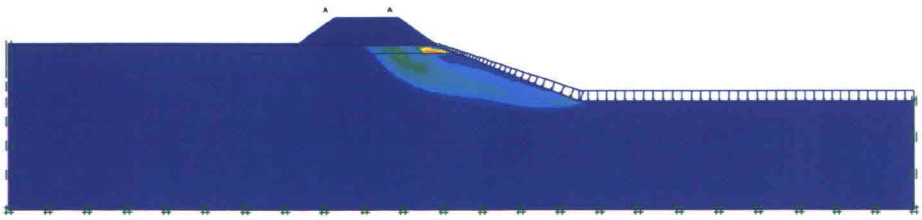
Consolidation = 1.89



Deformed Mesh (scaled up 100x)



Displacements after consolidation ($U_{tot.\max} = 147.33 \cdot 10^{-3}$ m)



Incremental shear strains (slip circle)

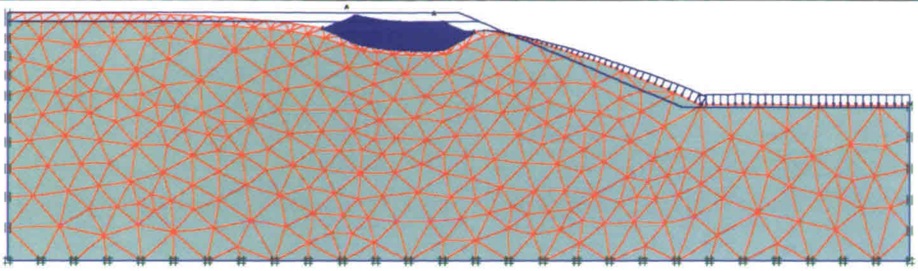
Section 3: Trunk (+0.0 m CD)

Safety Factors:

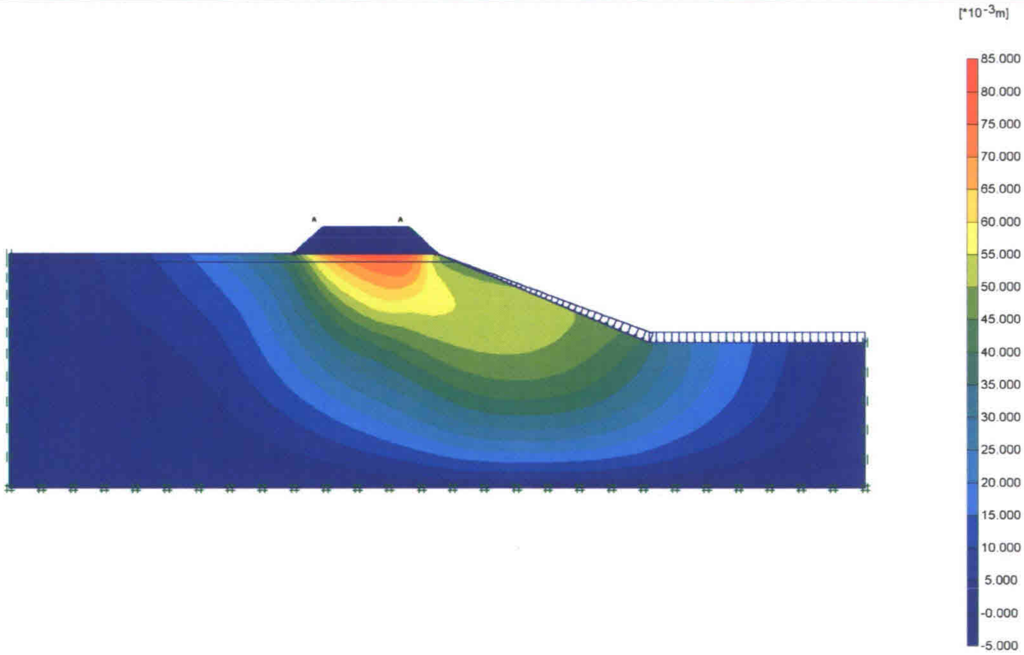
Initial = 1.95

Construction = 1.82

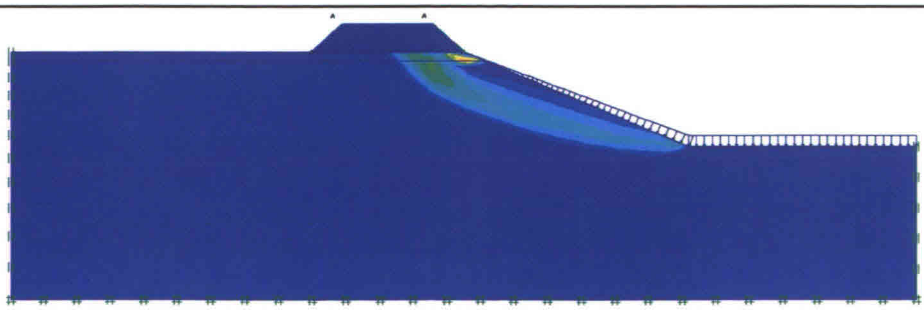
Consolidation = 1.86



Deformed Mesh (scaled up 100x)



Displacements after consolidation ($U_{tot.\max} = 82.41 \cdot 10^{-3}$ m)



Incremental shear strains (slip circle)

AVERAGE PORE PRESSURE

Average pore pressure dependent on time $P_{,t} - \psi = c_v P_{,zz}$ with conditions,

$$
\begin{aligned}
t < 0 &\rightarrow \psi = 0 \\
t > 0 &\rightarrow \psi = \psi_0 \exp(-\delta t)
\end{aligned}
.
$$

This condition expresses that densification starts at time zero and decreases with time. The constant δ depends on the maximum densification. It is related to the so-called preloading or pre-shearing effect, which expresses that due to densification the porous skeleton becomes stronger and less sensitive to liquefaction. The average pore pressure equation can be worked out using the Laplace transformation technique:

$$s\overline{p} = c_v \overline{p}_{,zz} + \frac{\psi_0}{s+\delta}.$$

f	$L_t[f(t)](s)$	Range
1	$\frac{1}{s}$	$s > 0$
e^{at}	$\frac{1}{s-a}$	$s > a$

Layer of finite thickness d founded on an impervious base give the boundary values and the solution:

$$\left.\begin{aligned} z = 0 &\rightarrow p = 0 \\ z = d &\rightarrow p_{,z} = 0 \end{aligned}\right\} \rightarrow \overline{p} = \frac{\psi_0}{s(s+\delta)} \left\{ 1 - \frac{\cosh\left[\left(\frac{1-z}{d}\right)\left(d\sqrt{\frac{s}{c_v}}\right)\right]}{\cosh\left[\left(d\sqrt{\frac{s}{c_v}}\right)\right]} \right\},$$

The approximate Laplace inverse gives a simpler result,

$$p = \frac{2t\psi_0}{(1+2t\delta)} \left\{ 1 - \frac{\cosh\left[\left(\frac{1-z}{d}\right)\left(\frac{d}{\sqrt{2c_v t}}\right)\right]}{\cosh\left[\left(\frac{d}{\sqrt{2c_v t}}\right)\right]} \right\}.$$

For large values of time this solution becomes:

$$p=\frac{\psi_0}{\delta}\left(\frac{d^2}{2c_v t}\right)\frac{z}{d}\left(1-\frac{z}{2d}\right) \qquad \text{for} \qquad t\to\infty\,.$$

Separation of variables

Assume that the solution has the following form:

$$p=p_\infty f(t) \qquad \text{with} \qquad p_\infty=\frac{\psi_0}{\delta}\frac{z}{d}\left(1-\frac{z}{2d}\right).$$

Substitution into the differential equation and integration of the entire area:

$$\int_0^d (p_{,t}-\psi-c_v p_{,zz})dz=0\,,$$

$$\int_0^d (p_\infty f_{,t}-\psi-c_v f p_{\infty,zz})dz=0\,,$$

$$\int_0^d \left(\frac{\psi_0 z}{\delta d}(1-\frac{z}{2d})f_{,t}-\psi_0 e^{-\delta t}+c_v f\frac{\psi_0}{\delta d^2}\right)dz=0\,,$$

$$\frac{d}{3\delta}f_{,t}-de^{-\delta t}+\frac{c_v f}{\delta d}=0\,,$$

$$f_{,t}=3\left(\delta e^{-\delta t}-\frac{c_v f}{d^2}\right).$$

The solution of this equation can be found by a homogeneous part:

$$f=f_0\exp\left(\frac{-3c_v t}{d^2}\right),$$

and a particular part:

$$f=f_1\exp(-\delta t)\,.$$

Substitution gives:

$$f_1 = \frac{\delta d^2}{c_v - \frac{\delta d^2}{3}}.$$

The solution becomes the sum of both:

$$f = f_0 e^{\frac{-3c_v t}{d^2}} + f_1 e^{-\delta t},$$

where the constant f_0 depends on the initial condition, which is expressed by:

$$p_0 = p_\infty f(t=0) = p_\infty \{ f_0 + f_1 \}.$$

Thus:

$$f_0 = \frac{p_0}{p_\infty} - f_1.$$

The solution for the excess pore pressure becomes:

$$p = p_0 e^{\frac{-3c_v t}{d^2}} + p_\infty \frac{e^{-\delta t} - e^{\frac{-3c_v t}{d^2}}}{\frac{c_v}{\delta d^2} - \frac{1}{3}}.$$

This approximate solution shows an initial effect and a final effect. If at the initial stage a sudden collapse of the skeleton occurs, then a sudden pore pressure increase p_0 is expected and the decay due to dissipation is shown. If the initial effect is zero ($p_0 = 0$), the solution becomes:

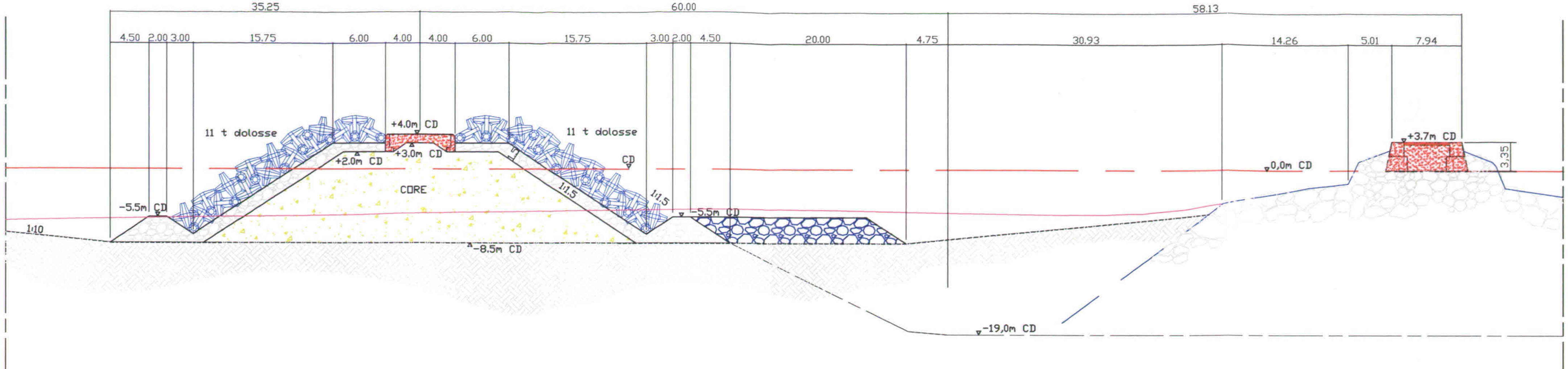
$$p = p_\infty f \qquad \text{with} \qquad f = \frac{e^{-\delta t} - e^{\frac{-3c_v t}{d^2}}}{\frac{c_v}{\delta d^2} - \frac{1}{3}}.$$

Average pore pressure dependent on time,

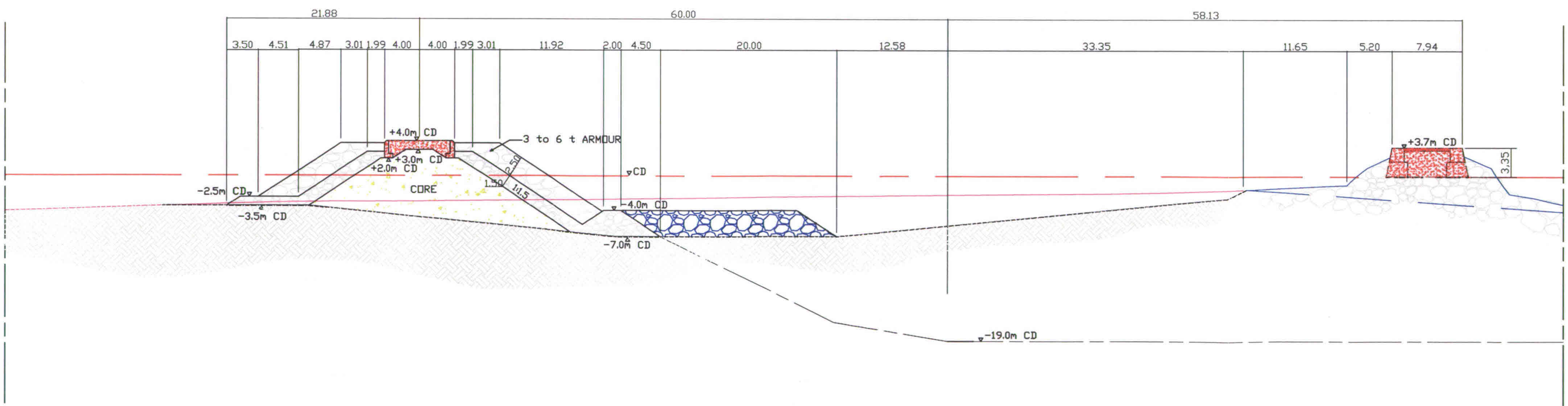
$$P_{,t} - \psi = -\theta P \quad \text{with} \qquad \theta = \frac{3c_v}{d^2}.$$

The result is,

$$P = \frac{\psi_0(\exp(-\delta t) - \exp(-\theta t))}{\theta - \delta}.$$



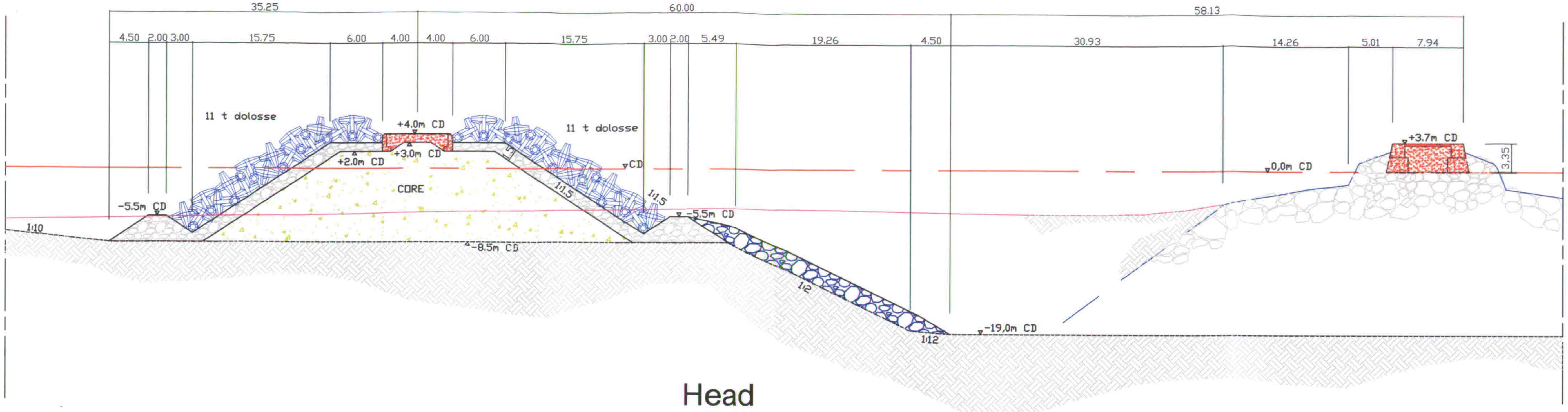
Head



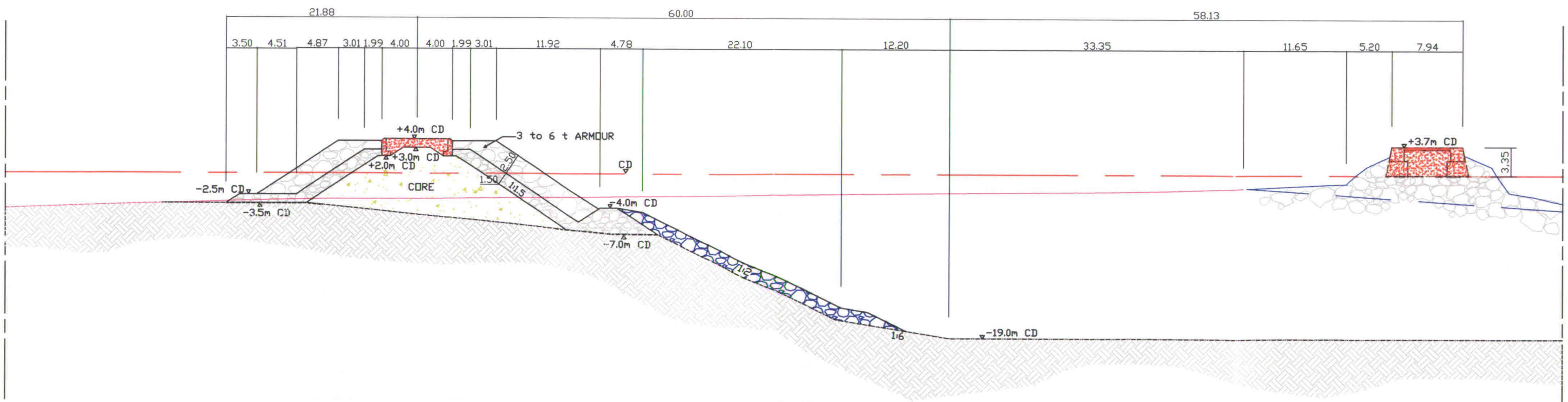
Trunk

- 3 to 6 ton rock
- 0.3 to 1 ton rock
- Quarry run
- Sea gravel
- Sand
- Concrete
- Falling Apron

Title: Appendix Y Placemant Plan Falling Apron			Scale: 1:500	Format: A3
TUDelft Civil Engineering and Technical Geosciences	PRDW Consulting Coastal, Ocean and Environmental Engineers	GROUP CF44	Date: 11-8-2005	
			CT4061	



Head



Trunk

- 3 to 6 ton rock
- 0.3 to 1 ton rock
- Quarry run
- Sea gravel
- Sand
- Concrete
- Falling Apron

Title: Appendix Z Final Design		Scale: 1:500	Format: A3
TUDelft Civil Engineering and Technical Geosciences	PRDW Consulting Coastal, Ocean and Environmental Engineers	GROUP CF44	Date: 11-8-2005 CT4061