1985-2

NYE HAVNE

SEMINAR PÅ TEKNOLOGISK INSTITUT I ÅRHUS

25. OKTOBER 1983

DANSK VANDBYGNINGSTEKNISK SELSKAB

DANISH SOCIETY OF HYDRAULIC ENGINEERING

v/ H. F. Burcharth, AUC, Sohngårdsholmsvej 57, 9000 Aalborg. Tlf. 08 - 142333

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<u>INDHOLDSFORTEGNELSE</u>

10. TESTING OF MARINE STRUCTURES IN
HYDRAULIC MODELS
A. Hasle Nielsen and Jens Kirkegaard (DHI)

11. HYDRAULIC STUDIES FOR BINTULU DEEPWATER PORT
Jens Kirkegaard and A. Hasle Nielsen

Yderligere materiale vedrørende emnet kan bl.a. findes i:

Konference om Havn-Skib (16.11.78)
Skibsteknisk Selskab og
Vandbygningsteknisk Selskab

<u>Tilmelding til VBS's seminar den 25. oktober 1983 i Århus.</u>

<u>"Nye havne"</u>

Århus Havnevæsen, Europaplads 2, 8000 Århus C.	Havnedirektør Kaj Schmidt og havneingeniør G. Deding
Rambøll & Hannemann Rådgivende Ingeniører A/S, Gøteborg Alle 12, 8200 Århus N.	4 personer (Christophe Delrive)
Cowiconsult Rådgivende Ingeniører A/S, Fåborgvej 65A, 5700 Svendborg.	Gerner Juhl Petersen
Odense Havnevæsen, Londongade 4, 5000 Odense C.	Havnedirektør René Petersen og ingeniør Knud Erik Simonsen
Odense Teknikum, Niels Bohrs Alle 1, 5230 Odense M.	Civilingeniør E.O. Nielsen
Rambøll & Hannemann Rådgivende Ingeniører A/S, Teknikerbyen 38, 2830 Virum.	Erik Reker
H. Hoffmann & Sønner A/S entreprenørercivilingeniører, 8220 Brabrand.	Gudik Gudiksen og Troels Ramsussen
Steensen & Warming rådgivende ingeniører, Edouard Suensons Gade 4, 8200 Århus N.	Poul Fusager
Christiani & Nielsen A/S, V. Farimagsgade 41, 1501 København V.	Tom Block-Jørgensen og Ole A. Madsen
ISVA, Bygning 115, Danmarks tekniske Højskole, 2800 Lyngby.	Lektor Jørgen Buhr Hansen
Danmarks tekniske Højskole, Bygning 101, 2800 Lyngby.	Kaj Kristensen
Axel Nielsen A/S Rådgivende Ingeniører, Langelinie 5, 5230 Odense M.	Civilingeniør Erik Sørensen
Monberg & Thorsen A/S, Oslo Plads 12, 2100 København Ø.	Lasse Løvgren
Dansk Geoteknik A/S, Granskoven 6, 2600 Glostrup.	Niels K. Danielsen og Arne Buhl Pedersen
Geoteknisk Institut DGI, Saralyst Allé 52, 8270 Højbjerg.	Knud Mortensen
Havnecon Consulting, Havnen 4, 7620 Lemvig.	Jørgen Bülow Beck
Havnekontoret, Londongade 1, 5000 Odense C.	Havnekaptajn K. Bruun

Civilingeniør, lektor Torben Larsen,	Visborggaardvej 13, 9200 Aalborg SV.
Rambøll & Hannemann Rådgivende ingeniører A/S, Gøteborg Alle 12, 8200 Århus N.	Flemming Bligaard Pedersen, Lasse Nielsen, Lars Rande og Anne Marie Flensted.
A/S Betongården, Lærkevej, 6700 Esbjerg.	Keld Eriksen (÷ spisning)
Statshavnsadministrationen, Postboks 2, 6700 Esbjerg.	B.O. Juhl og P.H. Frisch.
Carl Bro A/S, Nordlandsvej 60, 8240 Risskov.	Søren Vestergaard.
DSB Generaldirektoratet, Sølvgade 40, 1349 København K.	Afd.ing. E. Hess Thaysen, afd.ing. H. Frandsen og afd.ing. B. Pankchik.
Geodan, Karlskogavej 12, Postbox 13, 9100 Aalborg.	Per Krogh Mortensen.
Københavns Teknikum, Ingeniørhøjskolen, Prinsesse Charlottes Gade 38, 2200 København N.	Birte Rodevang.
Statshavnsadministrationen, Postboks 190, 9900 Frederikshavn.	E. Amtoft, Erik Olesen, J. Viggers og H. Kjær, Marius Kristensen.
Aktieselskabet Danena, Europaplads 2, 8000 Århus C.	K. Heiberg Petersen.
COWIconsult Rådgivende Ingeniører A/S, Østeraa 6, 9000 Aalborg.	K. Bundgaard Nielsen.
Jens Chr. Eriksen,	Lundemosen 12, 5450 Otterup.
Dansk Geoteknik A/S, Vestergade 17, 1456 København K.	C.P. Olsen.

Ialt: 48 personer

Bestyrelsesmedlemmer:

Per Tryde, Danmarks Tekniske Højskole (tilmeldt pr. tlf.)

H.F. Burcharth, AUC. (tilmeldt pr. tlf.)

Helge Gravesen, Rambøll & Hannemann.

Havnebygmester C. Warming, Københavns Havnevæsen.

Civilingeniør B. Steen Christensen.

J. Kjærgaard kommer ikke.

Foredragsholdere:

9 personer.

Ialt: 62 personer.

DANSK VANDBYGNINGSTEKNISK SELSKAB

PLACERING AF HAVNE

TEORI OG PRAKSIS

ved

Professor Helge Lundgren

Instituttet for Strømningsmekanik
og Vandbygning, DtH.

I TOO AM HEADING
FOR BRIGHTON

Seminar om Nye Havne, Århus.

1. Indledning

I det følgende gives en fremstilling af problematikken ved valg af en ny havns placering. Opstillingen af teori og systematik for dette virker, løsrevet fra de konkrete forhold, relativ selvindlysende, så disse opstillinger er i fremstillingen løbende illustreret med konkrete eksempler.

Det kan være spændende at beskæftige sig med lysthavne, men det virkelig lystige ved en havn er at vælge placeringen. Alt det andet der så følger efter er "kedelig" projektering. En ny havn skal nødvendigvis være en del af et fordelagtigt projekt, dvs der skal foregå import eller export i forbindelse med udvikling af minedrift, industri, landbrug, fiskeri etc. Der er en række faktorer, der gør sig gældende, ved valg af placering. Man plejer at begynde med nummer 1. Men det er fristende at indføre et nummer 0: Man må være lige så klog som forsynet.

VALG AF EN HAVNS PLACERING

EN NY HAVN SKAL NØDVENDIGVIS VÆRE EN
DEL AF ET FORDELAGTIGT PROJEKT

D.V.S. IMPORT ELLER EKSPORT
I FORBINDELSE MED UDVIKLING AF:

MINEDRIFT
INDUSTRI
LANDBRUG
FISKERI
ETC.

Figur 1

FAKTORER VED VALG AF PLACERING

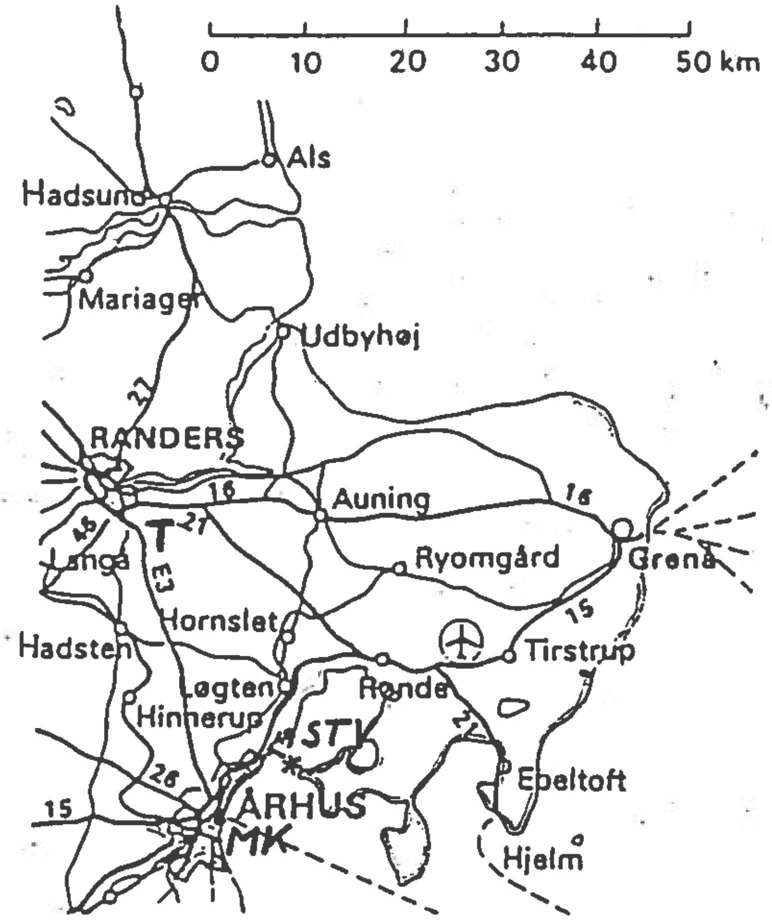
0. MAN MA VÆRE LIGE SA KLOG
   SOM FORSYNET
1. POLITIK
2. MILJØ
3. SOCIOLOGI
4. ØKONOMI

Figur 2

I Vandbygningsteknisk Selskab har der været præsenteret store udenlandske havneprojekter, men historien om de nyere danske havne er ikke ordentligt beskrevet. Så det følgende giver en beskrivelse af de afgørende faktorer for valget af en række nye danske havneanlæg.

2. Studstrupværket

Der går jo så mange Århus-historier, men de gode Århus folk er nu ikke så dumme som historierne giver indtryk af. Midtkraft havde et kraftværk midt i Århus havn, som var for lille, og det var nødvendigt at vælge en ny placering af det næste kraftværk. Idet man valgte placeringen af Studstrupværket så "forudså" man faktisk oliekrisen midt i 70erne og hele den deraf følgende danske energiplanlægning med naturgas og fjernvarme. Således kan man i dag takket være Studstrupværkets beliggenhed i forhold til Århus udnytte en stor del af spildvarmen som fjernvarme. Da man i 1962 valgte placeringen lå tyngdepunktet for elforbruget ved pkt. T på figur 3. Nord og øst for Djursland er der meget langt ud til dybt vand. På østsiden af Mols er der forholdsvis store bølger, så et havneanlæg placeret her vil være dyrt, og så skulle der ydermere trækkes transmissionsledninger ind over det kønne Mols. Ebeltoft er naturligvis udelukket af æstetiske grunde, og det er østsiden af Kalø Vig også. Tilbage blev en række placeringsmuligheder i den vestlige del af Kalø Vig. Man valgte derefter det sted nord for sommerhusbebyggelsen ved Skæring hede, der lå nærmest Århus, nemlig Studstrup, som desuden havnemæssigt var det billigste. Placeringsvalget af Studstrup var altså nemt gjort.



Figur 3

Detailplaceringen krævede en undersøgelse af de geotekniske forhold af hensyn til funderingen af de tunge konstruktioner. Der er lokalt en endemoræne, idet isen stod sydfor Mols/Djursland og skubbede materialer op til de kønne bjerge. Det medfører, at der geoteknisk set er et kolossalt rod i dette område. Detailplaceringen krævede derfor en omhyggelig undersøgelse med mange boringer. Nu er omkostninger ved eventuelt at skulle udføre pælefundering under kedelhus og turbogenerator trods alt en mindre ting i sammenligning med omkostninger ved sejlrende og havneanlæg.

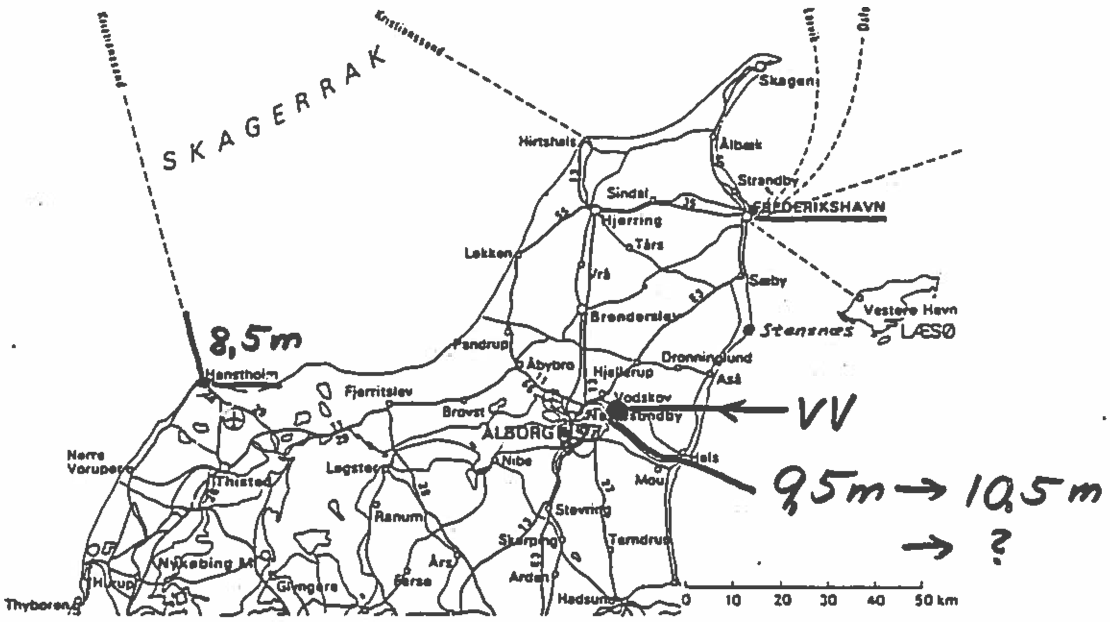
Denne sag er beskrevet indledningsvis, fordi Studstrupværket alligevel repræsenterer en lykkelig bommert. I placeringsanalysen var ikke medtaget de omkostninger, som gik til at oprense marine ammunitionsdumpninger. Der var tilsyneladende rigeligt med vand, men der stod på søkortet, at opankring osv. var forbudt, så der var advaret.

Det kostede i første omgang 1 million, som så voksede til 5 millioner, inden oprensningen var afsluttet. At resultatet alligevel er lykkeligt fremgår af, at et havneanlæg for eksempel på østsiden af Mols alene ville have kostet omkring 20 millioner mere end ved Studstrup. Hertil kommer så yderligere væsentlige ekstraudgifter til transmissionsledninger, ohmske tab osv.

Dette var altså historien om Studstrups placering.

3. Vendsysselværket

Der kan naturligt fortsættes fra Studstrupværket til Vendsysselværket. Her drejede det sig om forsyningsområdet nord for Limfjorden. Nordkraftkommissionen undersøgte 4 placeringer ved henholdsvis Frederikshavn, Stensnæs, Laden (10 km øst for Nr. Sundby), og endelig Hanstholm. Sidstnævnte dog nærmest symbolsk, idet forbruget lå mod øst, og vanddybderne ved Hanstholm gav store problemer.



Figur 4

Ved Nr. Sundby var begrænsningen dengang 9,5 m vanddybde, som jo fornylig er øget til ca. 10,5 m ved uddybning af Hals Barre. Nordkraftkommissionen bestående af 5 specialister (fragt, havneanlæg, osv) måtte lave en kvantificeret placeringsanalyse (se figur 5).

Som det kan ses, er der ikke særlig stor forskel på de beregnede totaludgifter. Men små procentvise forskelle må ikke fejlfortolkes, idet udgifter til kedelhus og turbogenerator m.v. er uafhængige af placeringen.

De to mest gunstige blev efter analysen Laden og Deget. Resultatet var noget uventet, idet Deget på forhånd var vurderet til at stå gunstigt. Konklusionerne i analysen blev taget til efterretning og Vendsysselværket er nu bygget øst for Nr. Sundby.

PLACERING AF DAMPKRAFTVÆRK I NORDJYLLAND

(Nordkraftkommissionen af 1960)

Placering	Udgifter henført til 1967		
	I alt 1967-80	Uafh. af plac.	Afh. af plac.
Laden (v. Nørresundby)	564 Mkr.	513 Mkr.	51 Mkr.
Deget (v. Frederikshavn)	586 "	513 "	73 "
Hanstholm	593 "	513 "	80 "
Stensnæs (syd f. Sæby)	603 "	513 "	90 "

Laden i forhold til Deget:	Billigere havneanlæg	-	12 Mkr.
	Billigere ledningsanlæg	-	12 "
	Mindre ledningstab	-	3 "
	Højere fragter for brændsel	+	6 "
	I alt	-	22 Mkr.

Figur 5

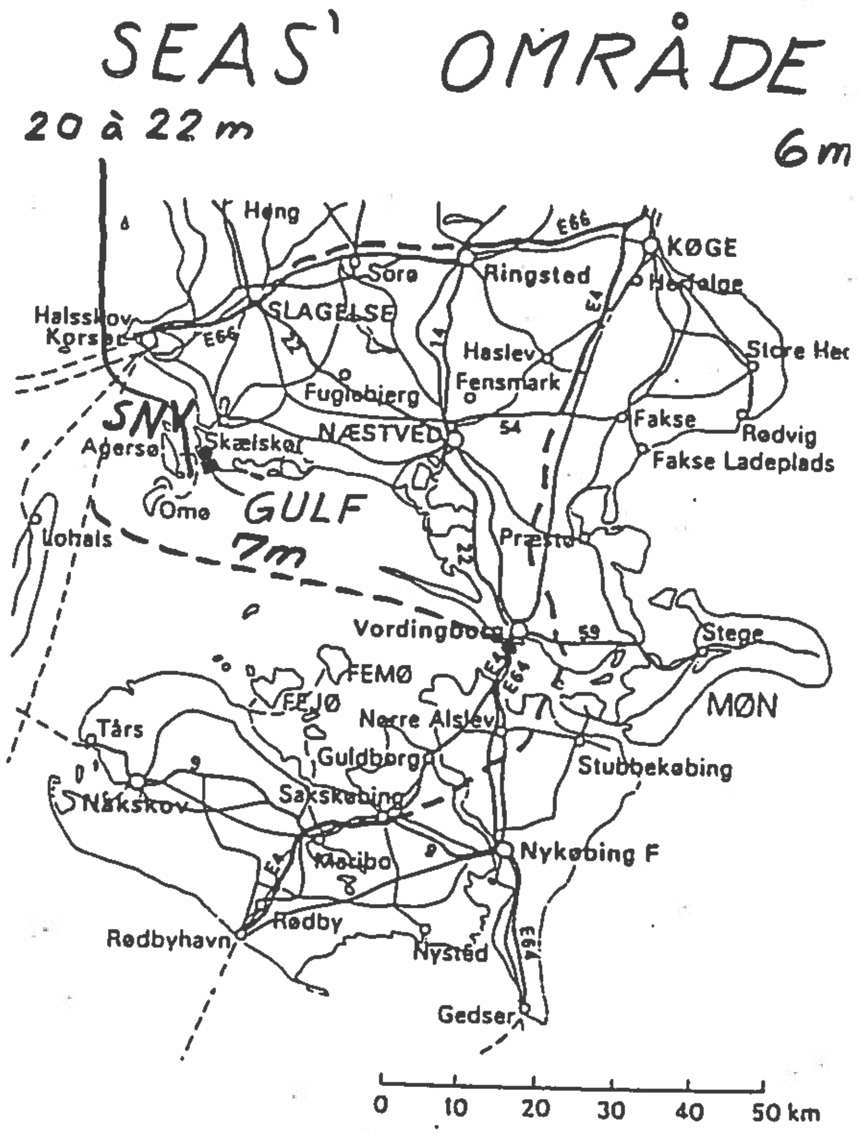
Området ved Laden var også interessant funderingsmæssigt. Her er det postglaciale aflejringer som giver problemer, men det lykkedes at finde en detailplacering, hvor de svære konstruktioner kunne funderes direkte på postglacialt sand, på trods af, at dette havde et vist indhold af tang.

4. Stigsnæsværket

Vi vandrer nu over til Sydsjælland, SEAS område for kraftforsyning.

Her havde SEAS i 30erne bygget Masnedøværket med mulighed for en sejlrende med 7 m vanddybde. Omkring 1960 var det blevet nødvendigt at kunne tage større skibe. Øresund var udelukket på grund af ca. 6 m vanddybde ved

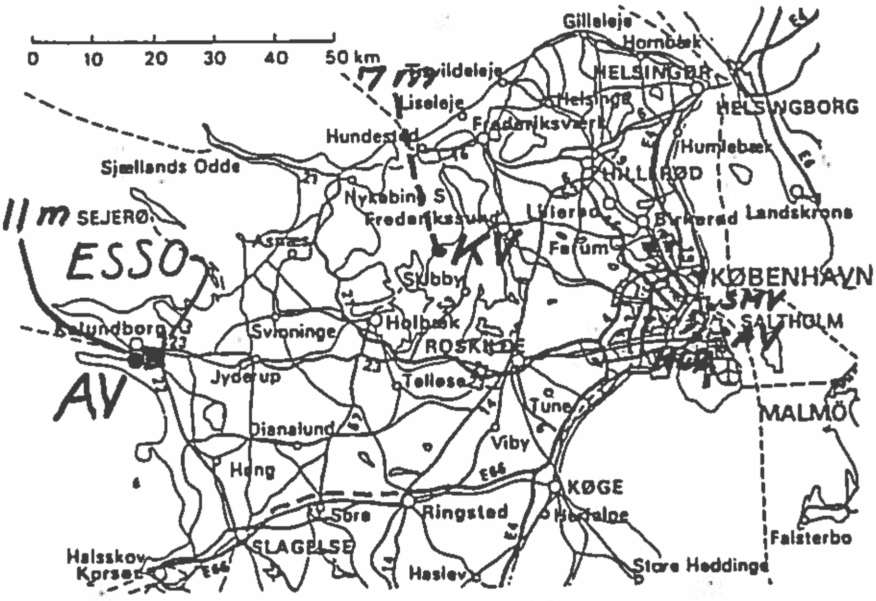
Drogden. Dybderne ud for Sjællands Odde sætter grænsen for skibe der kan besejle Storebælt. Agersøsund har en tilstrækkelig stor vanddybde (30 m). Dette medførte placeringen af Stigsnæsværket. Det forårsagede så yder- mere, at Gulf placerede et raffinaderi ved siden af og SEAS kunne dermed i en længere årrække undgå selv at bygge havneanlæg, ved at olien blev pumpet direkte over fra raffinaderiet. Ved Stigsnæs var der ikke nogen særlige funderingsproblemer. Kystlinien viste direkte et blottet profil af moræneler, så det var indlysende, at der var gode funderingsforhold, undtagen hvis man stødte på lommer af smeltevandsler.



Figur 6

5. <u>Asnæsværket og ESSO Raffinaderiet</u>

IFV's & KK's OMRÅDER

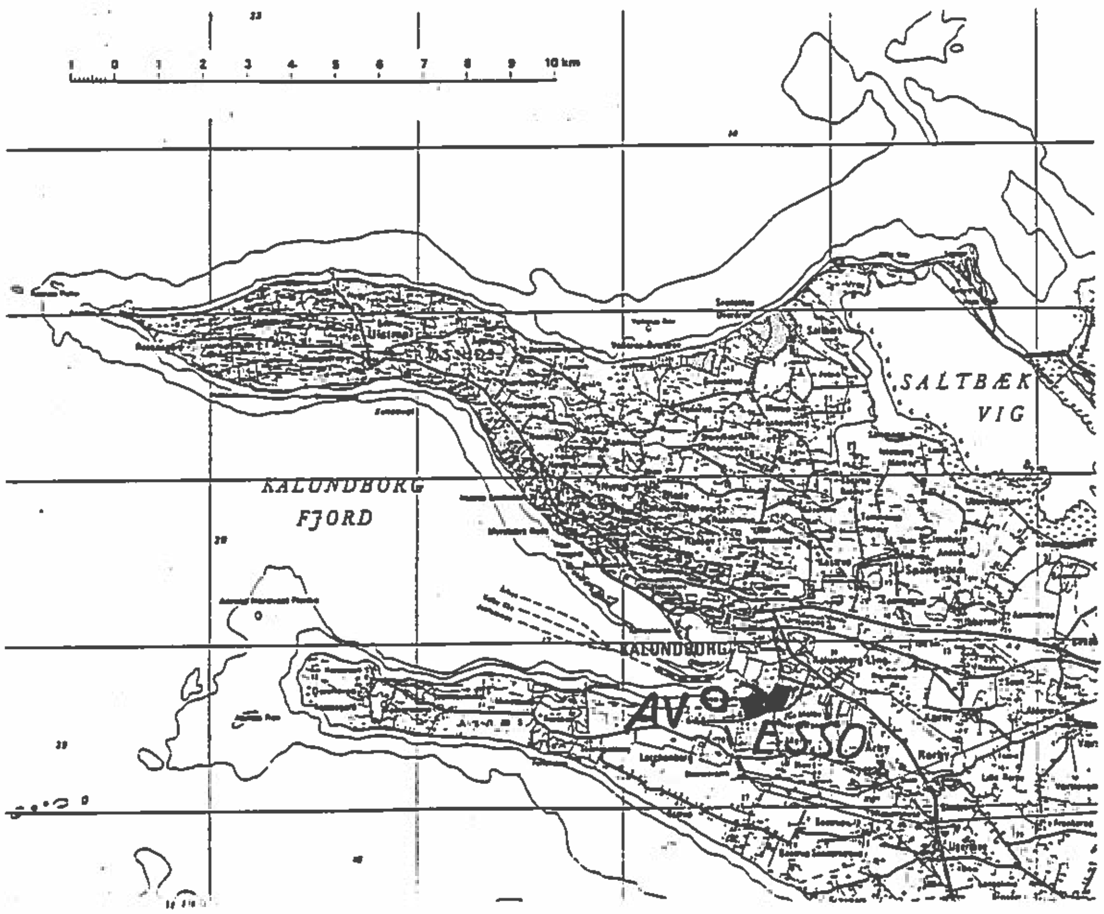


Figur 7

Det næste eksempel er Isefjordværkets og Københavns Kommunes område i den nordlige del af Sjælland.

Her byggede NESA i 30erne Kyndbyværket med en 7 m dyb sejlrende. Det lå godt for forbruget i den nordlige del af Sjælland, nord for linien Korsør-Køge. Sammenslutningen blev opkaldt efter lokaliteten som "Isefjordværket".

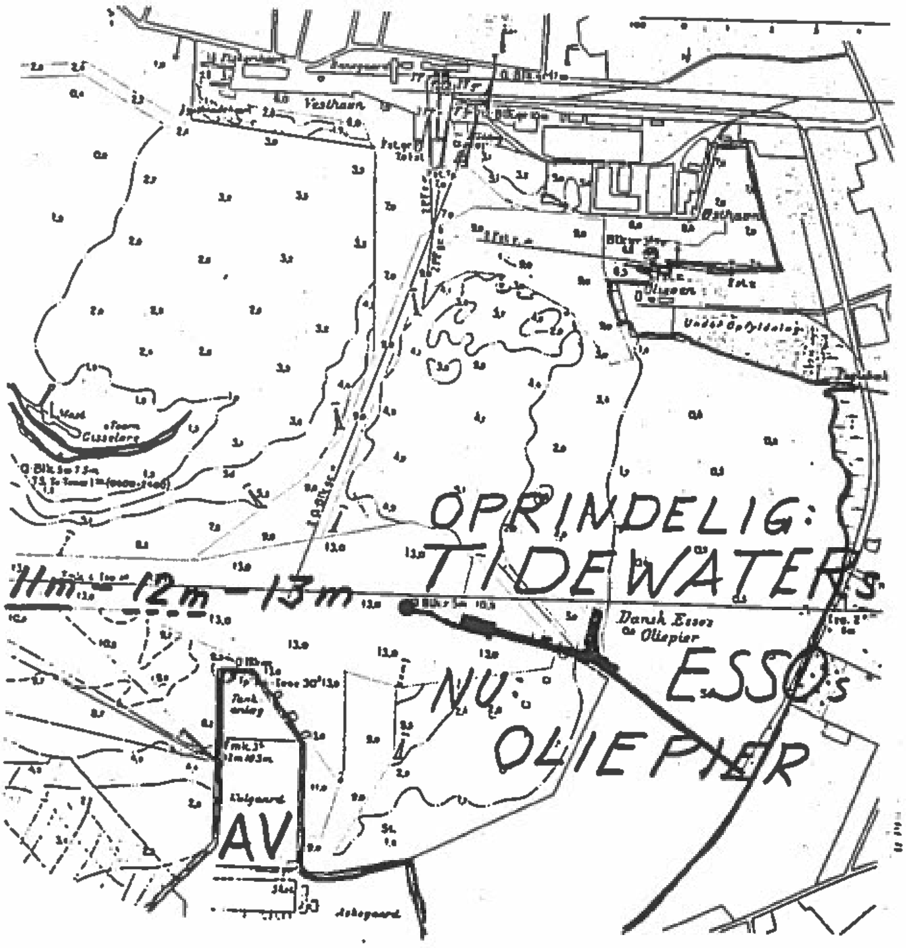
I 50erne blev det nødvendigt at bygge et nyt værk og placeringsvalget af denne blev ledet af overingeniør Svendsen, Vandbygningsdirektoratet. En placering ved Strandvejen var udelukket; de udstrakte sommerhusområder begrænsede yderligere; på Nordkysten var der store Kattegatbølger og sandvandring, Sejrøbugten havde også væsentlig sandvandring, så resultatet blev Kalundborg Fjord. Her er der en gunstig spredning af de bølger, der trænger ind i det dybe område, som giver gunstige bølgeforhold for Asnæsværket. Placeringen blev altså styret af rent havnemæssige betragtninger, idet det jo fremgår klart, at Asnæsværket er placeret langt fra tyngdepunktet i forsyningsområdet. Der blev lavet en 11 m dyb sejlrende ind til værket.



Figur 8

Der er en række interessante historier omkring udviklingen i Kalundborg. Esso raffinaderiet blev placeret ved siden af. Men det var oprindeligt slet ikke Esso's raffinaderi. Omkring 1960 sendte verdens rigeste mand, Getty, nogle folk fra sit firma, Tidewater, til Danmark for at undersøge muligheden for placering af et raffinaderi. De danske politikere var meget imponeret af, at der kom nogen som ville investere 200 millioner kroner.

Beslutningen skulle tages meget hurtigt. Tidewater krævede 12 m vanddybde. Den lokale interesse var meget stor, og der var trussel om en alternativ placering i Sverige.



Figur 9

Den daværende statsminister blev en lørdag eftermiddag opsøgt af en lokal politiker, som krævede tilsagn fra staten om at sikre 12 m vanddybde inden mandag for at undgå, at raffinaderiet blev placeret i Sverige. Overfor denne trussel blev tilsagnet givet. Tidewater projekterede herefter en ganske kort oliepier, så det blev op til staten at lave en stor uddybning i det lavvandede område. Statsministeren burde kun have lovet 12 m vanddybde for den optimale placering af pieren. Den optimale placering blev senere bestemt af Ministeriet for Offentlige Arbejder, så hele historien endte med en lille notits i aviserne: Finansudvalget har bevilget x kroner til y meters forlængelse af pieren inklusive tilhørende rørledninger. Dette sparede staten for alternativet, en bekostelig uddybning. Senere er der uddybet til 13 m.

Denne lille historie viser, hvorfor politik kommer ind så højt oppe på listen over afgørende faktorer.

6. Økonomiske Faktorer - Eksempel Odden - Ebeltoft

De afgørende økonomiske faktorer kan opdeles som vist på figur 10.

ØKONOMISKE FAKTORER

1 LANDTRANSPORTOMKOSTNINGER
(NYE ELLER FORBEDREDE VEJE)

2 SØTRANSPORTOMKOSTNINGER
(OPTIMALE SKIBSSTØRRELSER)

3 ANLÆGSOMKOSTNINGER FOR HAVNEN

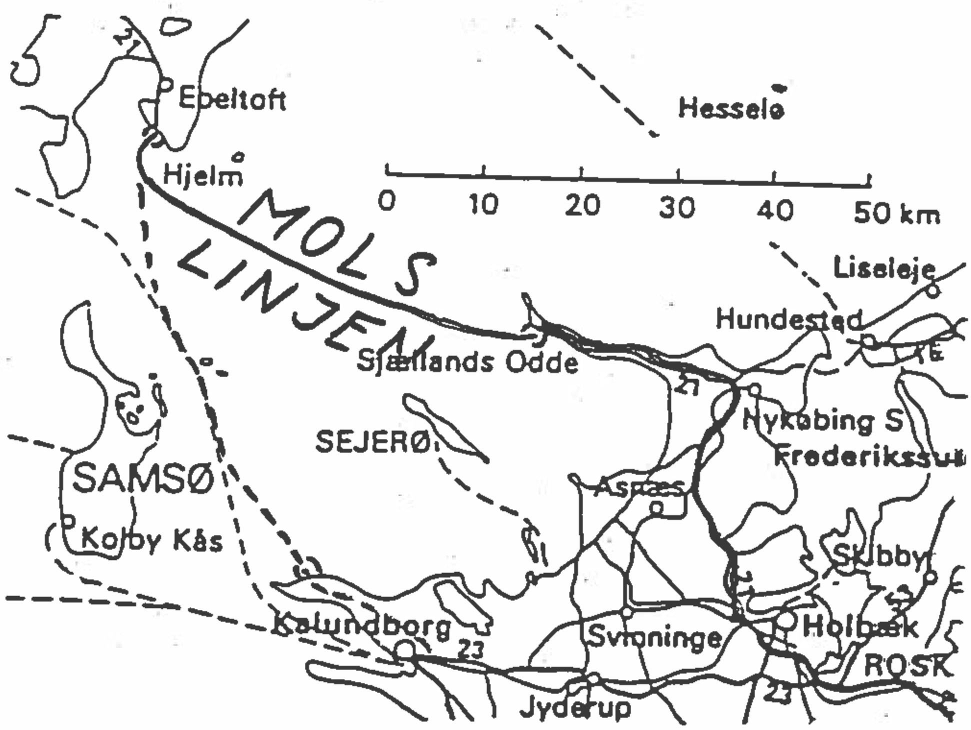
4 DRIFTS - - -

ALT FORRENTET OG AMORTISERET
OVER PROJEKTETS NYTTIGE LEVETID

D.V.S.: DER KRÆVES PROGNOSER FOR
UDVIKLING AF TRAFIKKEN

Figur 10.

Det er klart, at ved valg af placering kan der ikke arbejdes uden fastlagte forudsætninger. Man må have prognoser for skibsstørrelser, trafikstørrelse, cirka krav til kajlængde, manøvrer osv., dvs. en skitseplanlægning af en havn for alle de steder, som skal sammenlignes. Landtransportomkostningerne spiller en betydelig rolle.



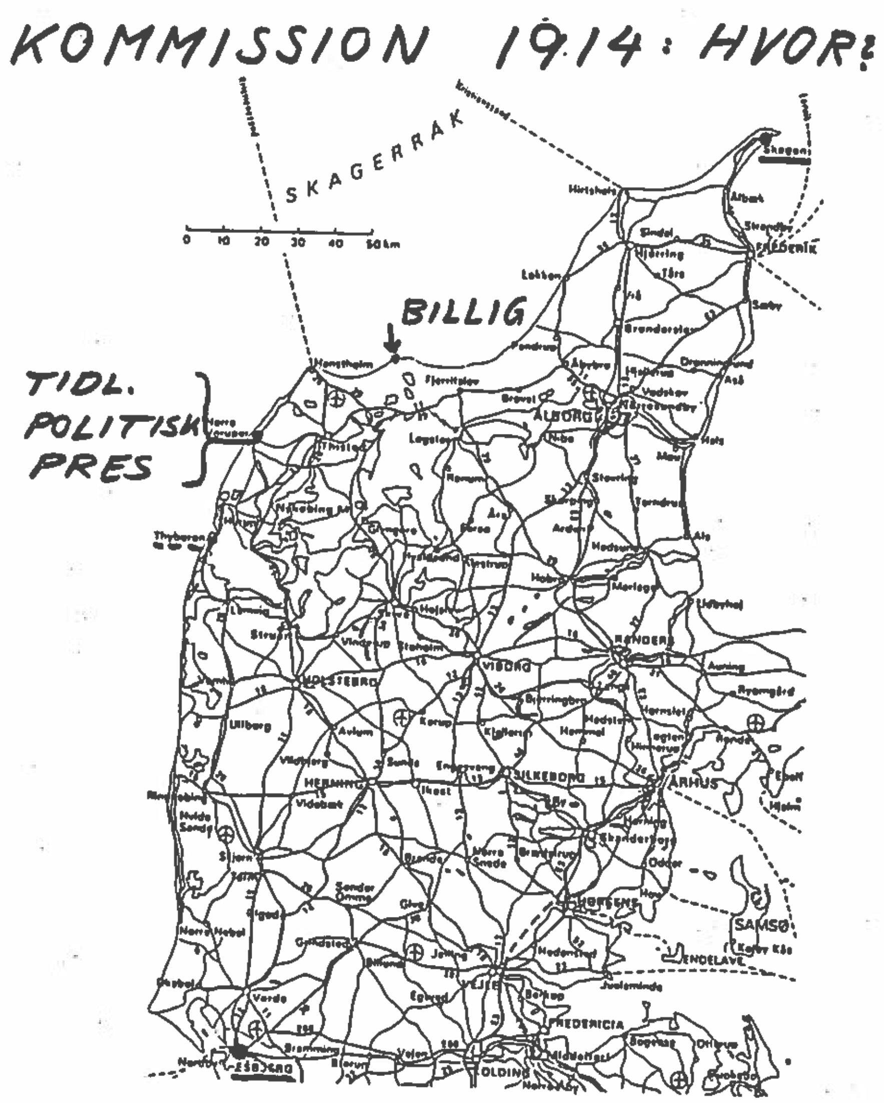
Figur 11

Der blev i sin tid etableret en færgeforbindelse mellem Ebeltoft og Sjællands Odde. Havnene blev placerede efter Molsliniens interesser, idet Molslinien betalte havneanlæggene. Placeringen af havnene er naturligvis blevet godkendt i Ministeriet for Offentlige Arbejders havneafdeling. Samme ministerium har jo også en vejafdeling, som så lige så naturligt må forventes at have set på effekten af den nye tunge trafik som følge af ruten, så placeringen er påvist optimalt i forhold til de samlede udgifter.

Til anlægsomkostningerne for havnen bør ikke glemmes omkostninger ved den følgende bydannelse. Der hører ofte meget til en havn. Ved Studstrupværket kunne det f.eks. undgås at bygge boliger til de ansatte, idet de fortsat kunne bo i Århus. Dette var også væsentligt for placeringen.

7. Hanstholm havn

Efter denne gennemgang af en række nyere havne primært med tilknytning til elproduktion og raffinaderier skiftes emnet over til Vestkysten.



Figur 12

Der har i tidens løb været 19 Tyborønudvalg samt adskillige Hanstholmudvalg. I 1914 blev der nedsat en kommission til at foreslå udbygninger ud over de eksisterende havne i Skagen og Esbjerg og (delvis) Tyborøn. Hvor skulle en ny vestkysthavn placeres? Der var oprindelig forslag om Sandnæshage (billig mulighed) og Nørre Vorupør (politisk pres fra Kompaniet). Men i 1914 kom Fibiger til, og han blev den ledende kraft i kommissionsbetænkningen af 1916. Den blev efterfulgt af en lov i 1917 som besluttede, at der først skulle bygges ny havn ved Hirtshals og dernæst ved Hanstholm.

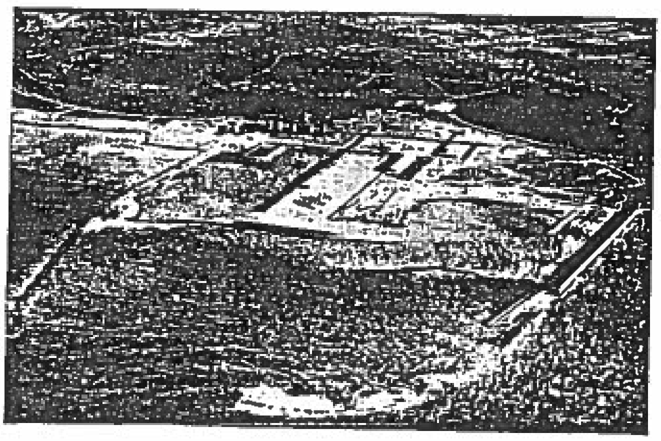
1917 - LOV:

FØRST BYGGES HAVN VED HIRTSHALS
DEREFTER HAVN VED HANSTHOLM

1967: HANSTHOLM HAVN INDVIES

HILSEN FRA


Ingeniørens Ugeblad
11. Årgang Fredag 8. september 1967 Nummer 36
UDGIVET AF DANSK INGENIØRFORENING



I DAG INDVIES HANSTHOLM HAVN

Figur 13

Vestmolen i Hanstholm blev bygget inden 2. verdenskrig. Den blev så delvis ødelagt under krigen og siden repareret efter krigen. I 1960 vedtog folketinget for fjerde gang, at Hanstholm havn skulle bygges. Havnen blev indviet i 1967 allerede inden færdiggørelsen, men den kunne da anvendes som fiskerihavn. På selve indvielsesdagen kom der en hilsen på forsiden af Ingeniørens Ugeblad. Som det fremgår af efterfølgende overskrift, måtte befolkningen, som havde ventet på havnen i 50 år, anse denne hilsen for mindre taktfuld.

Hanstholm-havnen er blevet placeret forkert

— men den er ikke det eneste afskrækkende eksempel på manglende planlægning i forbindelse med havnebyggeri

BEGRUNDELSE HERFOR:

A. BESEJLINGSFORHOLDENE

B. BEGRÆNSET AREAL VED HAVNEN

C. BAGLANDETS KARAKTER (HØJ KLINT)

D. GEOGRAFISK BELIGGENHED
UGUNSTIG FOR TRAFIKHAVN

KONKLUSION:

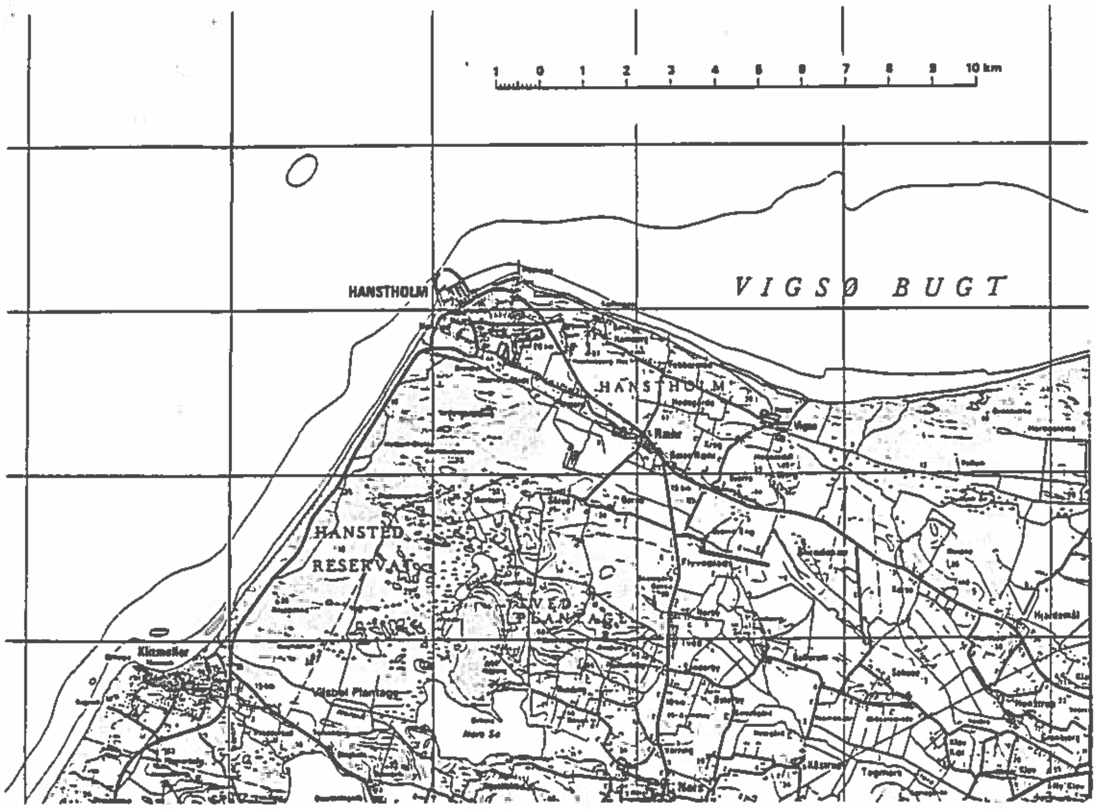
HAVNEN BURDE VÆRE FLYTTET 2 KM MOD ØST

Figur 14

Begrundelsen for kritikken var besejlingsforholdene, begrænset havneareal, baglandets karakter ved tilknytning til Fibigers gamle vestmole, samt at den geografiske beliggenhed var ugunstig for trafikhavnen.

Alle punkter er forsåvidt rigtige. Dem kendte man allerede, da trafikminister Kai Lindberg forelagde loven i 1960. Artiklen konkluderede, at havnen burde være flyttet 2 km mod øst, at man burde have overvejet, om det var rigtigt med en tilknytning til den allerede færdiggjorte vestmole, om der skulle fedtes videre på den bestående havn, eller om en helt radikal løsning skulle vælges.

Svaret på kritikken af placeringen ses ud fra en kystteknisk analyse.

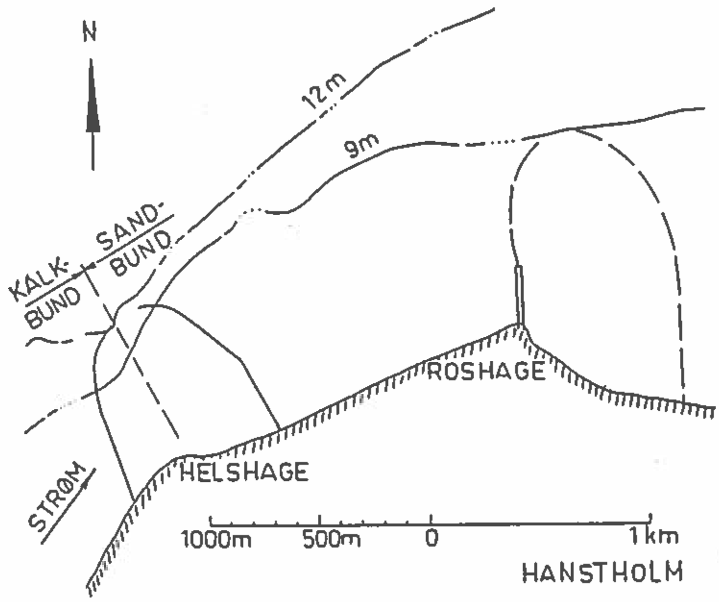


Figur 15

SVARET HERPA VAR:

Sand, fisk, gods ved Hanstholmpynten

— eller hvorfor ingeniør Jørgen Fibigers placering af Hanstholm havn er rigtig



Figur 16

På kysten er der fremspring ved Klitmøller og to knæk ved Hanstholm, henholdsvis ved Helshage og ved Roshage. Spørgsmålet her drejer sig alene om den lokale placering og ikke om den regionale placering. Der var bygget læmole ved Roshage allerede før Fibigers tid. Men Fibiger besluttede at flytte hen til Helshage. Den valgte placering kan forklares med ordene: Sand, fisk og gods ved Hanstholmpynten. I 100 år har problemet på Vestkysten været SAND. Havne KAN sande til. Sejlrender VIL sande til. Sandproblemet overskyggede alle andre problemer. Idet der var tale om en fiskerihavn var det afgørende, at prognoser vedrørende fiskeriets størrelse var gode, så investeringen var samfundsmæssigt rentabel, således at den uanset de tekniske vanskeligheder burde bygges. Gods derimod spillede en helt underordnet rolle for beslutningen.

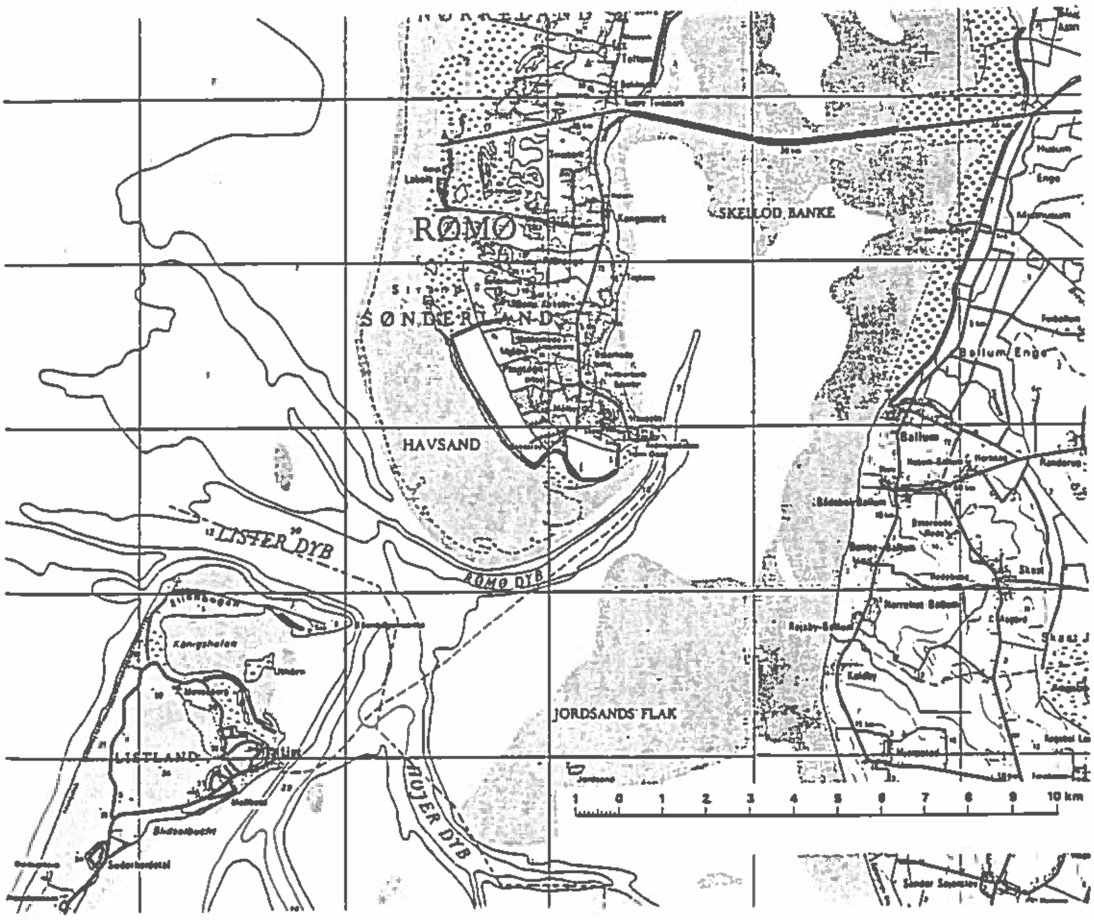
At placeringen ved Fibigers gamle mole er rigtig fremgår af figur 16. Bemærk dybdekurvernes placering. Fibigers gamle mole når ud til kalkbund. Det beviste, at under storm opslemmede de brydende bølger sandet, således at sandet ikke blev liggende på bunden udenfor havnen. Det kom det dog til senere. Da havnen var bygget, skød sandet frem for at komme forbi havnen. Men kyststrømmen var koncentreret på hjørnet af Jylland, og den kunne dermed bruges til at bringe sandet forbi med undtagelse af ca. 7%, der, som det blev forudset, ville lægge sig i havnen. En flytning til Roshage ville have givet længere moler, mindre vanddybde og større sandproblemer, men naturligvis også et større bagland.

For tiden planlægges udvidelse nummer to af Hanstholm Havn, og det viser sig, at der faktisk er rimeligt billige udvidelsesmuligheder, der ikke nødvendiggør en bekostelig og miljøskadelig fjernelse af dele af klinten.

8. Rømø havn

På fig. 18 er baggrunden for Rømø havn beskrevet.Det fremgår vist klart, hvor kolossalt ladet med politik placeringen var. Baggrunden for, at folketinget for fjerde gang vedtog, at der skulle bygges ved Hanstholm har også sammenhæng med Rømø. Samtidig med loven om Hanstholm i 1960 blev der forelagt en anden lov om bygning af en havn på sydspidsen af Rømø ved Havneby, og de to lovforslag fulgtes pænt ad gennem folketinget til samtidig vedtagelse.

Begge områder var underudviklede dele af landet, og havneudbygningerne skulle bidrage til forøget erhvervsudvikling. Sådan er de to havne altså knyttet sammen.



Figur 17

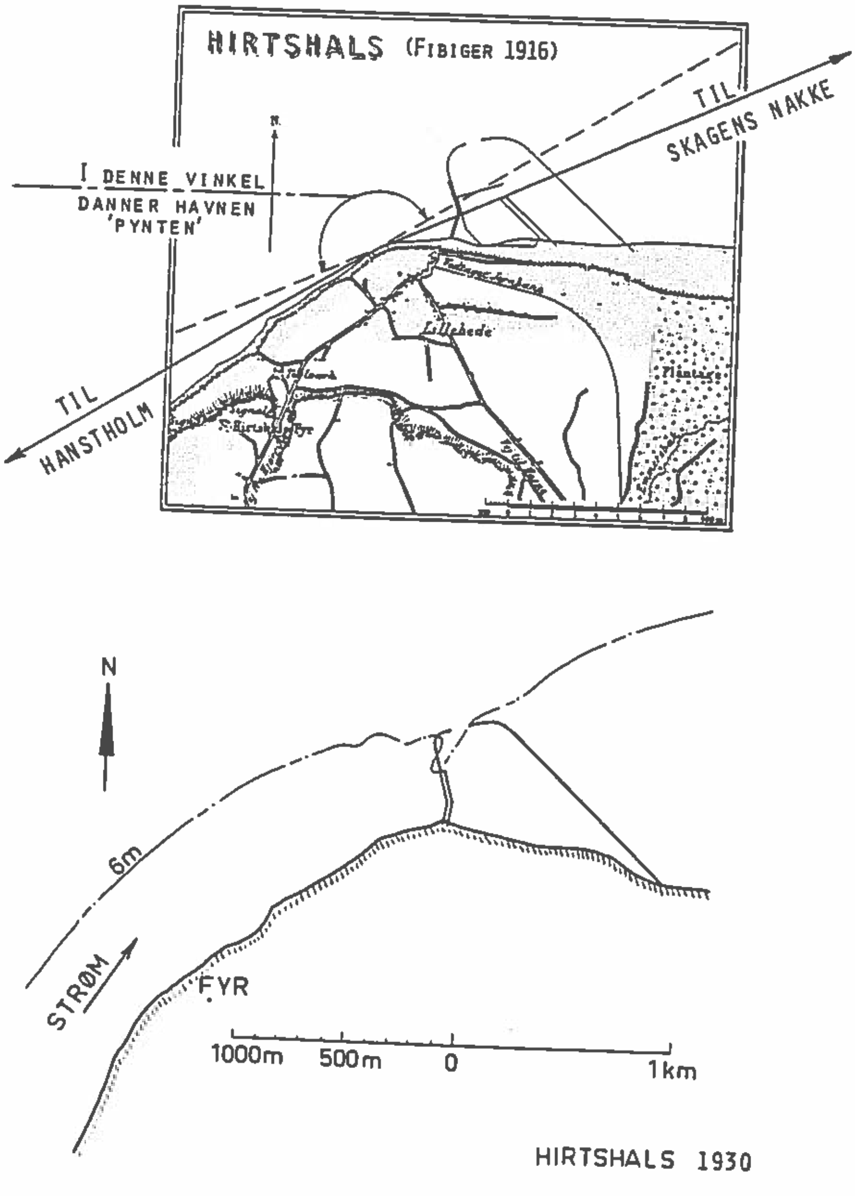
Ligesom på Fanø lå også Rømø's vigtigste landingsplads ved øens sydøstpynt. Her gik dybet tæt under land, her var læ for vestlige vinde. Så tidligt som 1858 blev stedet udpeget som det bedste til anlæg af en vestjysk havn, også i tysk tid havde man planer, men først i 1960 blev loven om en fiskerihavn på Rømø vedtaget - efter at dæmningen havde givet sagen realitet. I maj 1964 blev Rømø Havn indviet.

Hensigten med havnen var at skabe forudsætning for et havgående fiskeri med alle dets følgeindustrier. Herved håbede man at kunne standse Rømø's affolkning. Men det var lettere sagt end gjort. Syd for Esbjerg findes ingen tradition for havfiskeri, og tilgangen nordfra var trods egnsstøtte begrænset. I 1967 søgte man at skabe interesse for fangst af hesterejer, et fiskeri af stor betydning i det tyske vadehav. Man fik da også startet, fik bygget en fabrik, og søgte at skole folk i fangst og behandling. Man måtte dog opgive at gøre Rømøserne til et rejepille-folk, og fangsten eksporteres nu upillet til Tyskland. Fællesmarkedet har åbnet nye muligheder, idet en betydelig del af den nordfrisiske rejeflåde benytter havnen som base.

Færgefart er blevet havnens hovedfunktion. De høje banetakster over Hindenburg-dæmningen gør det billigere for både fragt- og persontrafik at lægge vejen til Sild over Rømø.

Figur 18

9. Hirtshals havn



Figur 19

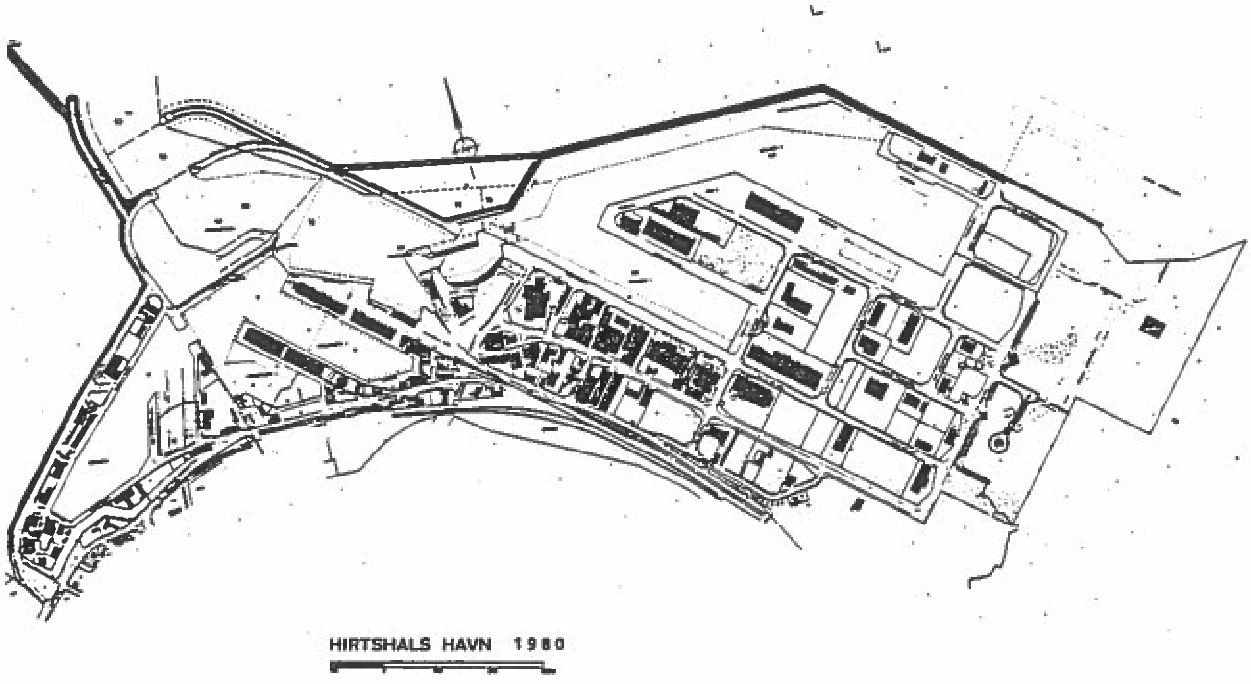
Som det fremgår af det foregående, er der også kraftig berøring mellem Hanstholm og Hirtshals havne. Hirtshals havn blev ganske vist indviet allerede i 1930, så den er ikke særlig ny. Men historien om Hirtshals havn hænger nøje sammen med de øvrige Vestkysthavne.

Fibigers begrundelse for valg af placering af havnen er egentlig forkert fysisk set. Det skal ikke vurderes alt for kritisk med dagens briller, for den fysiske forståelse af sandvandring på kyster i 1916 var naturligvis særdeles beskeden. Fibiger byggede på "Pyntteorien", som er illustreret ud fra Fibigers ord i betænkningen for Hirtshals Havn. Hvis havnemundingen ligger i vinkelen mellem linierne til Hanstholm og Skagen Nakke, danner havnen pynten.

Idet "pynter er sandfrie", skulle Hirtshals havn dermed også blive sandfri. Det kom nu ikke helt til at passe. På den baggrund udtalte en kommission i 1951, at en Hanstholm havn ville sande til, idet det ville være for dyrt at opretholde en sejlrende til havnen. Denne vurdering blev baseret på de ca. 2 millioner m^3 tilsanding øst for Hanstholm vestmole samt en forventning om tilsvarende forhold ved Hanstholm som ved Hirtshals. Men der er store forskelle imellem Hirtshals og Hanstholm; så der kan ikke direkte sammenlignes.

Kysten ved Hirtshals har også to knæk, først et ved fyret og dernæst ved vestmolen (se figur 19). Fibigers grund til at placere havnen ved det andet knæk i modsætning til ved Hanstholm har udover pyntbetragtningen sandsynligvis været, at den eksisterende læmole var ret lang og netop udbygget.

Med en munding, der efter forskellige udbygninger ser ud i 1980 som vist på figur 20, er der gode muligheder for at sandet kan lægge sig i hvirvlen bag dækmolen, der udgår fra vestmolen. Det giver kvaler med tilsanding og tillige navigationskvaler for fiskerne. Derfor måtte dækmolen fornylig yderligere forlænges. Som havnen nu efterhånden er blevet udformet med udbygning af et oprindeligt principielt dårligt koncept, er der ingen økonomisk rimelig udbygning, der kan ændre havnen til noget, der ligner en bedre udformning svarende til Hanstholm. Med den valgte placering var Fibigers glatforløbende munding ikke særlig heldig med hensyn til sandvandring.



Figur 20

10. Anlægsomkostninger for en havn

ANLÆGSOMKOSTNINGER

1 OCEANOGRAFI

2 HYDROGRAFI

3 TOPOGRAFI

4 GEOLOGI

5 KLIMA

Figur 21

I anlægsomkostninger for en havn indgår der en række faktorer af fysisk karakter, nemlig alt hvad der har med havet og havets materialer at gøre. Dybdeforholdene er naturligvis afgørende for f.eks. at undgå lange bekostelige sejlrender. Ofte kræver placeringsvalget ikke store beregninger, som f.eks. i tilfældet Stigsnæs/-Masnedø, hvor omkostningerne til uddybning af en sejlrende til Masnedø straks umuliggjorde denne placering. Topografien inde på land er vigtig af hensyn til de nødvendige landarealer. Geologiske og deraf følgende geotekniske forhold er tilsvarende indlysende vigtige. Men også klimaet kan have betydning af hensyn til opbevaring af varer på havnen og for arbejdsforholdene.

For en kysttekniker er de oceanografiske/kysttekniske faktorer de mest spændende. Den ideelle beliggenhed stiller krav som anvist på fig 22. Hvis der ikke er nogen bølger på stedet som i en naturhavn, er der ikke brug for moler, der ofte er det dyreste element i en havn.

REQUIREMENTS TO SITE

NO WAVES	⟹	NO BREAKWATERS
NO SURGING		NO BROKEN MOORINGS
NO SAND DRIFT		NO MAINTENANCE DREDGING
NO CURRENTS		NO COLLISIONS
NO SILT IN WATER		NO MUD
NO TIDES		NO PONTOONS
NO ICE		NO WINTER DAMAGE

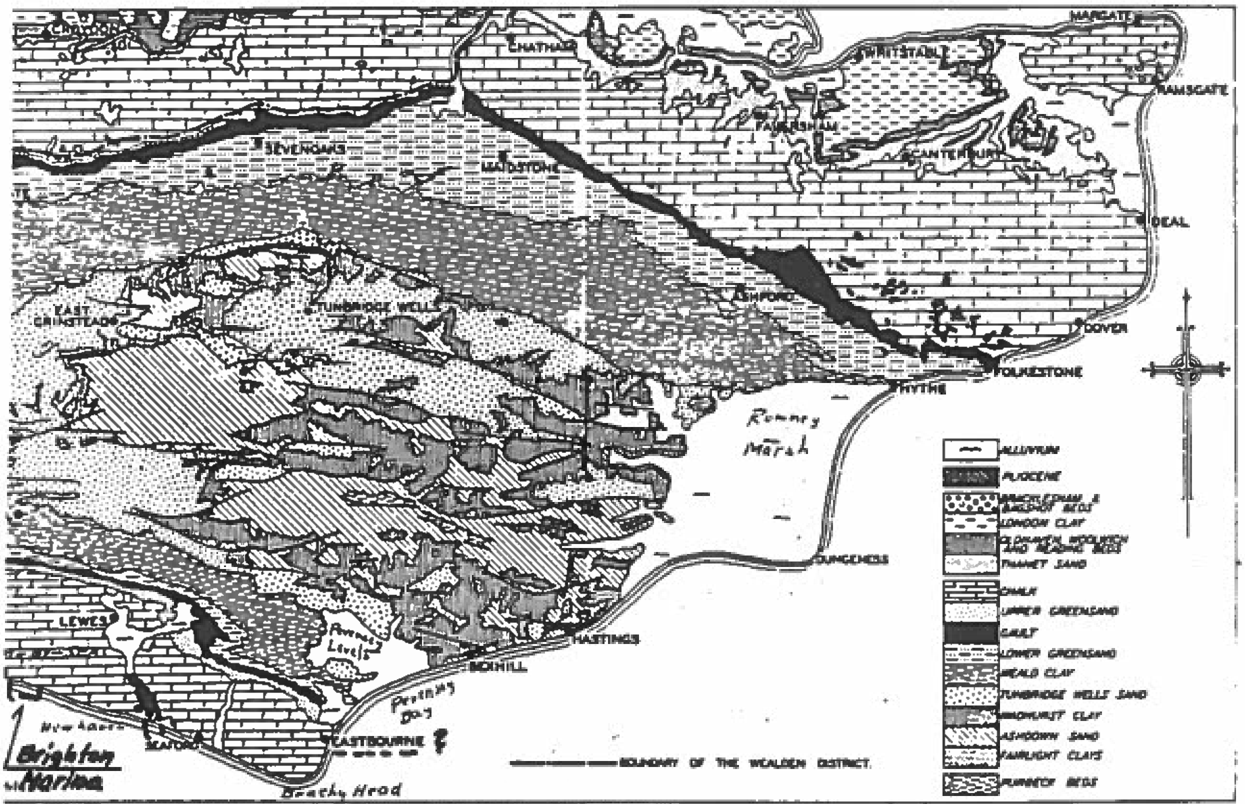
REQUIREMENTS TO DESIGN

NO COASTAL ENGINEER

Figur 22

Hvis der ikke er surging (langperiodisk bevægelse) i havnen, går fortøjningerne ikke i stykker. Hvis der ikke er sandvandring eller silt i vandet, er der ingen oprensning. Hvis der ikke er nogen strøm, sker der ingen kollisioner. Hvis der ikke er tidevand i en lystbådehavn f.eks., er der ikke brug for pontoner. Hvis der ikke er is, sker der ingen skader om vinteren. Dette er kravene til den ideelle beliggenhed. Heraf følger, at kravene til projektering bliver meget enkle, for ved den ideelle placering er der ikke brug for nogen kystekniker! Men det er jo heldigvis sådan, at der er nogen, der har mange penge eller af andre grunde er tvunget til at bygge på dyre steder, så der bliver spændende og udfordrende projekter, jfr. figuren på titelsiden.

11. Englands sydkyst.



GEOLOGICAL SKETCH-MAP OF THE WEALDEN DISTRICT.

Figur 23

Et eksempel kan tages fra den engelske sydkyst med mange lystbådehavne. Ved Newhaven er der en flodmunding, der nærmest er fyldt med både. Den dyre Brighton Marina med meget dyre moler er blevet bygget , fordi området betragtes som fashionabelt. Ved Eastbourne er der projekteret men endnu ikke bygget nogen havn på grund af faldende konjunkturer. På figur 23 ses situationen omkring Eastbourne, hvor der ligger en grus/ral aflejring

"The Crumbles". Denne skyldes morfologisk, at det lavvandede Royal Sovereign Shoals, der i tidligere tid har været en ø, har dannet læ for bølger fra sydøst med det resultat, at kystlinien er rykket ud. Senere er øerne borteroderet. En placering af en havn ved Eastbourne ville kun kræve korte moler, og havnebassinerne kunne graves billigt ud i ral/grus lagene. De udgravede materialer kunne ydermere genanvendes f.eks. til betonstøbning. Der ville dermed være en størrelsesorden til forskel på omkostningerne på et projekt her og så den gennemførte Brighton Marina. Eksempel viser betydningen af de geologiske faktorer (figur 24).

GEOLOGISKE FAKTORER

1 REGIONALGEOLOGI

2 TEKTONIK

3 KYSTMORFOLOGI

4 MATERIALER TIL MOLERNE

5 FUNDERING

6 UDDYBNING

Figur 24

12. Diskussion af havneformninger

Det er i det foregående blevet påpeget, at ved en havn på en åben kyst er molerne dominerende for omkostningerne, med faktorer som er beskrevet i figur 25.

M O L E R

A. M I N I M A L L Æ N G D E

B. - V A N D D Y B D E

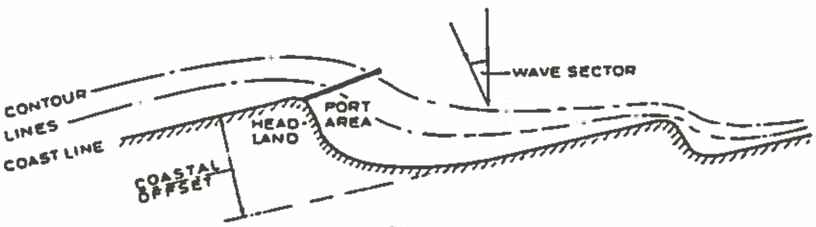
C. G O D F U N D E R I N G

D. S T O R E B L O K K E

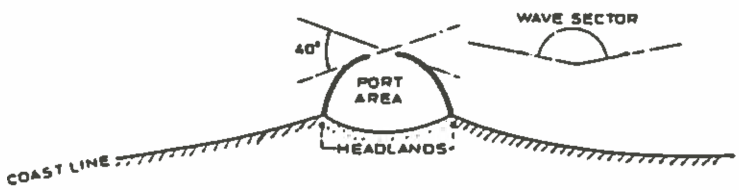
E. K O R T T R A N S P O R T
A F B L O K K E / C A I S S O N E R

Figur 25

Dette kan yde [illegible] ved nogle princip-skitser.



(a) Port Site Selection and Breakwater Alignment for Narrow Wave Sector



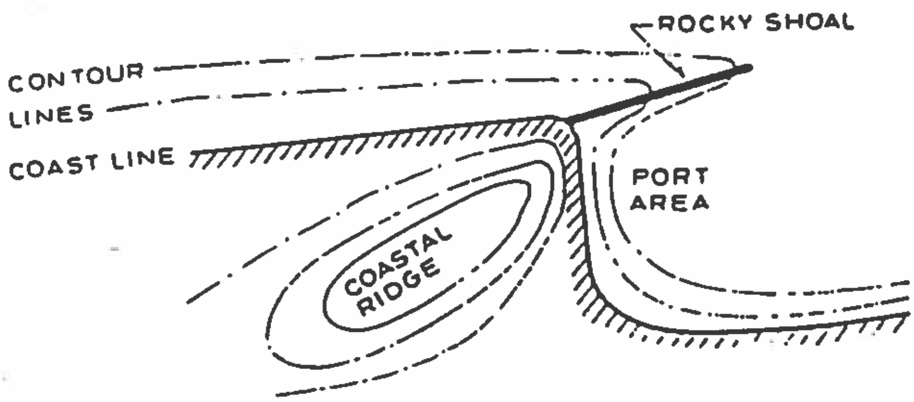
(b) Port Site Selection and Breakwater Alignment for Wide Wave Sector

Figur 26

Under tropiske forhold, hvor der ofte er bølger (dønninger) næsten kun fra én retning, kan der fordelagtigt udbygges fra et fremspring fra kysten (figur 26a).

Med bølger fra en bredere sektor og måske med sandtransport er det nødvendigt med to moler. Her er det pragtfuldt, hvis der i forvejen er to fremspring, der giver et passende havneareal. Især er det gunstigt, hvis fremspringene er klippefremspring med blød bund imellem, hvor det er billigt at uddybe (figur 26b).

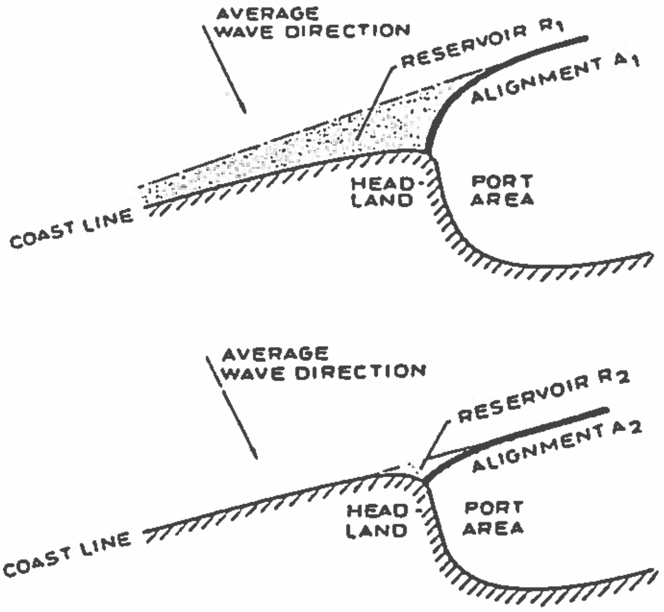
I en lærebog kan der jo sagtens tegnes ideelle situationer. I fig. 27 er der vist et bjergfremspring med tilhørende rev, som giver særlig gode forhold. Alt skal jo sammenlignes med f.eks. transportomkostninger for den malm, der skal udskibes gennem havnen.



Port Site Selection and Breakwater Alignment for Rocky Shoal and Coastal Ridge

Figur 27

En sandtransport på halve eller hele millioner m^3 pr. år er dyr at håndtere. I visse tilfælde kan ulempen dog vendes til en fordel, hvis der er tale om en uberørt kyst, der har et fremspring som vist på figur 28. Figuren viser to alternative moleudbygninger. Bølgerne fylder de viste reservoirer op med sand og fortsætter på en dønningskyst med aflejringer, der med tiden spærrer indsejlingen.



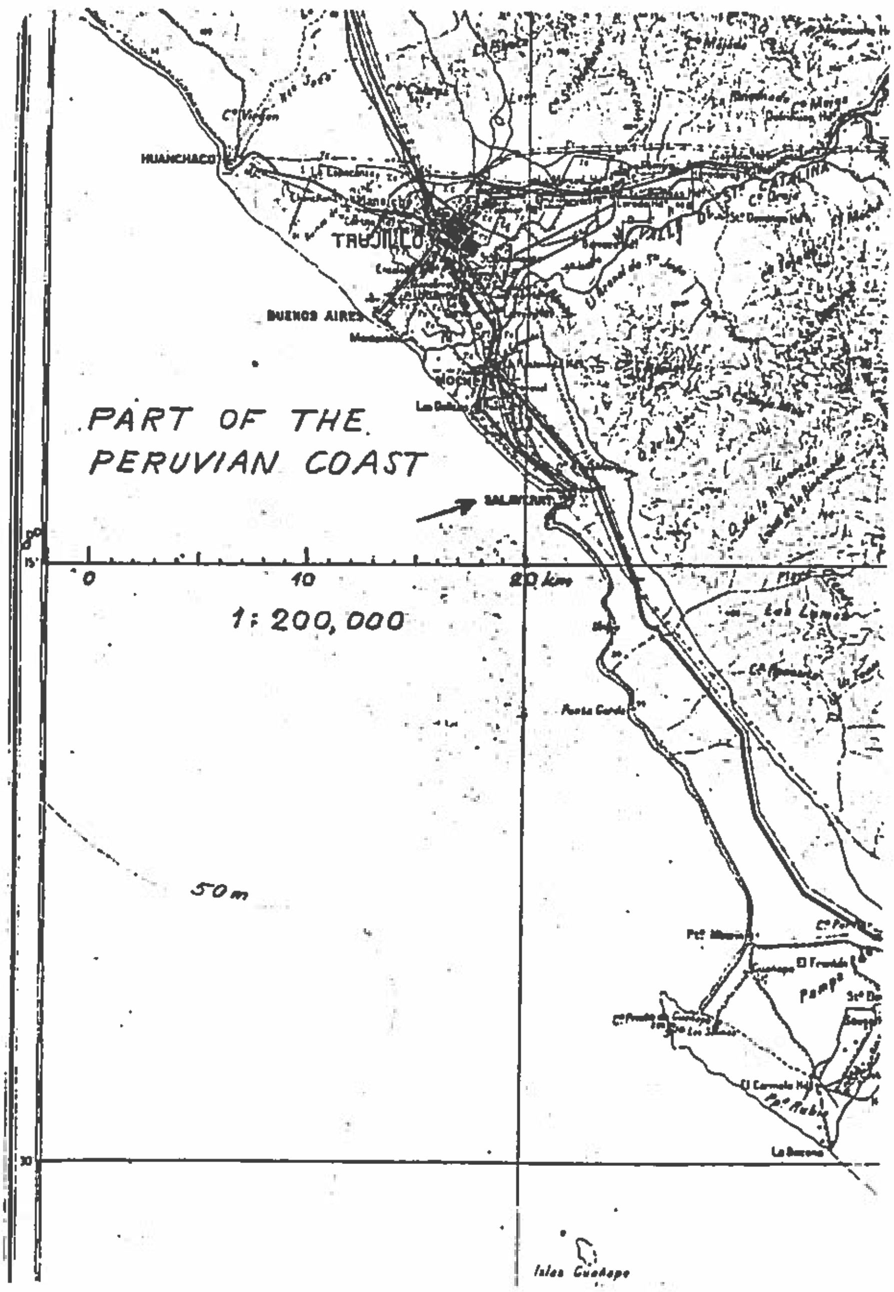
Breakwater Alignment for Coast with
LargeLittoral Drift.

Figur 28

Denne mekanisme kan bruges til fordel for havnebyggeriet, idet en placering af molen ved A_2 giver sandaflejringer efter udførelse af en første delfase af molen netop på de steder, hvor de næste faser skal bygges. Hvis tiden tillader en sådan strategi, kan molen altså bygges på langt mindre vanddybde ved en etapevis udbygning i takt med sandaflejringen. Der må derudover suppleres med en sandfanger der må forlænges hvert andet eller tiende år for at undgå tilsanding af havnen eller store udrensningsudgifter, men så vil der også opstå et godt terræn tæt på molen. Der ædes naturligvis kraftigt af kysten i læ af havnen, hvor der derfor ikke bør bygges over en passende lang strækning.

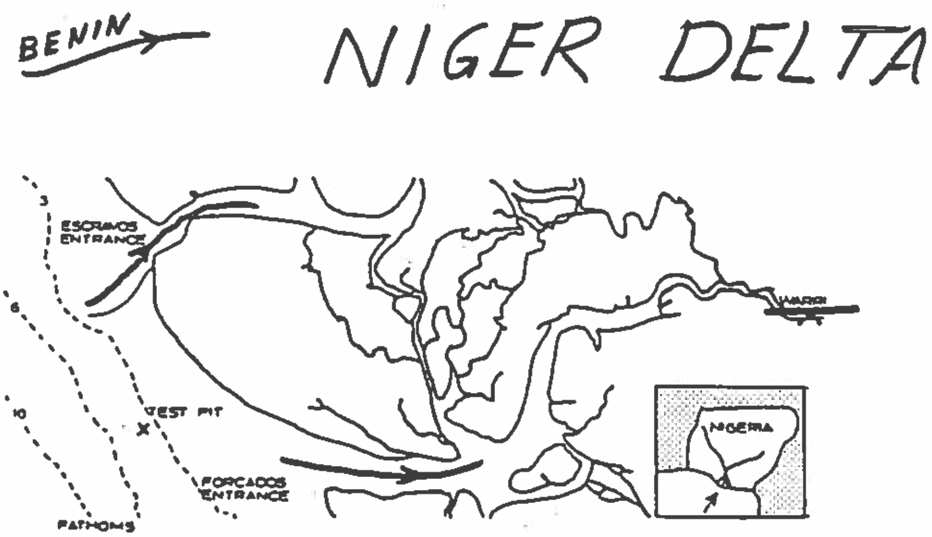
13. Salaverry, Peru

Denne mulighed fandtes ved Salaverry i Peru, hvor entreprenøren havde projekteret og bygget en mole på 10 m vanddybde på 2 år. Efter 4 års tid var der blot 4 m vanddybde ved molespidsen. Hvis arbejdet havde været bedre planlagt, kunne der have været bygget på ca. 5 m vand med omkostninger på ca. 1/4 af omkostningerne ved molebyggeriet.



Figur 29

14. Havneplaceringsvalg, Niger Deltaet



Figur 30

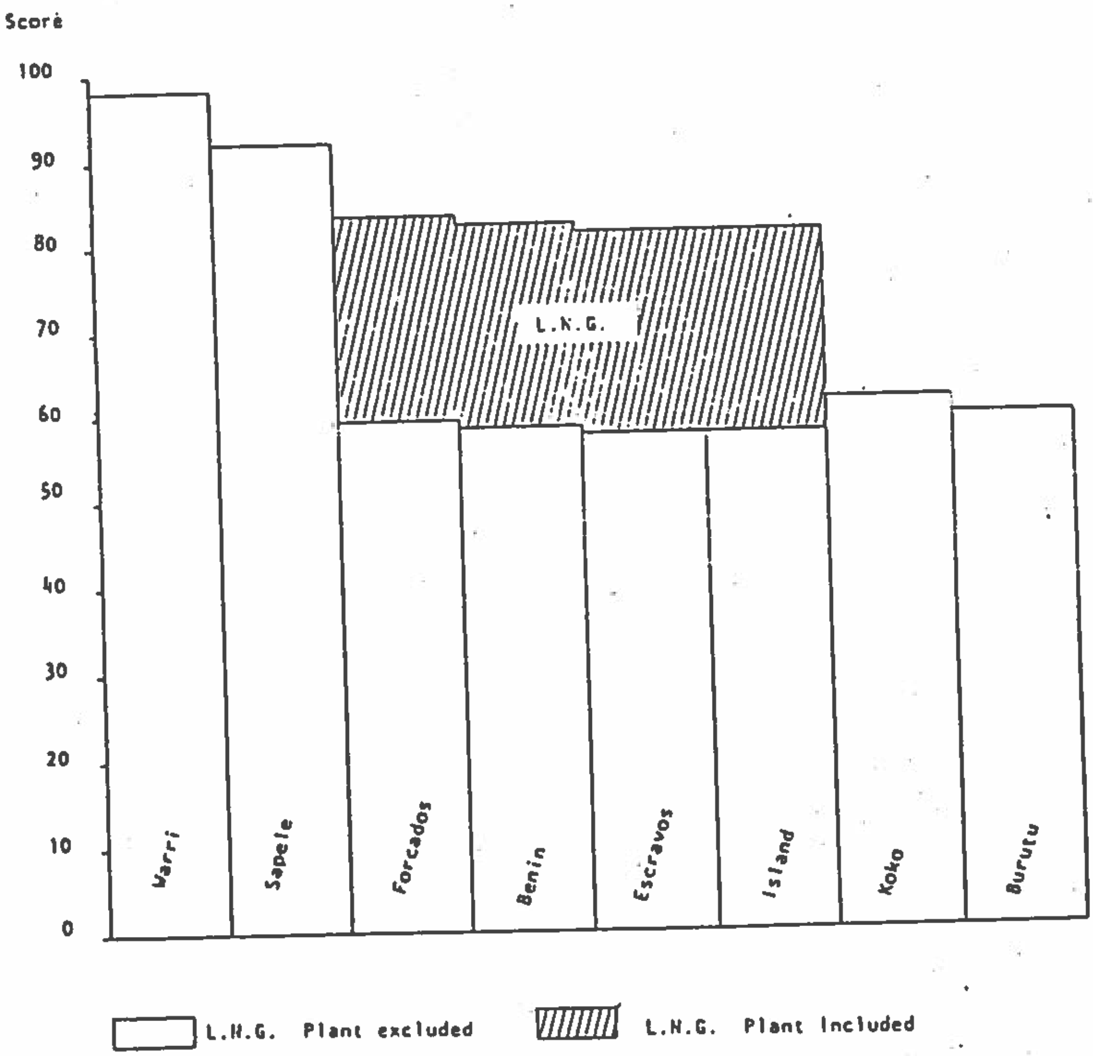
Derefter skal omtales et studium under medvirken af bl.a. Dansk Hydraulisk Institut i Niger Deltaet, hvor man i en meget omfattende undersøgelse skulle vurdere om Warri Port eller en af 7 andre lokaliteter i dette delta skulle udbygges. De øvrige 7 lokaliteter, som er placeret i forbindelse med 3 forskellige indgange til deltaet, er ikke vist på kortet. Resultatet af studiet fremgår af figur 31.

Figur 32 viser de anvendte omkostningsfaktorer, der indgik i analysen.

I analysen indgik naturligvis også markeds- og trafikprognoser. Der blev anvendt et karaktersystem med skalaen 1, 2 og 3. De forskellige faktorer blev bedømt efter denne skala. Maximum blev 102 karakterer (figur 32).

Warri Port fik flest point, nemlig 98 point af de 102 mulige.

Market & Traffic Benefits of Alternative Port Sites



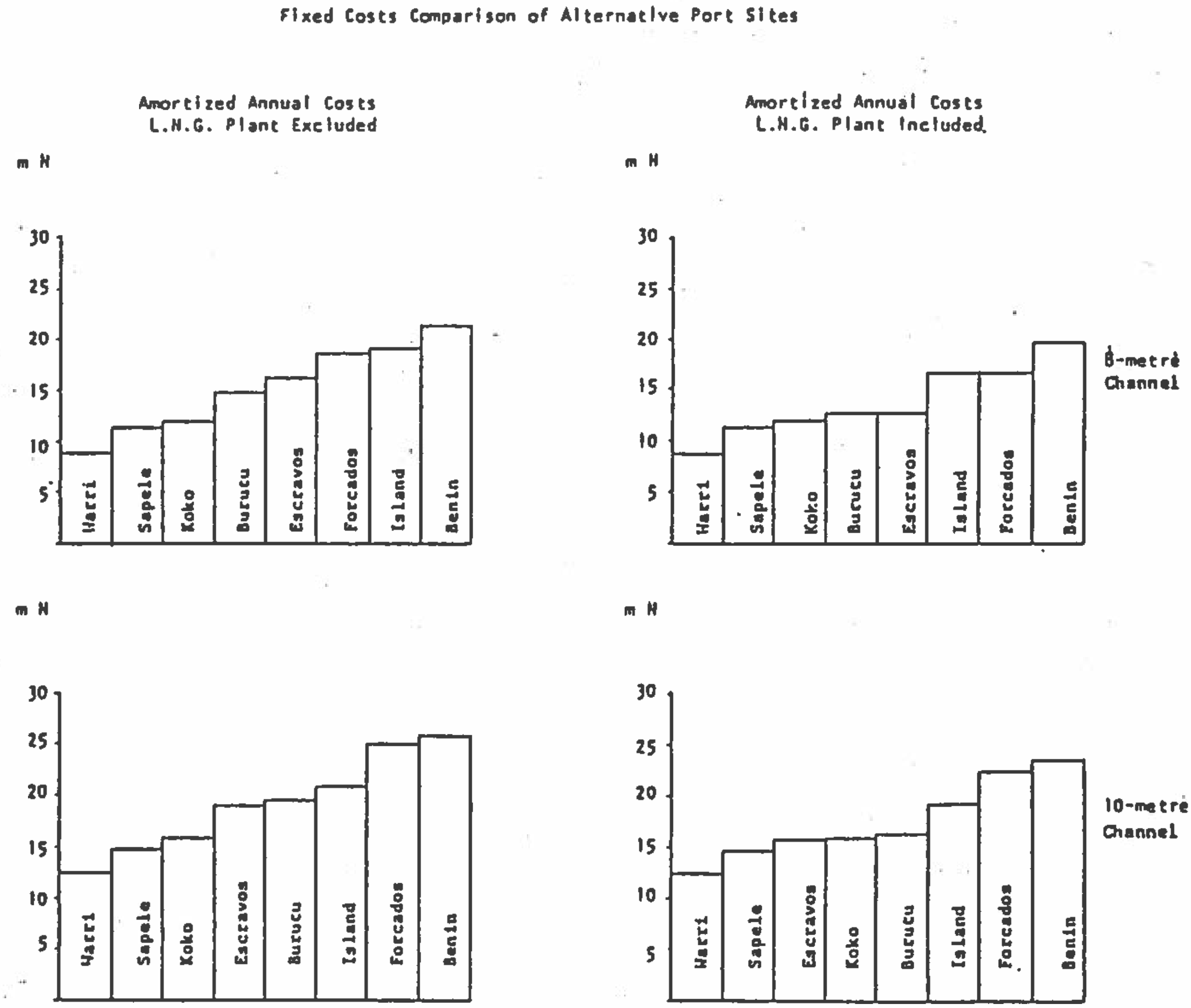
Figur 31

Market and Traffic Criteria in Port Site Selection:Expected Scenario

Proposed Port Site Area	Waterways (Rivers)	Facilities: Physical Plant: Port	Facilities: Physical Plant: Industrial	Facilities: Physical Plant: Commercial	Facilities: Physical Plant: Residential (Port Related)	Facilities: Physical Plant: Residential (Other than Port Related)	Facilities: Services: Social	Facilities: Services: Business Related	Facilities: Manpower Resources	Facilities: Total	Economic Activities: Public and Private Investments	Economic Activities: Port Traffic Generation: From the Vicinity of the Port-City Area	Economic Activities: Port Traffic Generation: From the Existing Hinterland	Economic Activities: Port Traffic Generation: From Beyond the Existing Hinterland	Economic Activities: Personal Consumption	Economic Activities: Total	Traffic Conditions: Access: To/From the Interior Via Inland Waterway	Traffic Conditions: Access: To/From the Interior Via Road	Traffic Conditions: Access: To/From the Sea: Improved Channel	Traffic Conditions: Access: To/From the Sea: Ease of Access & Distance	Traffic Conditions: Proximity to Market (Benin City)	Traffic Conditions: Favourable Meteorological Conditions	Traffic Conditions: Total	Grand Total	Percentage of the Maximum Score (34 x 3 = 102)
Weight		3	2	1	3	1	1	2	1	14	2	2	3	2	1	10	1	3	3	1	1	1	10	34	
Coastal Ports Excluding LNG Plant																									
Benin Entrance	Benin	3	1	1	3	1	1	1	1	26	1	1	2	3	1	17	2	3	3	2	2	1	25	68	67
Escravos Entrance	Escravos	3	1	1	3	1	1	1	1	26	1	1	2	3	1	17	2	3	3	2	1	1	24	67	66
Forcados Entrance (Burutu)	Forcados	3	1	1	3	2	1	1	1	27	1	1	2	3	1	17	2	3	3	2	1	2	25	69	68
Island	Off the Coast	3	1	1	3	1	1	1	1	26	1	1	2	3	1	17	1	3	3	3	1	1	24	67	66
Creek Ports																									
Burutu	Escravos	3	1	1	3	2	1	1	1	27	1	1	2	3	1	17	2	3	3	2	1	2	25	69	68
Koko	Escravos	3	1	1	3	1	1	1	1	26	1	2	2	3	1	19	1	3	3	1	3	3	26	71	70
Sapele	Escravos	3	3	2	3	3	2	2	2	37	3	3	3	3	2	29	1	3	3	1	3	3	26	92	90
Warri	Escravos	3	3	3	3	3	3	3	3	42	3	3	3	3	3	30	3	3	3	1	2	2	26	98	96
Coastal Ports Including LNG Plant																									
Benin Entrance	Benin	3	2	2	3	1	2	2	2	33	3	2	2	3	2	24	2	3	3	2	2	1	25	82	80
Escravos Entrance	Escravos	3	2	2	3	1	2	2	2	33	3	2	2	3	2	24	2	3	3	2	1	1	24	81	79
Forcados Entrance (Burutu)	Forcados	3	2	2	3	2	2	2	2	34	3	2	2	3	2	24	2	3	3	2	1	2	25	83	81
Island	Off the Coast	3	2	2	3	1	2	2	2	33	3	2	2	3	2	24	1	3	3	3	1	1	24	81	79

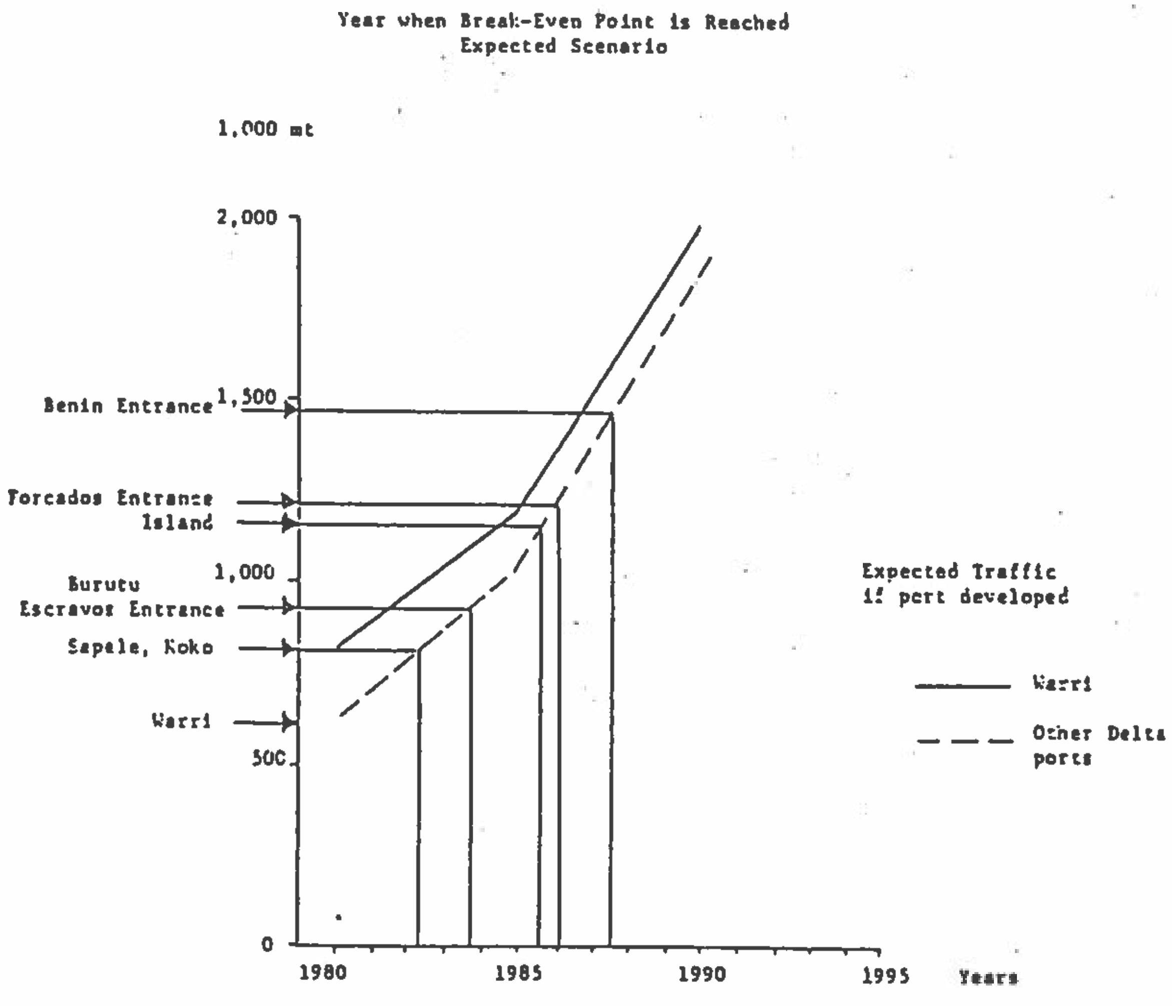
Figur 32

Det er klart, at der indgår skønsmæssige vurderinger af betydningen af de forskellige faktorer. Men analysen blev fortsat med en egentlig omkostningsanalyse med resultateter som vist i figur 33.



Figur 33

Tilsvarende blev det beregnet, efter hvor lang tid de forskellige alternative udbygninger ville blive betalt tilbage (figur 34).

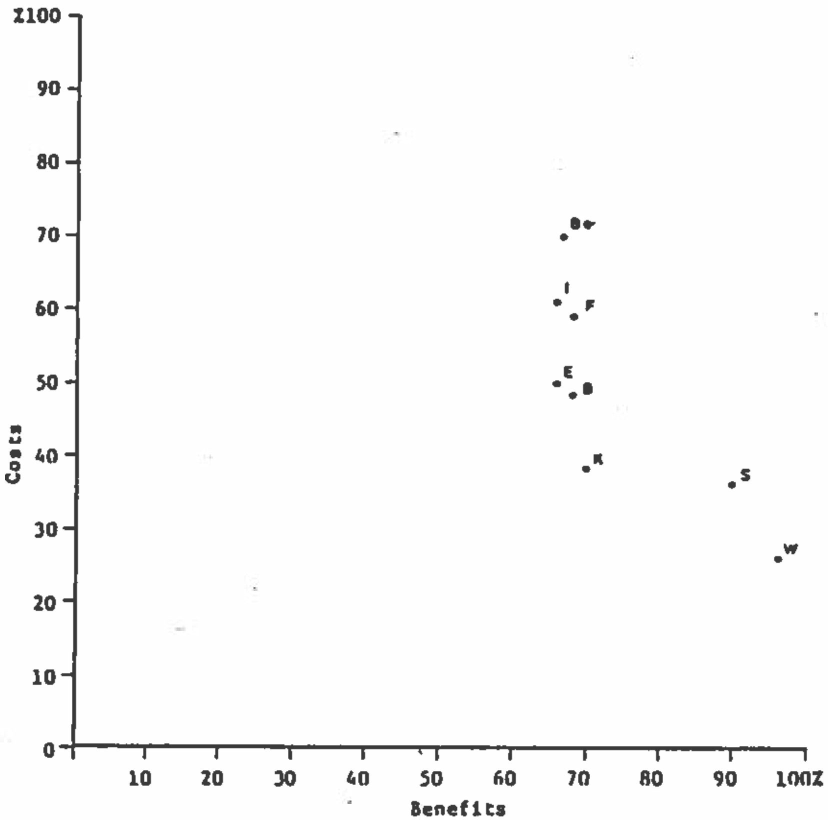


Figur 34

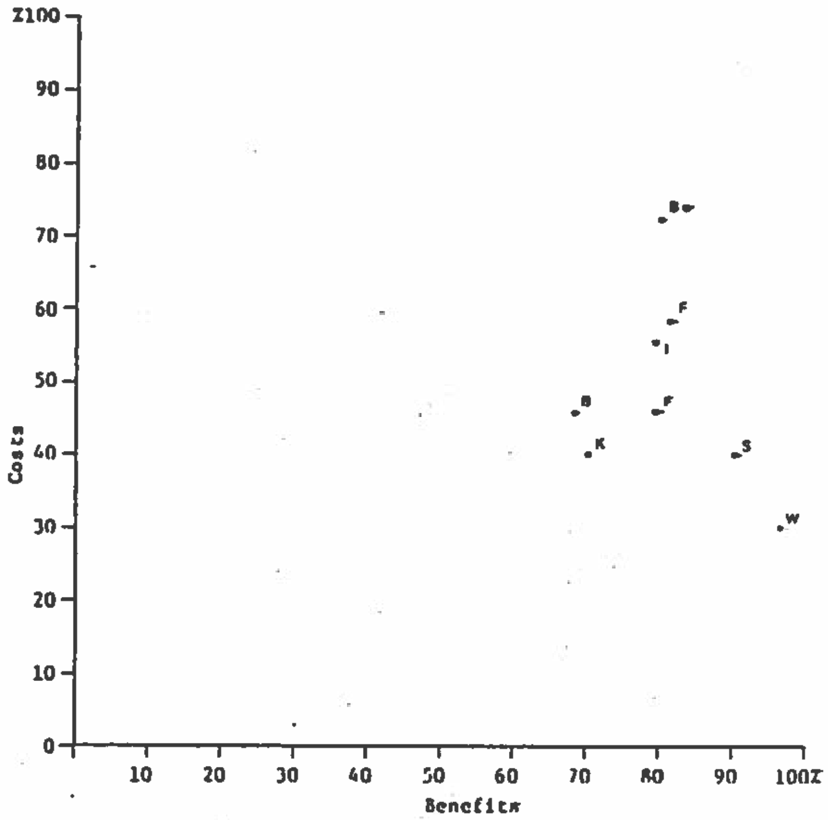
Endelig blev der opstillet et cost benefit forhold som vist på figur 35.

Placeringen ved Warri var i alle analysetyper den gunstigste.

Delta Ports Comparison
Fixed Annual Losts Versus Market & Traffic Benefits
8-metre Channel



L.N.G. Plant Excluded



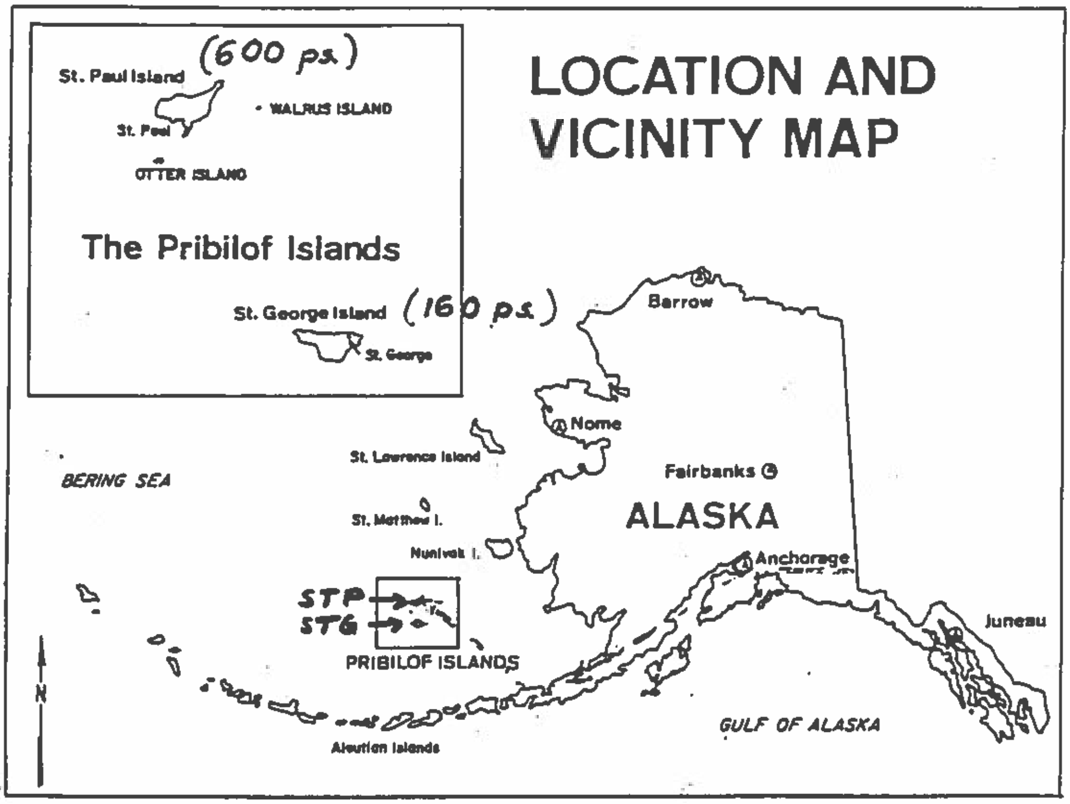
L.N.G. Plant Included

B_e	Benin Entrance
B	Burutu
E	Escravos Entrance
F	Forcados Entrance
I	Iwland
K	Koko
S	Sapele
W	Warri

Figur 35

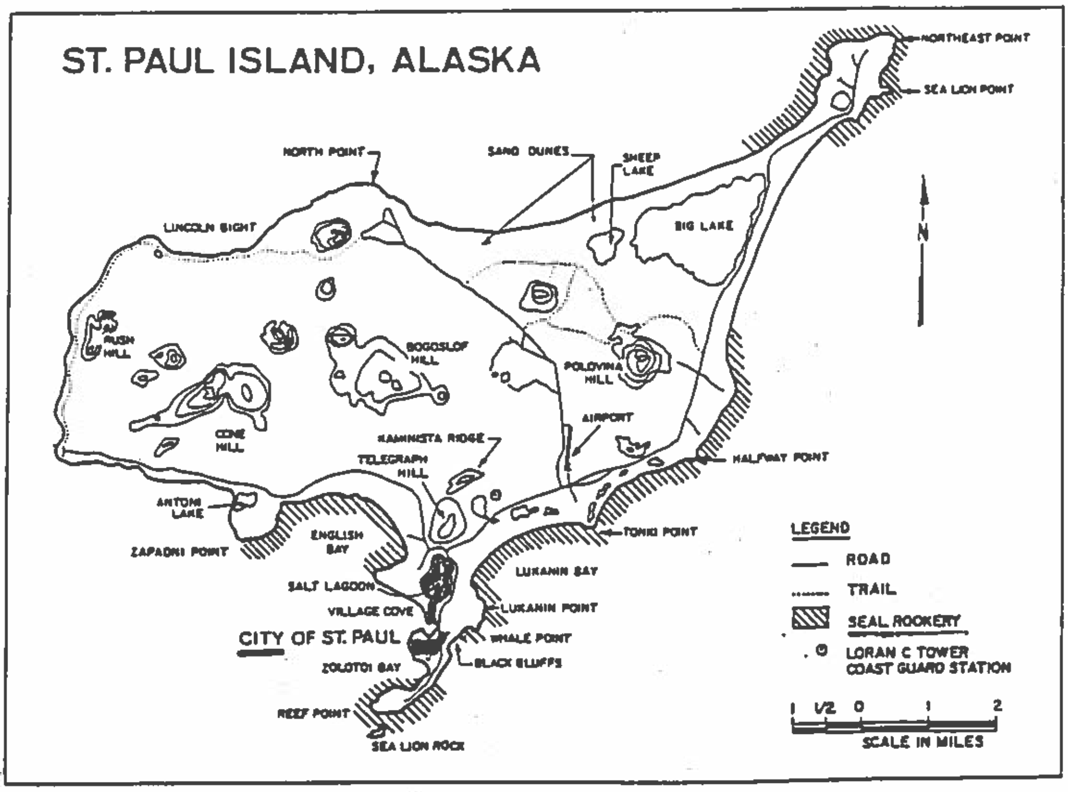
15. Pribilof Islands, Beringhavet

Som et andet eksempel på placering af fiskerihavne kan nævnes projekterne for Pribilof Islands i Beringhavet ved Alaska. (Se figur 36).



Figur 36

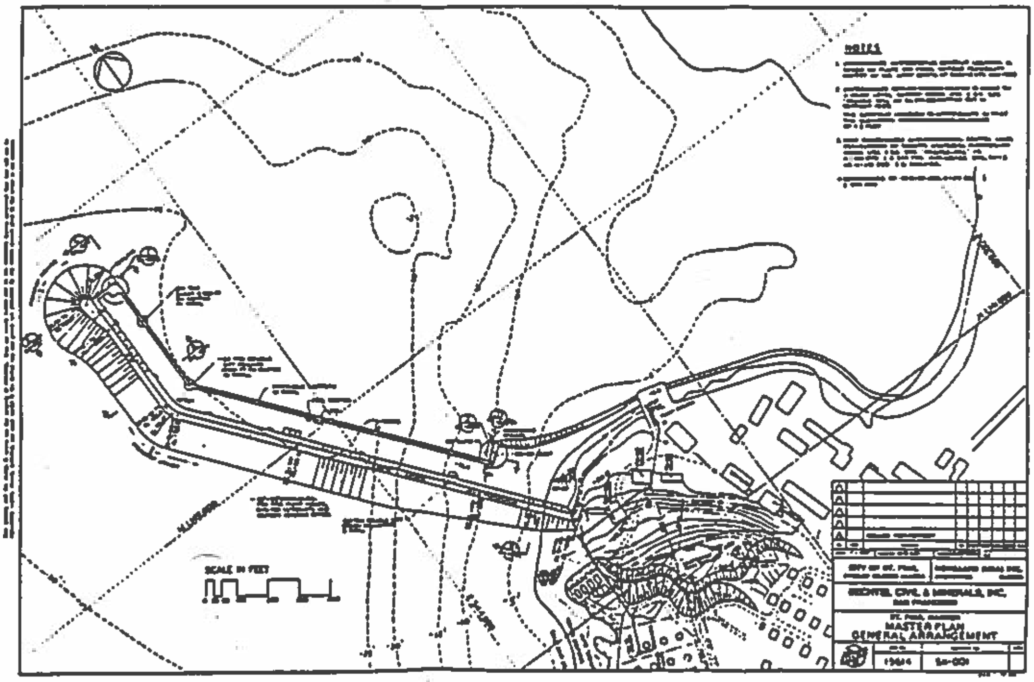
Der bor kun henholdsvis 600 og 160 personer på de to øer. På øerne er der i avletiden 1,7 millioner pelssæler, som hidtil har spillet en dominerende rolle for øernes økonomi. Men dette er ikke så profitabelt mere. Udenfor øerne fiskes der meget af fremmede fiskere, idet området er et af verdens rigeste fiskefarvande. For at opnå, at den lokale aleutbefolkning kan overleve, skal der bygges to fiskerihavne. Der vil være fiskeriøkonomisk basis for en havn af Hanstholm størrelse på hver af øerne.



Figur 37

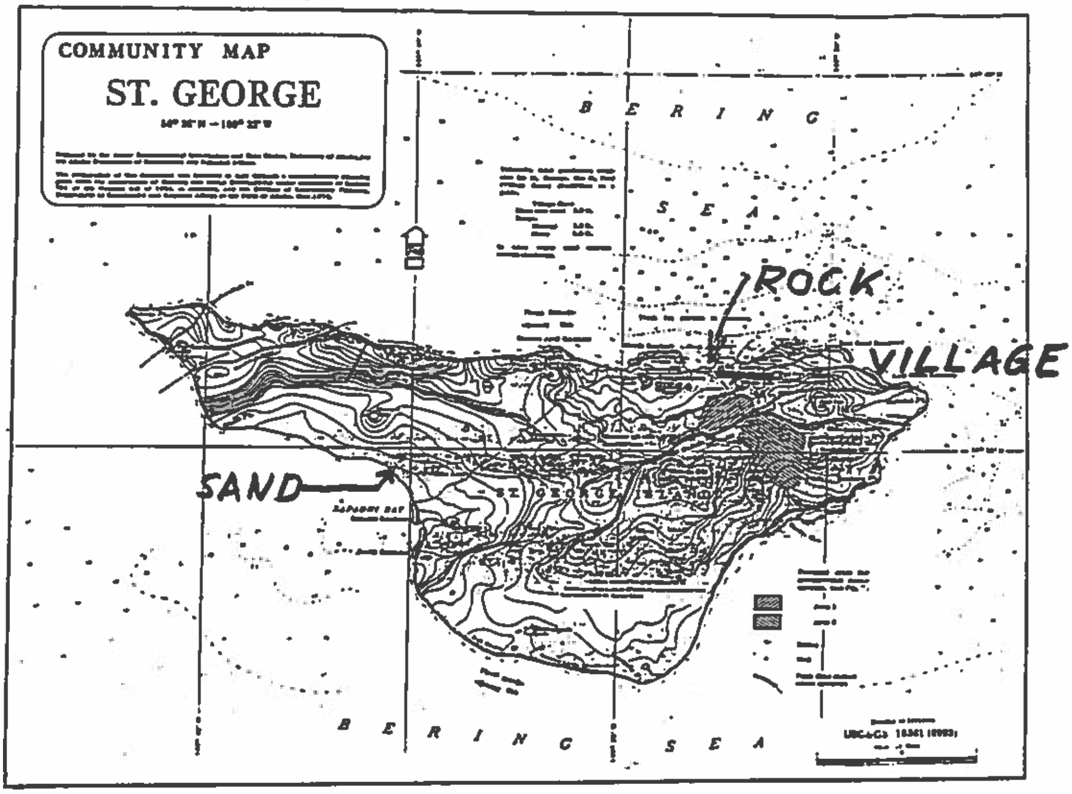
Den vulkanske ø St. Paul ses på figur 37. På kortet ses Salt-Lagoon. Hvis der blev bygget en lille mole her til dækning af en gennemskæring af tangen, kunne der indrettes et meget billigt havneanlæg i lagunen. Men dette er ikke tilladt på grund af en forekomst af sjældne fugle, så lagunen er totalfredet inklusive de omkringliggende klipper. Tilsvarende er de skraverede områder, hvor sælerne avler, også fredede.

Der måtte vælges en havneplacering i Village Cove nær selve byen St. Paul (fig. 37-38).



Figur 38

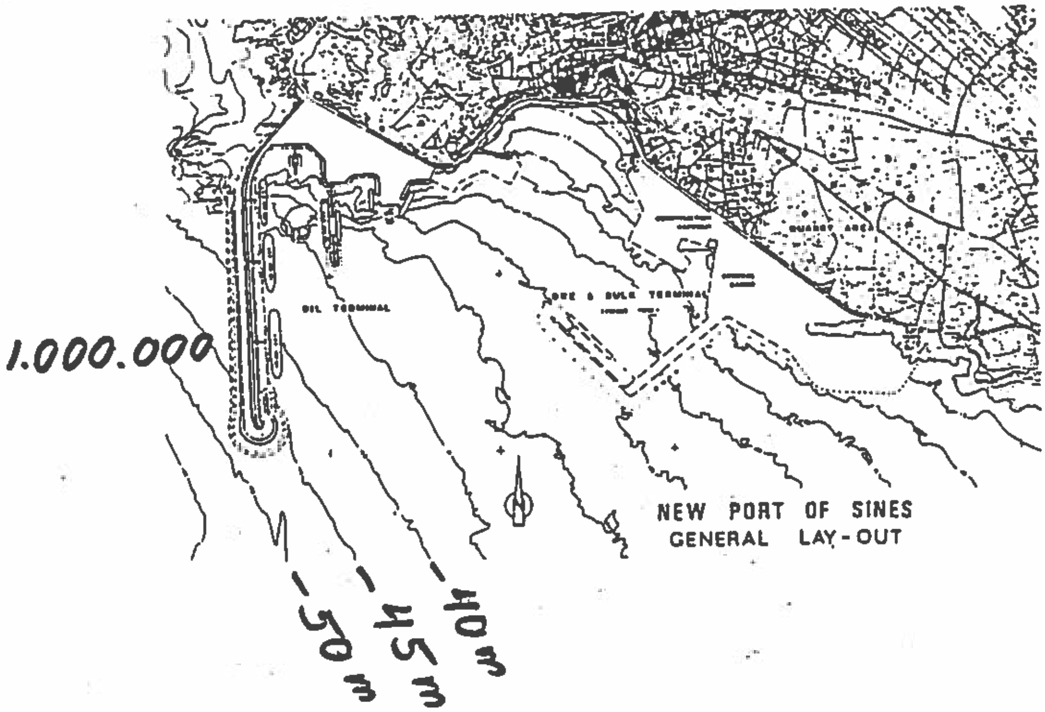
Projektet omfatter en stor mole samt delvis bortsprængning af et bjergparti "The Village Hill", der vil give materiale til molen samt skabe et større bagland. Studiet for projektet omfattede også arbejde af en antropolog, der skulle beskrive projektets sociologiske indflydelse på lokalsamfundet. Dette aspekt bør tages med i alle større projekter. På denne ø vil der komme mange koreanske, japanske og andre søfolk ind på en ø med en i forvejen dominerende mandlig befolkning, så man må holde fiskerihavnens aktiviteter adskilt fra byen lidt længere mod øst!



Figur 39

Den anden ø, St. George, ses på figur 39.
Der er fjeldpartier tæt på den eksisterende by, som ville give dyre uddybninger ved en havn valgt her. I den sandede Zapadni bugt er der meget større fremtidsmuligheder også på grund af baglandets form. Sandvandringsproblemet er begrænset til ca. 10.000 m^3 årlig oprensning.

16. Sines, Portugal

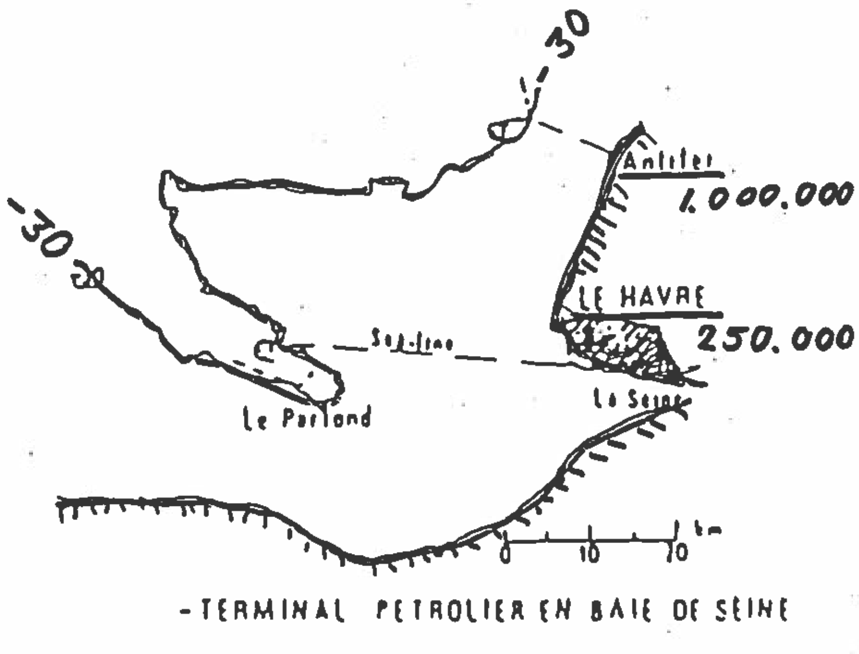


Figur 40

Ønsket om industriudvikling var begrundelsen for at bygge den store Sines havn i Portugal. Den sydlige del af Portugal, som er landbrugsdomineret med en meget lav levestandard, skulle gives en industriudvikling først og fremmest ved et raffinaderi og tilhørende petrokemisk industri og anden mineralindustri. Den kæmpestore mole der går ud på næsten 50 m vand ses på figur 40. Havnen skulle kunne tage skibe på op til 1 mill. tons. Overfor den store oliepier skulle der bygges en massegods terminal.

17. Antifer, Frankrig

Til slut kan nævnes et stort nyt havneanlæg i Frankrig. Le Havre kunne tage skibe op til 250.000 tons, men 70'er optimismen krævede anlæg op til 1 million tons. På figur 41 er vist 30 m kurven og det var derfor klart, at man måtte finde et sted med kort afstand til denne dybdekurve. Dette bestemte placeringen af havnen ved Antifer.



Figur 41

19. Redigering af H. Lundgrens Indlæg

Det i det foregående præsenterede indlæg er udarbejdet på grundlag af foredraget og redigeret af civilingeniør Helge Gravesen.

TWO LARGE PORTS IN THE MIDDLE EAST
CONCEIVED AND BUILT WITHIN A 2-YEAR PERIOD

by

Hans Hartelius, Chief Engineer, M.Sc.
Rambøll & Hannemann, Consulting Engineers and Planners
(Member of Dangroup International)
Denmark

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ABSTRACT

The paper briefly describes the way in which two large ports in Saudi Arabia, each one including 10 berths and all necessary additional facilities, one located at the Red Sea coast and the other located at the Gulf coast, were planned, designed, constructed and commisioned, ready for use in only 2 years time, including site investigations, hydraulic model tests etc. The first two berths of each port were ready for use only about 15 months after the order to proceed with the initial planning.

A brief outline of the case is as follows: In connection with the approval of a large housing programme, the government decided to construct two complete, self contained ports for the import of construction materials. The ports were to be constructed in undeveloped desert areas. In order to comply with the time schedule of the housing programme, it was necessary to establish the ports in the short time mentioned above. In order to meet the time schedule the following procedure was adopted:

- Planning and design were initiated in parallel with site selection, model tests, etc. The running design was adjusted according to incoming results from site investigations, tests, etc.
- The two first berths of each port were constructed as floating pontoon jetties, build in a ship yard and towed to the sites where they became operational after few days installation work.

- The tendering for the civil works, buildings, utilities and equipment was based on complete programmes for the facilities, main lay-outs, principal design of main structures and site investigations sufficient to be sure of the main lay-outs and construction principles. Complete conditions, technical specifications and bills of quantities covered all types of work.

- Where necessary more detailed site investigations (e.g. soil investigations) were included in the construction contracts. Elaboration of the detailed design and the construction drawings for buildings and utilities was executed by the contractors.

In order to give an impression of the nature of the projects, a brief outline of the projects and special technical problems encountered is given. The total investment in the two ports was 450 mill. U.S. dollars at the price level of 1976-78.

1. INTRODUCTION

The author was design project manager for the port at the Red Sea Coast and resident project manager in Saudi Arabia, for the construction of both ports.

The background for the construction of the two large ports within a very limited time, as described in the following, was a sudden increase in the need for import of construction materials to cover the demands of a vast housing programme to be initiated by the government.

At the time of approval of this programme it was realized that the prevailing congestion of the major ports would be a serious obstacle for the programme.

It was initially envisaged, that each main contractor bidding for sections of the housing programme would have to construct his own landing facilities to import the necessary materials. However, it was soon realized that it would be more rational if the government were to construct two new ports for this purpose, conveniently located in relation to the future housing sites: one at the Red Sea coast and the other at the Gulf coast.

The planned start of the crash housing programme made it necessary to realize the ports within a very short period.

The first berths had to be operational within the shortest possible time, preferably only about 1 year after the decision to commence the initial studies for the ports, and the complete ports should be operational about 2 years after the commencement of the studies.

In the following it is shown how these goals were obtained through an approach where the client, through his consulting engineer, kept full control of, and influence with, all details of the project. This was achieved even though it was necessary to leave part of the detailed design to be carried out by the contractor concurrently with the construction, and leave certain design decisions and the specification of part of the equipment to be procured under the main contracts till after the contract award.

2. A SHORT DESCRIPTION OF THE PROJECTS

2.1 General Remarks

The two new ports were constructed in completely undeveloped desert areas and therefore they had to constitute complete self-contained units with accomodation facilities for the staff and work force, and all utilities and services to keep the ports operating.

The total construction cost of the two ports was about 450 million U.S. dollars, at the price level of 1976-78.

2.2 Location of the Red Sea Port

The port at the Red Sea Coast is constructed in a natural lagoon in the large coral reef area along the coast. The lagoon is perfectly sheltered from waves by a reef, which is penetrated by a natural entrance channel of 300 m width and 30 m depth. The lagoon forms a perfect natural harbour with a sandy bottom, and the depth gently decreases from 30 m at the entrance at the reef to a depth of 10 m at the northern berths. From the opening in the protecting reef, a marked navigation route of 25 km length leads through the outer reefs to the open sea.

2.3 Port and Ancillary Facilities

The lay-out of the port is shown in figure 1. The port has 10 berths with 10 to 12 m water depth for 10.000 - 30.000 dwt ships. Six berths are constructed as floating pontoon piers, and 4 berths are located along a marginal wharf. There are 2 ro-ro ramps and rail foundations for a future bulk-unloader.

Quays and storage areas are laid out on reclaimed areas on a coral reef. At the landward side of the ship terminal the entrance area is laid out, containing administration buildings, authorities offices and utilities: a 10 MW diesel power plant with oil storage tanks; desalination plant (1000 m3/day) with water intake; sewage treatment plant. About 3 km from the port facilities, the housing area is located with accomodation buildings and family houses for 1500 persons, canteens, shops, recreational buildings etc. The port is thus completely self-contained with its own water and power supply, welfare facilities etc. Telephone and telex connections are secured by a microwave link to the nearest public telephone excange 100 km away. The port also has a ship-to-shore radio station and a walkie-talkie system for communication within the port area.

The port is completely equipped with cargo handling equipment in the form of fork lift trucks including large units for container handling, tractors and trailers, service vehicles in the form of fire engines, ambulances, buses for personnel transportation, and has fully equipped maintenance workshops and spare part stores for vehicles, utilities and buildings. Land access is secured by a two lane road, leading to the nearest main highway, and a helicopter landing pad is also provided.

2.4 Lay-out of the Gulf Port

The Gulf coast location of the other port is of a completely different nature to the Red Sea coast. It is an almost straight sandy beach with gently increasing water depth, reaching 10 to 12 m depth about 5 km from the coastline.

The lay-out of the port is shown on fig. 2. In this case the ship terminal is located about 3 km offshore where the natural water depth was about 8 m, and a navigation channel and harbour basin with 10-12 m water depth has been dredged. The ship terminal is connected to the shore by a causeway, and protected against the waves from the north and the north-west by a rubble mound breakwater.

The land facilities and utilities etc. are similar to those described above for the Red Sea port.

A 50 km long navigation channel marked with buoys leads from the open waters of the Gulf between banks and shoals to the mouth of the dredged entrance channel to the port.

3. GENERAL PLANNING AND PROJECT ORGANIZATION

3.1 Overall Time Schedule

The Generel time schedule for the project including all activities from the start of the initial planning to the completion of the construction works is shown in fig. 3.

It is seen that the time from the commencement of the planning to the issuing of the tender documents was only 3 months, the tendering took place approx. 4½ months after the start, and construction contracts were awarded a little more than 7 months after the commencement of the planning.

The construction was divided into two phases, to enable the port operations to start early. As indicated on the schedule the first phase became operational with 2 berths including the necessary storage and ancillary facilities 7 months (8 months) after start of construction for the Gulf Port (Red Sea Port).

The second phase, the total completion of the contract works for the 10-berth ports was accomplished within a construction period of 13.5 months (16.5 months) for the Gulf Port (Red Sea Port).

Thus the two ports were completed and fully operational only 21 months, respectively 24 months after the initiation of the planning and design of the projects.

3.2 General Principles of the Implementation

In order to meet the very tight overall time schedule the following principles were adopted:

- Types of structures: The first two berths of each port were constructed as floating pontoon jetties of steel, built in a shipyard and towed to the sites, where they became operational after a few days installation work.

The other quay structures were back-filled anchored steel sheet pile walls with quay areas paved with asphalt. Where the soil conditions permit it this type of quay is considerably quicker to construct than concrete block wall quays, caisson types or concrete deck structures on piles.

- Principles of the implementation: The tendering for the construction and civil works was based on complete programmes for the facilities (capacities, operational demands etc.), main lay-outs for the facilities, and principal detailed design of the major civil works. The requirements were specified in: General and Special Conditions, Technical Specifications and detailed bills of quantities covering all types of work, and items to be procured by the contractor.

The site investigations during the design phase were carried through to such an extent as to ensure that the main lay-out was feasible and the main structural types selected were suitable. Where necessary, supplementary site investigations were included in the construction contracts, and final decisions on detailed lay-out or structural questions were to be taken a couple of months into the construction period.

Detailed design of buildings and utilities was to be carried out by the contractors and approved by the consultant in the first part of the construction period, in parallel with the mobilization and start of civil works.

- Planning of Consultant's work: In order to complete the tender documents within the very limited time available, it was necessary to carry out the various consulting activities almost simultaneously, instead of arranging them in a sequence as normally adopted for such projects.

Under the Project management team a number of project groups were formed, each working with a certain part of the project or field of specialization needed within the project.

All groups worked with a maximum of individual responsibility, but coordinated in their efforts by the project management. Shortly after the start of the initial planning, the initial site reconnaissance, analysis of port operation, transport study and conceptual design studies were initiated. A little later bathymetric, topographical and geotechnical site investigations and hydraulic model tests were started, and shortly hereafter the detailed design and preparation of tender documents began.

Relevant information was exchanged between the project groups at frequent, regular project meetings, and the design basis for the groups was subject to a running adjustment according to results from the work of the groups and incoming results from site investigations, model tests etc.

After issuing the tender documents, some additional site investigations and design calculations were performed in order to secure that the basic designs were feasible, before the construction contracts were finally negotiated.

During the first part of the construction period, final decisions on a number of design aspects were taken, following the additional site investigations performed by the contractors, and checks and approvals of contractors' designs of buildings and utilities were carried out.

During the whole of the construction period, project management and construction supervision was carried out.

3.3 Project Organization

The general project organization is shown in figure 4.

The main project office was located in Copenhagen at the main office of Ramboll & Hannemann and the project groups were manned with engineers and technicians from the company and from the other member companies of Dangroup International and associated companies. Site investigations and hydraulic model tests were carried out under the responsibility of Danish Hydraulic Institute.

The main office project staff was mobilized very swiftly, reaching its maximum of 55 staff members working full time on the project about 2 months after the start of the design. In addition at this time about 40 specialists were occupied part-time with the project.

During the construction period, the project management and supervision staff counted a maximum of 30 staff members at the construction sites and the resident project management office in Saudi Arabia. This staff was backed up by a home office support staff of about 10 persons.

As indicated on the organization diagramme, specifications for port operations and maintenance were also prepared, tenders were called for these services from relevant service contractors, contracts were awarded, and for the following years, the consultant also supervised for the client port operations and maintenance.

4. PRELIMINARY STUDIES AND SITE INVESTIGATIONS

4.1 Initial Planning

The overall planning of the project was carried out as already outlined in the previous chapter and design and study activities which could start without detailed knowledge of the site conditions, or which should be used as input to the site selection were initiated, e.g. analysis of the housing programme in order to determine quantities and types of import cargo, general operational planning of the ports, design of floating pontoon jetties, land transport analysis.

4.2 Initial Reconnaissance and Site Selection

For the Gulf Port, a specific site was pointed out by the Client, due to the fact that the coastline of the entire zone, relevant for the housing programme was rather heavily utilized, or planned to be utilized for other port facilities, petrochemical plants etc.

Reconnaissance of the area indicated that the location was well suited to an offshore location of the ship terminal avoiding large dredging quantities in the shallow coastal area. A location of the storage area, administration etc. on the coast avoided extensive reclamation work in the ship terminal area with a natural water depth of 7 to 10 m.

For the Red Sea Port the site was not pointed out beforehand. A coastline several hundred kilometres long stretching north and south of Jeddah could provide a suitable location for the port. The enormous system of coral reefs gives many problems for navigation near the coast, but it also renders good possibilities for finding natural sheltered water for a harbour.

The first reconnaissance along the coast was made from a small aircraft. From the air 3 locations were spotted as possible port locations. Each of the locations had reasonably deep water near to a coral reef area which could be the basis for quay and storage areas, but the degree of shelter provided by the outer reefs and the ease of access both from the sea passing between the outer reefs, and from land over shallow water areas, varied between the locations.

Air photos were obtained covering the three different locations, and reconnaissance from land was performed. Based on the information gathered, preliminary projects and comparative cost estimates were prepared for the three locations.

A transport-economic study comprising an estimated 6-year housing programme to be supplied from the port was conducted, covering the three locations.

Finally the northern site was selected as the optimal location, all aspects taken into consideration.

The selected location did not provide the lowest capitalized transport cost, but the two other sites had higher capital costs. The first site was at a less sheltered location, making an outer breakwater necessary, and the other side had higher dredging costs due to the navigation route and area, and had a long causeway over a shallow water area.

The coral lagoon at the selected location has from 1898 been mentioned in the British Admiralty's Red Sea Pilot as an excellent natural harbour, well sheltered by the barrier reef, with a wide and deep approach channel, ample water depth in the bassin and good sea bed for anchorage.

4.3 Site Investigations and Model Tests

For each of the selected locations, preliminary lay-outs were sketched, as a basis for the planning of bathymetric and topographical surveys and soil investigations: geotechnical borings and seismic subbottom profiling. These investigations were planned with the aim of covering the necessary areas, with a reasonable margin for adjustments during the design.

Through the bathymetric survey for the Gulf Port, two favourable locations were discovered, the first a basin with 10-11 m water depth located approx. 4 km from the shore, and the second a basin with approx. 8 m depth located about 2,5 km from the shore.

The geotechnical investigations revealed sound firm soil, mainly sand and limestone, and a hard layer of caprock of varying thickness (from 0.2 m to more than 1.0 m) over the whole area.

At the Red Sea Site, the bathymetric survey confirmed the favourable water depths of the lagoon. Only a few isolated coral reefs had to be dredged away.

The geotechnical investigations revealed that the material inside the coral reefs had very varying density, which could make pile foundations of heavy structures on the reefs necessary. Furthermore it was found that the coral sand at the bottom of the lagoon down to level - 20 m to - 30 m was very loosely deposited, and slightly cemented, and consequently the sand structure might collapse and the sand liquefy under the influence of vibrations or increased loads.

Investigations of the seismic conditions of this region revealed that a potential risk of earthquakes of a certain magnitude is present, thus presenting a danger of liquefaction of the loose sand deposits.

The wave conditions at the sites were evaluated. At the Red Sea Site no wave problems were encountered in the lagoon, but at the open Gulf Site the conditions had to be carefully analyzed.

Wave conditions were measured for a period by a waverider buoy located in the ship terminal area, and storm waves hindcasted from selected storm situations.

A large model to scale 1:80, of the ship terminal with its breakwater was constructed at the laboratories of Danish Hydraulic Institute in Denmark.

In this model the mooring conditions of vessels at the berths were tested for different breakwater lay-outs, and the movements and mooring forces of the floating pontoon pier were measured.

Flume model tests for optimizing breakwater and causeway designs were also carried out.

5. DESIGN AND TENDERING

5.1 Operational Analysis

Parallel with the above described site investigations, model tests and preliminary design work and analysis of port operation was carried out.

The basis for the analysis was the plans for the housing programme, which were used for estimating the yearly quantity of cargo to enter the ports, and the distribution over types of cargo, weight per unit etc. It was decided to design the ports for an annual quantity of cargo of 2 million tons for each port. Hereof the main part is foreseen as general cargo and containers, whereas 500,000 t for each port is foreseen as bulk cargo, e.g. cement clinker for a future cement mill at the port area.

Based on the operational analysis the capacities of all main and ancillary facilities were determined: i.e. number of berths necessary to obtain a sufficiently short average waiting time for the ships, necessary manpower (stevedoring gangs etc.) and equipment to unload the ships in an appropriate time, necessary facilities to shelter and feed the manpower and maintain the equipment, administration facilities and facilities for the necessary authorities in the ports.

Actually 6 berths at each port were found to be sufficient for the mentioned quantities, but the client later decided to increase the number of berths to 10 at each port, to cover possible future needs.

5.2 Conceptual Design

Based on the initial planning, the site reconnaissance and investigations and the operational analysis, conceptual design reports were prepared for each of the ports, comprising descriptions, drawings, tables and item lists covering all aspects of the entire projects.

These reports made it possible for the Client, to check the projects before the completion of the detailed design and tender documents, and hereby make sure that the documents would meet his requirements.

The approved conceptual design reports formed the basis for the detailed design.

5.3 Detailed Design and Tender Documents

At the Gulf Site, as mentioned under 4.3 above, two favourable locations of the offshore ship terminal were located.

The first location would entail practically no dredging work but a 4 km long causeway at up to 11 m water depth. The second location would entail a rather large quantity of dredging work, but a causeway of only 2.5 km and only up to 8 m water depth.

The imprtant factors which would determine the choice between the two alternatives were: The exact nature of the sea bed, the contractor's access to different kinds of dredging equipment and hereby the cost for dredging work, the contractors access to suitable quarry materials and capacity for moving such materials, and hereby the costs for execution of reclamation and stone protection works.

It was finally decided to include both alternatives in the tender documents, and an extended soil investigation programme was also included in order to determine exactly the nature and exent of the cap-rock and other layers below the sea bed.

The Client would have to decide between the two alternatives at the tender evaluation, or he could wait untill the additional soil investigations were carried out.

For the Red Sea Site, the evaluation of the geotechnical investigations, indicating the aforementioned deposits af very loose coral sand, and in the southern project area furthermore extensive soft clay layers under the sand layers at the sea bottom, led to a decision to move the ship terminal northwards on the reef selected as the site. Furthermore the possibility was included of reducing the length of fixed quaywalls and introducing more floating pontoon jetties, if this solution turned out to be economic after the tender prices were received.

The results also led to the introduction of vibro compaction in the specifications and bill of quantities. This method is a way to strengthen and densify the soil layers in situ and it has been used on many occassions as an alternative to complete exchange of poor soil layers, resulting in considerable savings in construction costs.

An extensive soil investigations programm was included to be performed within the first 60 days of the contract period. This programme also concluded trial vibro compaction, in order to test the effectivity of the method on the existing soil, and a large number of dynamic soundings to determine the depth to the top of the firm layers.

The contract documents further stated that final instructions regarding the execution of the marine works would be issued 90 days after contract date, leaving 30 days for the evaluation of the detailed soil investigations and the decision regarding the method of execution of the marine works.

To cover the execution of these works, rather big quantities of soil exchange work and vibro compaction work at the seabottom were included in the bill of quantities.

The buildings (quarters, administration buildings, workshops, sheds etc.) and utilities (power plant, desalination plant, distribution system, flood lighting etc.) were included in the tender documents with indications of functional requirements, main dimensions, material quality specifications, and comprehensive bills of quantities.

In this way the detailed design period was shortened and an opportunity was given to the contractor to choose the detailed design or make of equipment which was best suited for execution by his personnel and construction equipment, and hereby he could cut his cost and give a low bid for the job.

For a number of items of equipment, prime cost amounts were included in the bill of quantities. For these items the contractor would have to present a list of suppliers for approval by the client, call bids for the equipment, accept the best offer and purchase the equipment at the price offered. The payment was covered from the prime cost amount. The contractor received a fixed percentage of the cost as his remuneration for the services in this connection.

If all parts of the project had been designed in detail, and all soil investigations, compaction tests etc. had been completed before the issuing of the tender documents, the tender date would have been delayed 6 to 12 months.

The legal basis for the tender documents were the International Conditions of Contract issued by FIDIC and the national tender regulations of Saudi Arabia.

For the floating steel pontoons, the tender documents were prepared according to normal procedure for ship tenders.

5.4 Tendering, Tender Evaluation and Contract Negotiations

Tenders were invited for the main contracts among major international contractors, and for the pontoons among a number of shipyards.

At the dates fixed, a satisfactory number of relevant tenders were received.

A careful check and evaluation of the offers were carried out, and a detailed tender evaluation report was prepared and submitted to the client, with recommendations of the offers to be accepted.

Following the approval of this report, contract negotiations were initiated. For the Gulf Port it was decided to keep the two alternatives within the contract until after the additional soil investigations had been completed (see under 5.3 above).

For the Red Sea Port, it was decided to reduce the length of fixed quay walls and introduce a second pontoon, because the tender prices received for the pontoons were competitive compared with the corresponding costs of civil works.

6. CONSTRUCTION

Based on the contract negotiations, the contracts were completed and officially awarded, the sites were handed over and the construction began.

6.1 Selection of Alternative at Gulf Port

At the Gulf Port, the most critical activity was the completion of the 2.5 or 4 km long causeway and the abutment for the floating pontoon pier, which had to be ready for installation of the pontoon 7 months after the start of construction.

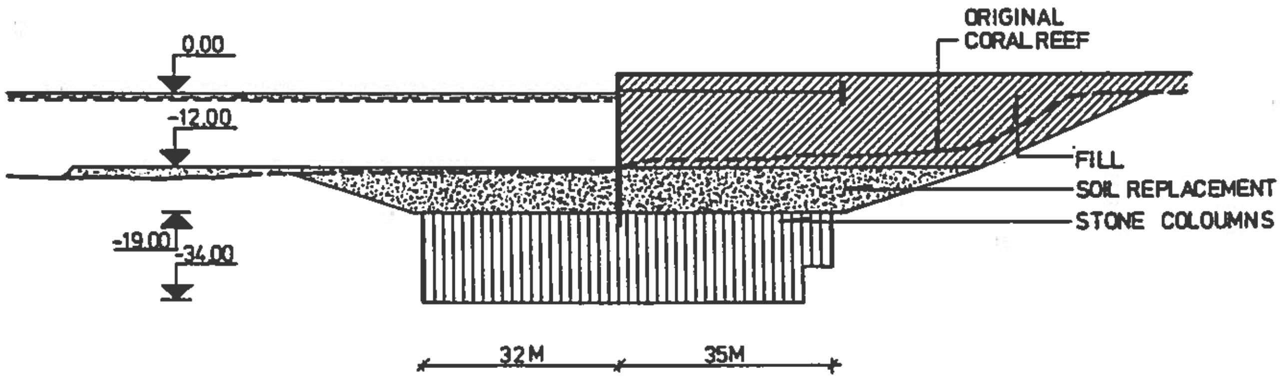
Concurrently the additional soil investigations were carried out. The evaluation of these had the conclusion, that major dredging works were feasible (the thickness of cap rock was less than 1 m in average) and could be carried out at the unit rates quoted in the tender. Consequently the alternative with a shorter causeway (2.5 km length) and a longer dredged channel could be selected, resulting in a saving of about 25 mill U.S. dollars compared with the other alternative.

At the time when this decision was taken, the causeway had already reached a length of more than 2 km out from the shore, almost to the point where the abutment for the pontoon was to be constructed.

6.2 Soil Improvement Solution at Red Sea Port

At the Red Sea Port the supplementary soil investigations and trials indicated that a soil improvement procedure as follows would be the optimal solution for the major part of the quay front : Removal of loose layers down to level - 19 m; placing a layer of screened coarse gravel; vibro compaction by large vibrators down through the gravel blanket, forming "stone columns" down through the loose coral sand to the firm soil below, around level - 34. (See figure below). This solution: a combination of soil replacement and vibro compaction could be executed within the framework of the contract bill of quantities and resulted in savings of the order of 15 mill. U.S. dollars compared with a complete replacement of the loose layers.

For the northern part of the quay wall the firm coral rock was present from level - 20 to - 25 m. In this section conventional removal of loose soil and replacement with selected fill was carried out.



CROSS SECTION
PRINCIPLE OF SOIL IMPROVEMENT

An earthquake engineering study had the conclusion that the improved soil could carry the necessary loads from the structures, with the necessary factor of safety during the "design earthquake".

Settlements of the quay structures were measured regularly for a period of 2 years after the completion of the works, and these measurements revealed settlements of only a few centimetres, well within the predicted range.

6.3 Completion of the Works

After the selection of the mentioned features within the projects, and the check and approval of the design of buildings and utilities and the equipment proposals prepared by the contractor, all details of the projects were determined and the construction was completed according to schedule, under the supervision of the consultant.

7. CONCLUDING REMARKS

At the completion of the projects, detailed commissioning procedures were performed, and the ports were handed over to the client.

Prior to this the consultant had prepared tender documents for the operation and maintenance of the ports, tenders were called, a service contractor selected and the contract awarded.

A detailed handing over procedure from the Client to the service contractors was carried out and for the following years the consultant carried out supervision of the work of the service contractor, at both of the ports.

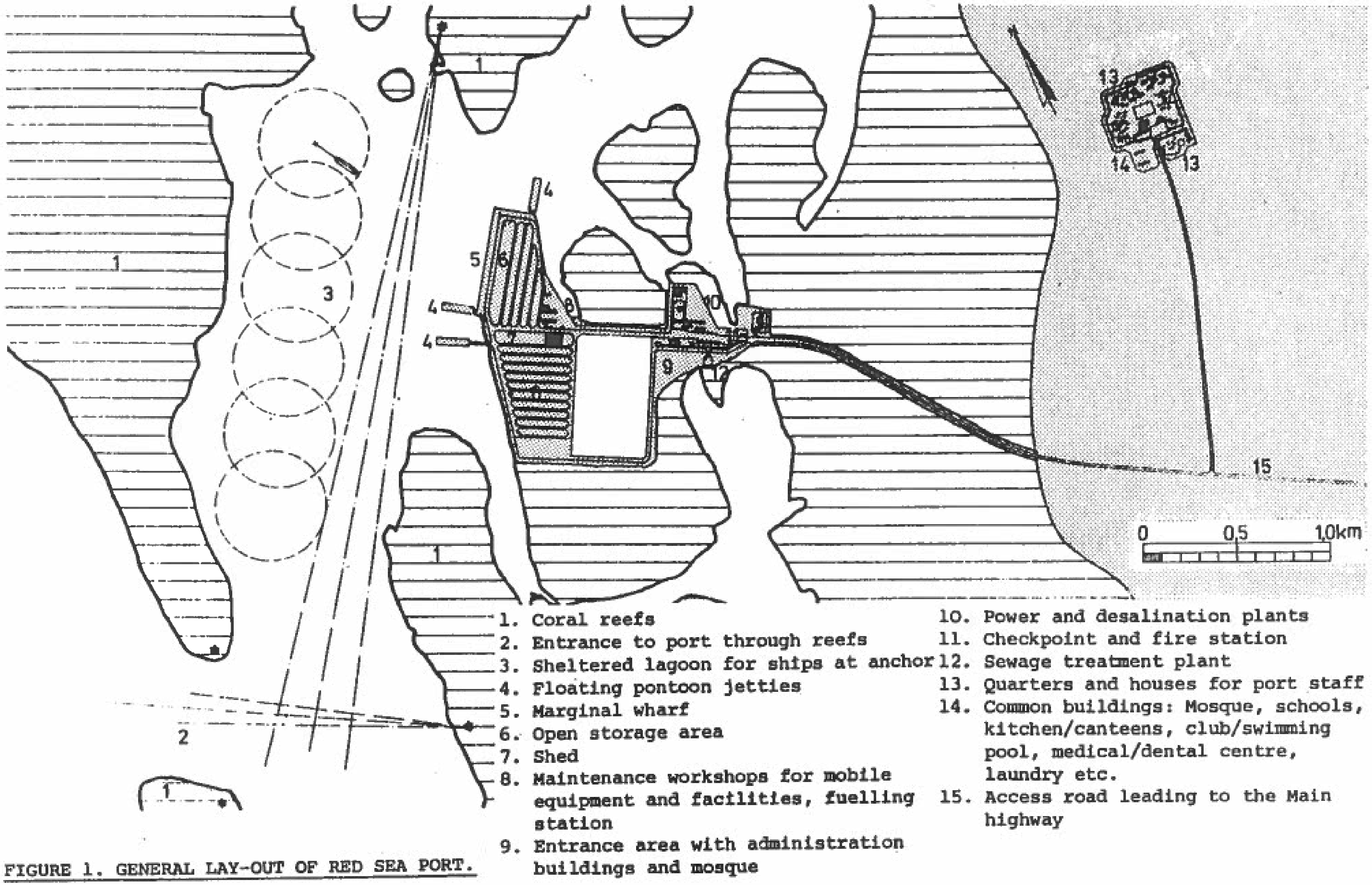


FIGURE 1. GENERAL LAY-OUT OF RED SEA PORT.

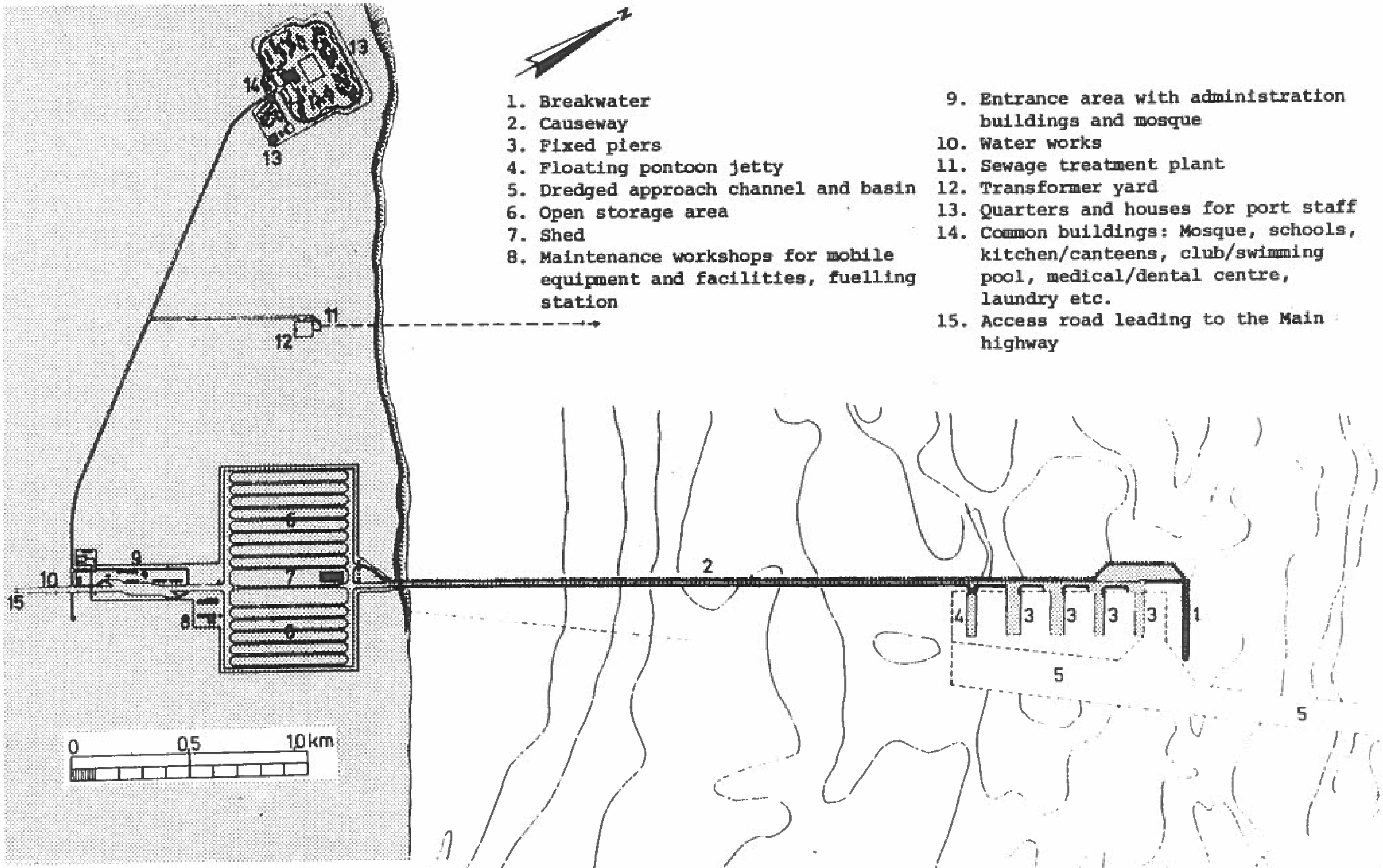


FIGURE 2. GENERAL LAY-OUT OF GULF PORT.

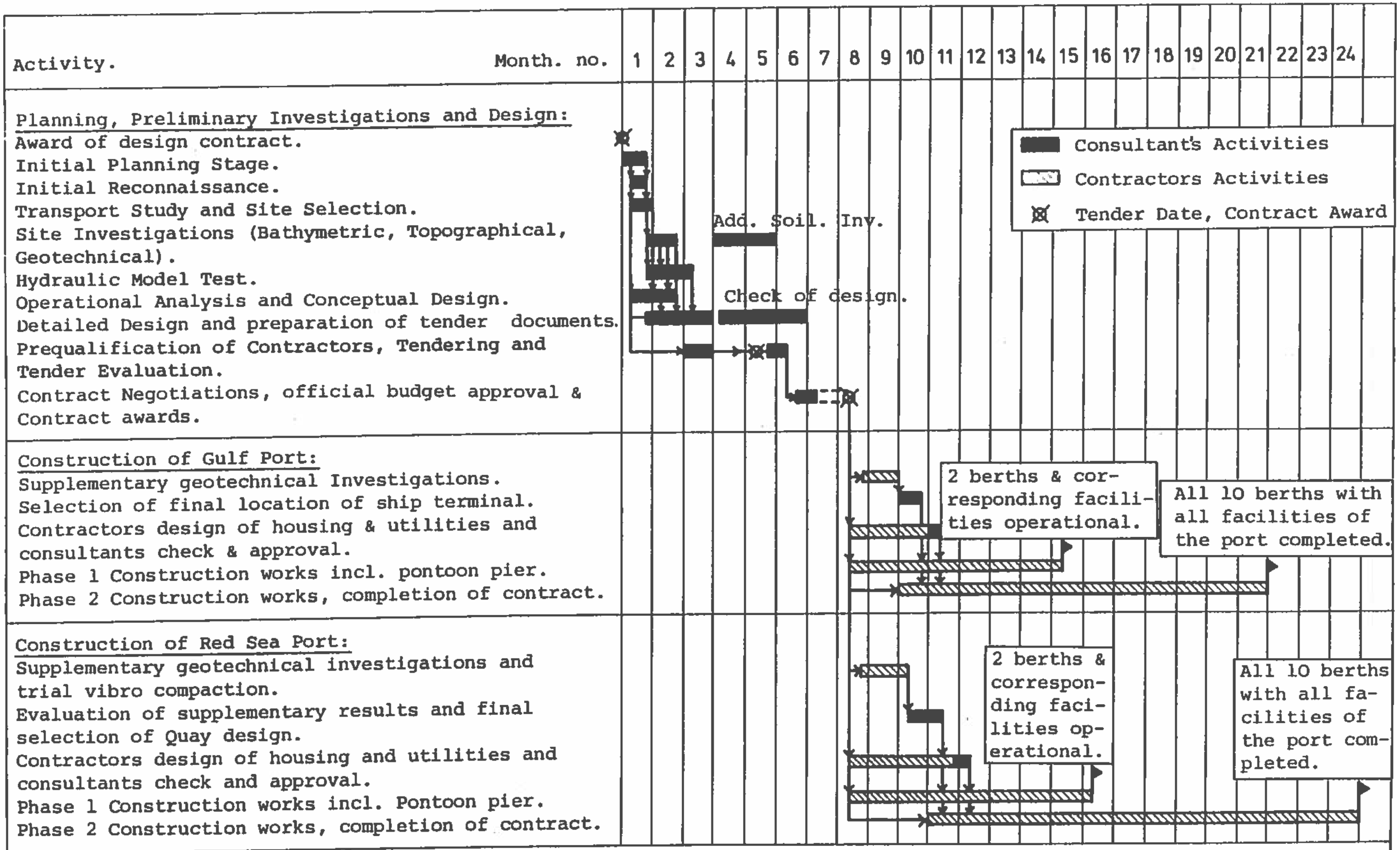


FIGURE 3: GENERAL TIME SCHEDULE FOR PLANNING, DESIGN AND CONSTRUCTION OF THE TWO SAUDI PORTS.

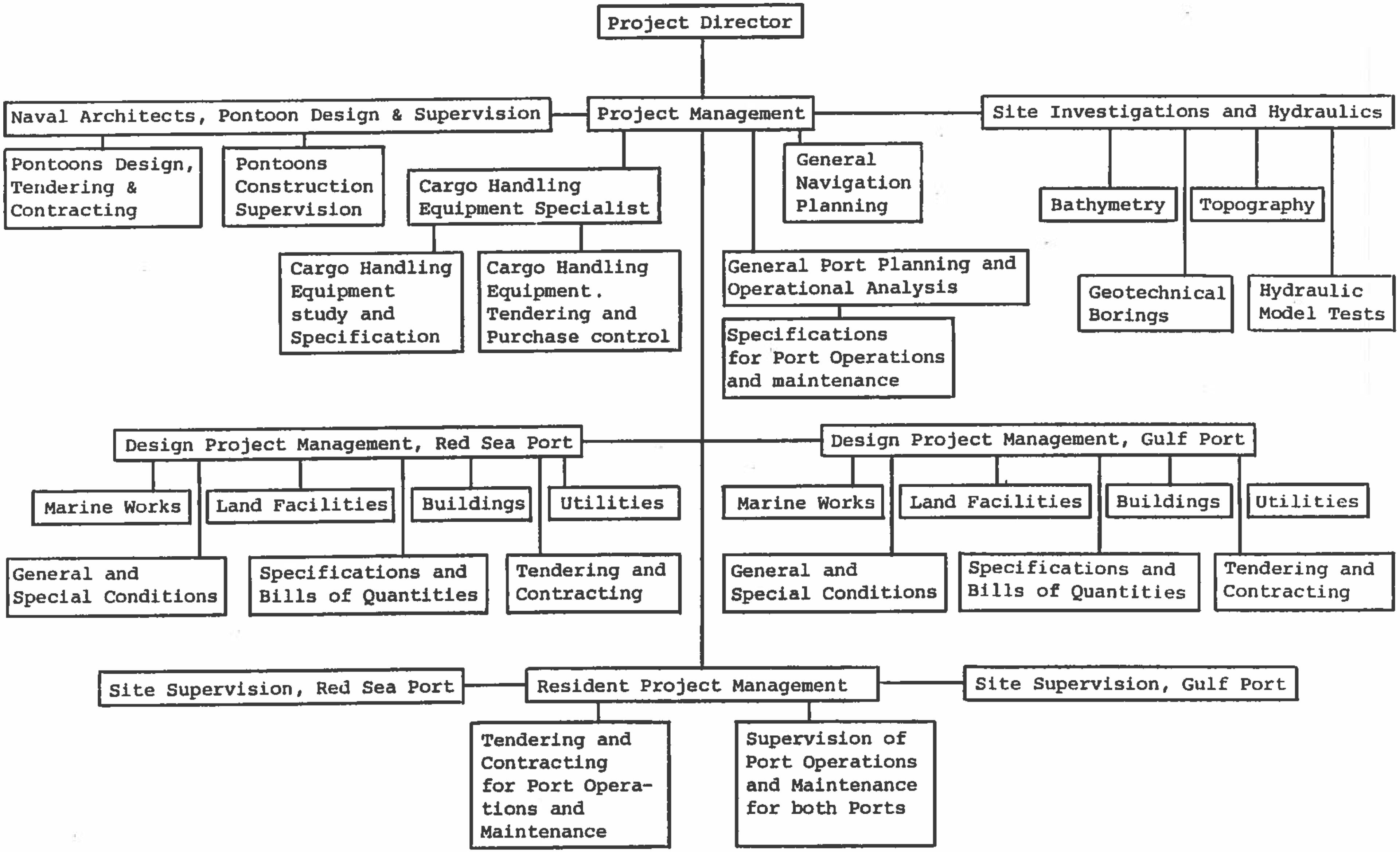
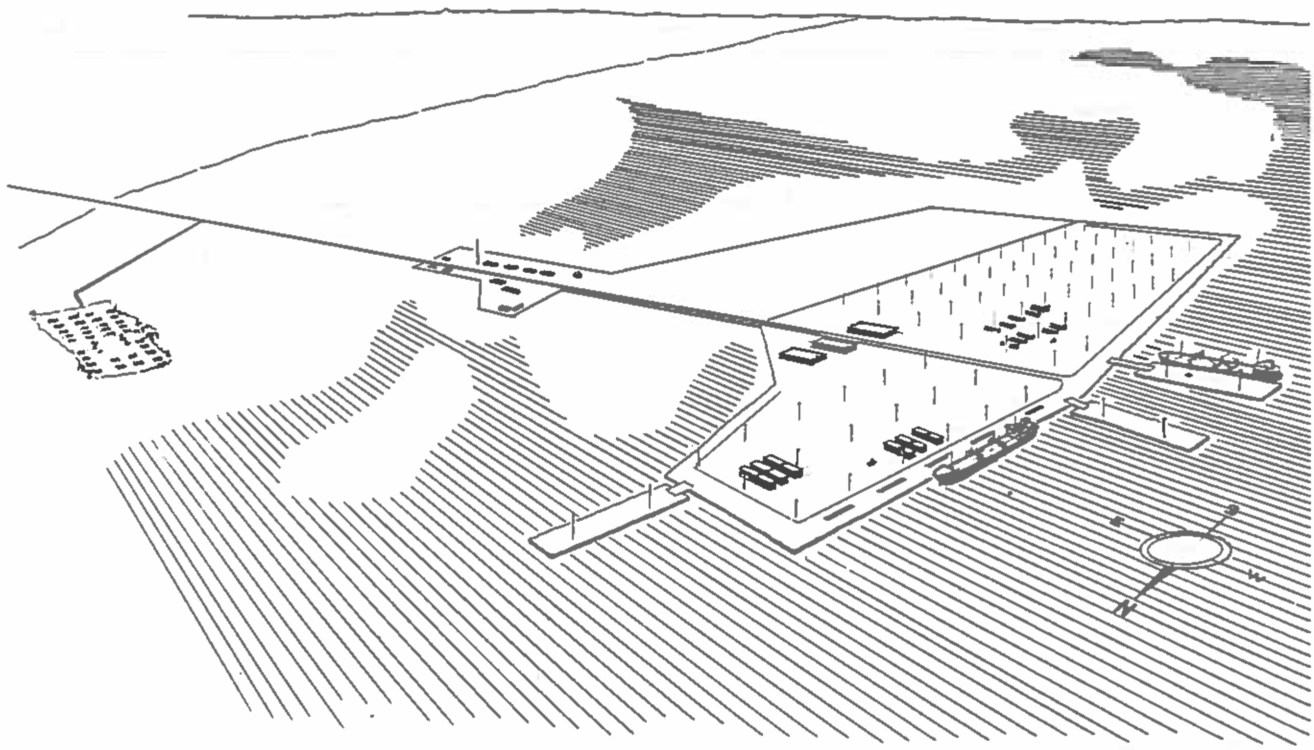


FIGURE 4: GENERAL PROJECT ORGANIZATION DIAGRAMME.

dangroup Consulting Engineers and Planners A/S



Port of Quadeema, Saudi Arabia

The Port is located at Quadeema on the Red Sea Coast some 90 kms north of Jeddah and placed in a well protected coral lagoon with water depths ranging from 11 to 20 m.
The Port has some 800 m marginal wharf, 3 pontoon jetties providing a total of 6 berths, each having a length of 180 m and minimum water depth of 11–12 m.
The Port of Quadeema is being constructed for the purpose of providing import facilities for a very extensive house building programme adopted by the Ministry of Public Works and Housing.
The completed Port shall be able to handle an annual throughput of 1.5 million tons of building equipment, material and components, and is further planned for incorporation of facilities for handling of some 0.5 million tons of cement in bulk.
The Ministry has entrusted the entire planning, design and supervision for the project to the Danish Hydraulic Institute, which has formed a Consulting Group together with Dangroup, Consulting Engineers and Planners A/S, and Knud E. Hansen, Naval Architects. The work of the group is governed by a very tight time schedule indicating a period of less than two years from the start of planning and site investigations to the completion of the Port.

The ship terminal, the open storage area of approximately 300,000 m^2 and a shed of 4,000 m^2 are served by cargo handling equipment including 3 tons, 15 tons and 40 tons fork lift trucks, and a fleet of multipurpose 40 tons capacity flat bed terminal tractors and trailers.
The project includes administration buildings, workshops and utility buildings for the operation of the Port. A permanent camp provides accomodation facilities for an estimated operation staff of approximately 1,500 persons.
Included in the project is furthermore provision of all necessary utilities, i.e. communication facilities, water supply, sewage treatment, electricity supply. A 5 km access road connects the Port with the main road Jeddah-Medinah.

Project under Construction
Start of construction March 1977.

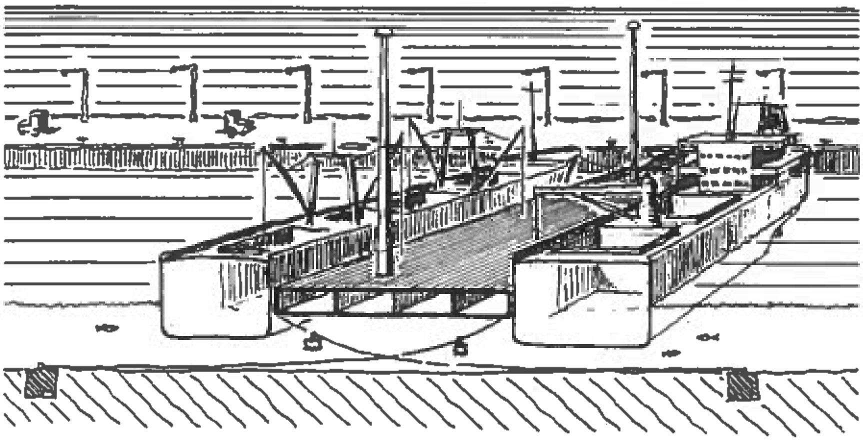
Client:
Ministry of Public Works and Housing,
Kingdom of Saudi Arabia.

Consultants
Danish Hydraulic Institute in collaboration with Dangroup and Knud E. Hansen.

Main Contractor:
Dong Ah Construction Co., Ltd., Seoul, Korea.

Cost of Works:
Approx. 150 million U.S. dollars.

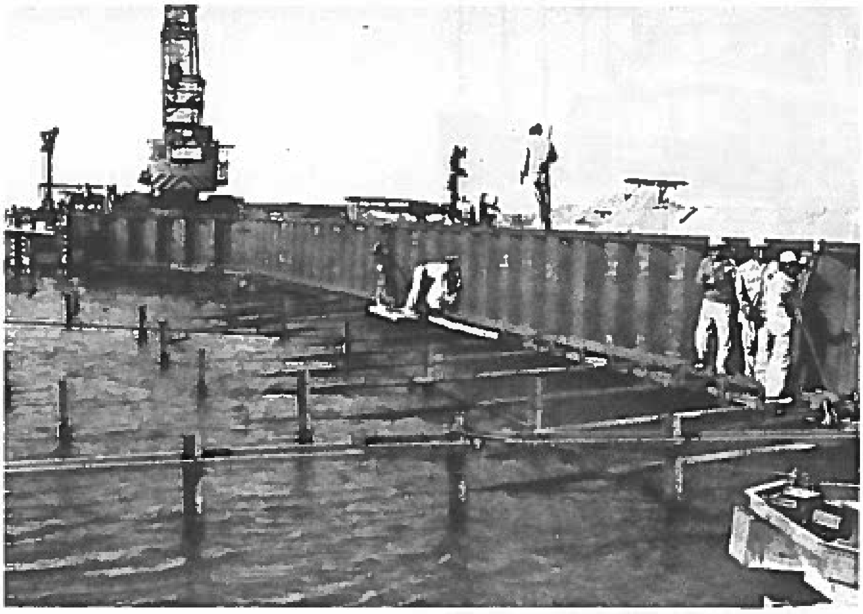
Consulting Services Rendered:
Planning, design and construction supervision for the entire project.
Preparation of specifications, contract conditions and evaluation of tenders.
Specifications for port operation contract.



Steel Pontoon Jetties

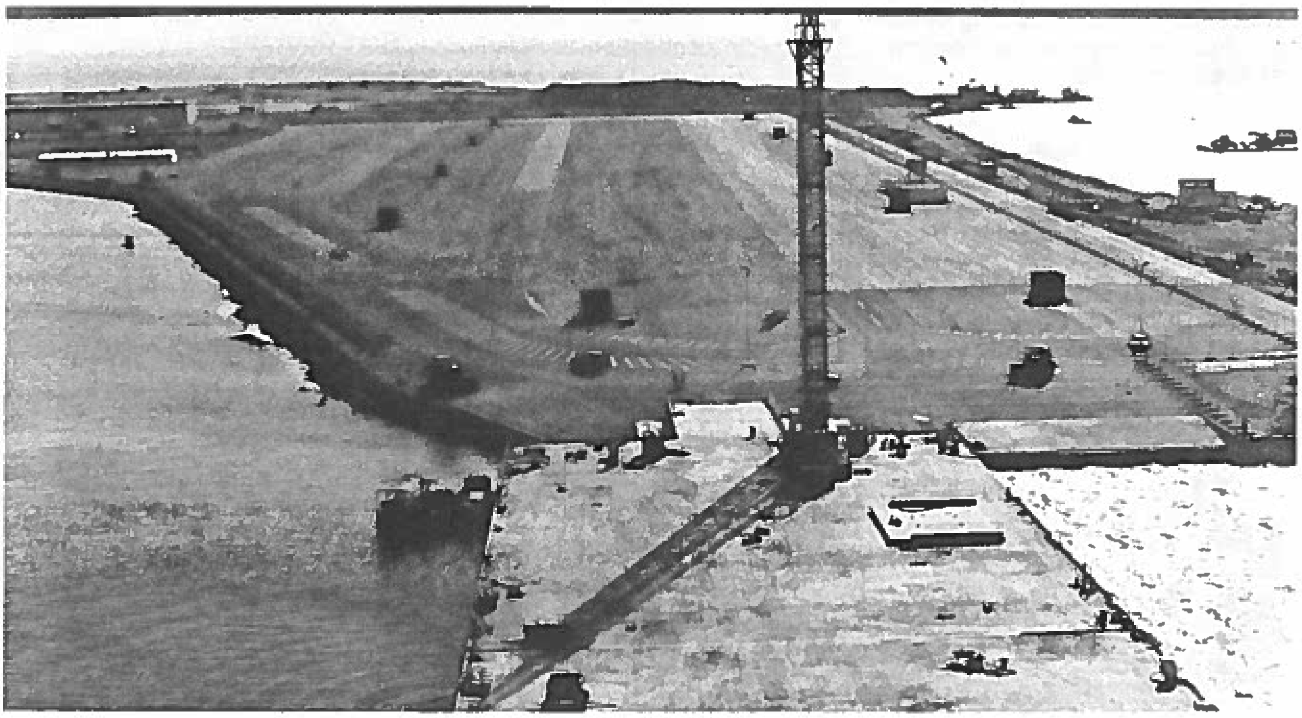
In order to make two berths available very early in the project, a 40×180 m steel pontoon was designed and ordered from Hitachi Shipbuilding and Engineering Co. Ltd. Japan, simultaneously with the award of the main contract. The idea was to gain time by utilizing existing shipyard facilities and thus avoid the long delivery and installation time for sheet piles for a fixed pier.

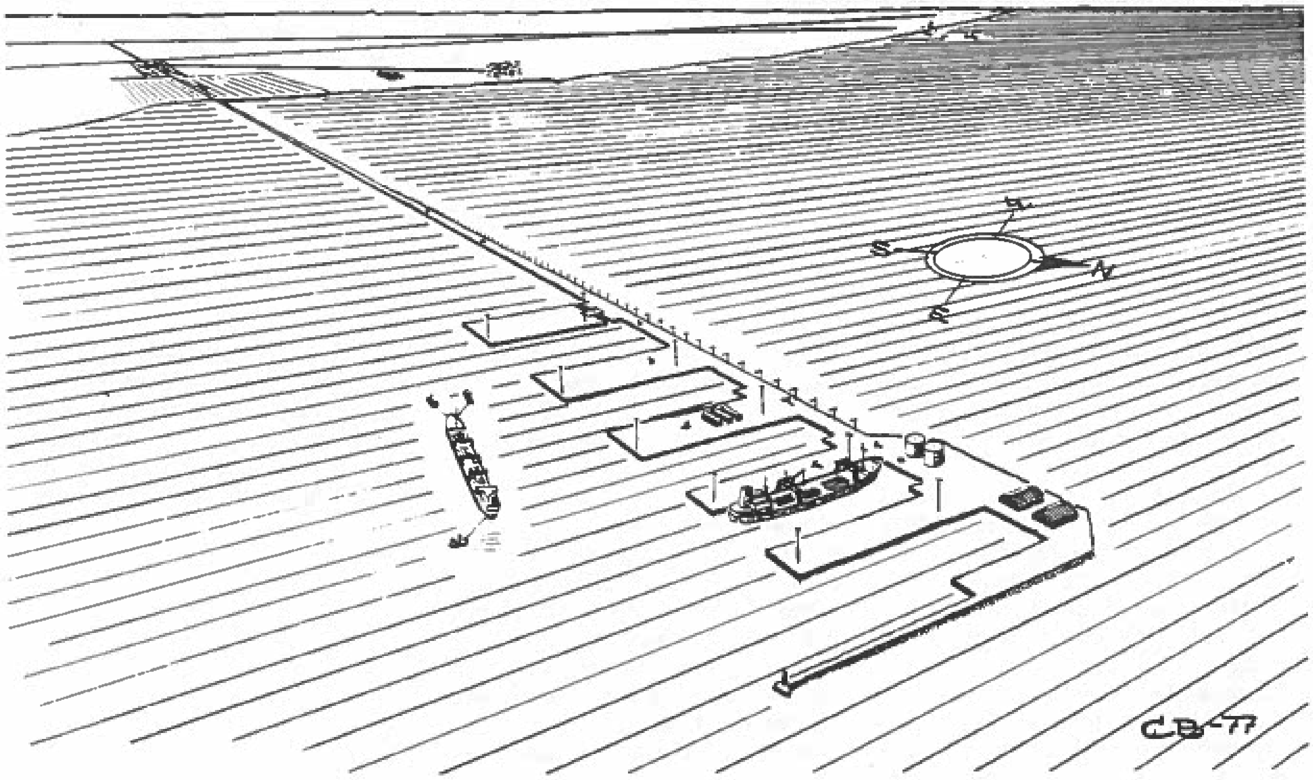
Due to the very expensive soil exchange requirements for any fixed wharf structure it was decided to limit the stretch of marginal wharf to 800 m and to provide the required number of berths by introducing two additional steel pontoon jetties.



The northern part of the 800 m quay under construction.

View over the Port area.
In the foreground part of the first steel ponton Jetty.





Port of Ras al Ghar, Saudi Arabia

The port of Ras al Ghar is being constructed for the purpose of providing import facilities for a very extensive house building programme adopted by the Ministry of Public Works and Housing.

The completed port shall be able to handle an annual throughput of 1.5 million tons of equipment, material and building components and is further planned for incorporation of facilities for handling of some 0.5 tons of cement in bulk.

The Ministry has entrusted the entire planning, design and supervision for the project to the Danish Hydraulic Institute which has formed a Consulting Group together with Dangroup, Consulting Engineers and Planners A/S, and Knud E. Hansen, Naval Architects. The work of the group has been governed by a very tight time schedule indicating a period of less than two years from the start of planning and site investigations to the completion of the Port.

The Port is located at Ras al Ghar on the Gulf Coast some 60 kms north of Dammam. The coast is characterized by very large distances to deep water, and consequently an approx. 3 kms causeway has been constructed for connection from ship terminal to shore.

The ship terminal consists of four fixed piers and one pontoon jetty providing a total of 10 berths, each having a length of 180 m and minimum water depth of 11–12 m. The berths are sheltered by a breakwater against the predominant waves from northern directions.

The ship terminal, the open storage area of approx. 300,000 m^2 and a shed of 4,000 m^2 are served by cargo handling equipment including 3 tons, 15 tons and 40 tons fork lift trucks, and a fleet of multipurpose 40 tons capacity flat bed terminal tractors and trailers.

A permanent camp provides together with administration buildings, workshops and utility buildings, accomodation and working facilities for Saudi officials and an estimated operation staff of approx. 1,500 persons.

The project includes provision of all necessary utilities i.e. communication facilities, water supply, sewage treatment, electricity etc.

A 5 kms access road connects the Port with the main road Dammam – al Jubail.

Project under Construction
Start of construction March 1977.

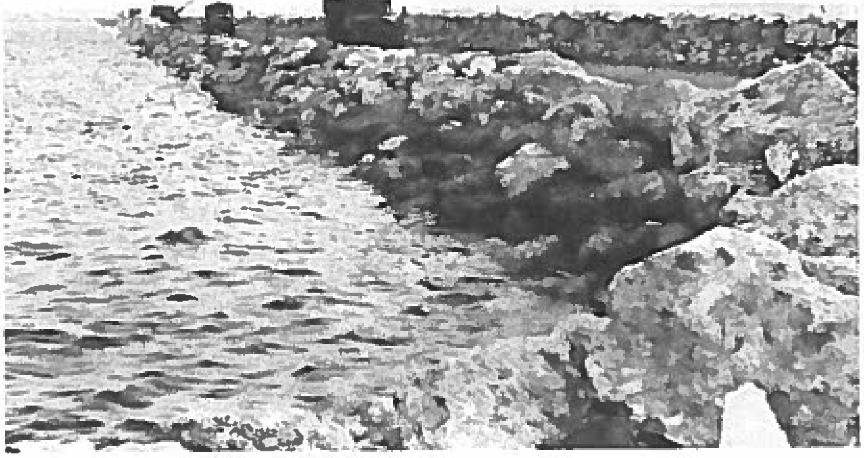
Client:
Ministry of Public Works and Housing,
Kingdom of Saudi Arabia.

Consultants
Danish Hydraulic Institute in collaboration with Dangroup and Knud E. Hansen.

Main Contractor:
Hyundai Construction Co., Ltd, Seoul, Korea.

Cost of Works:
Approx. 250 million U.S. dollars.

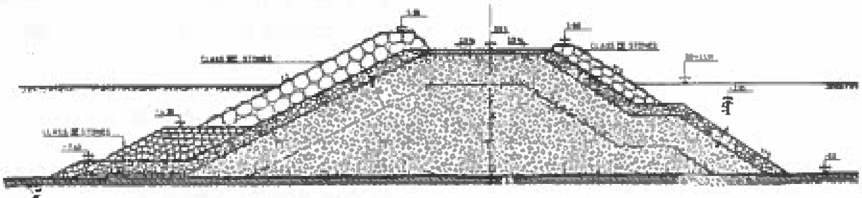
Consulting Services Rendered:
Planning, design and construction supervision for the entire project.
Preparation of specifications, contract conditions and evaluation of tenders.
Specifications for port operation contract.



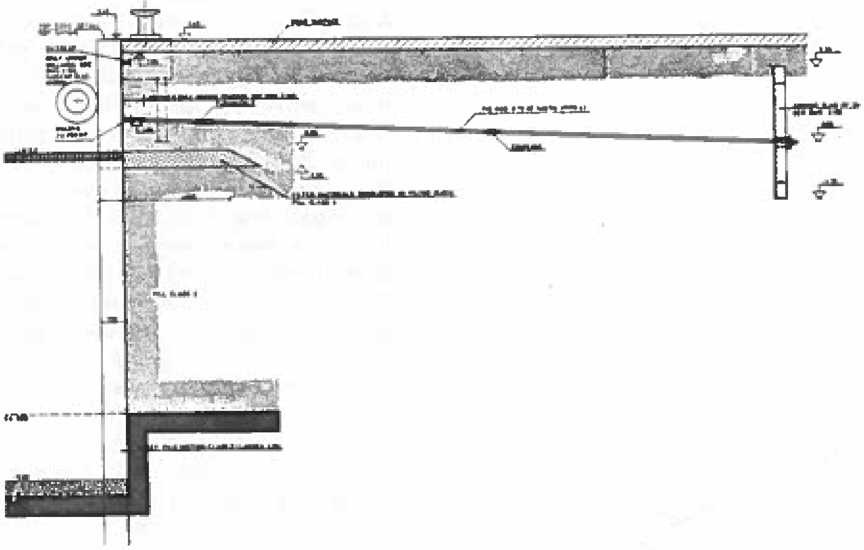
Causeway

The most critical activity in the project was the construction of the 3 km causeway in time for the operation of two berths only 7 months after start of construction.
By a very impressing effort the main contractor Hyundai Construction Co. managed to push the core, the inner stone protection layers and the 12 m road of the causeway all the way to the ship terminal within less than 5 months, leaving ample time for completion of the armour stone layer before the given target date.

The 3 kms causeway was available for traffic after 5 months.



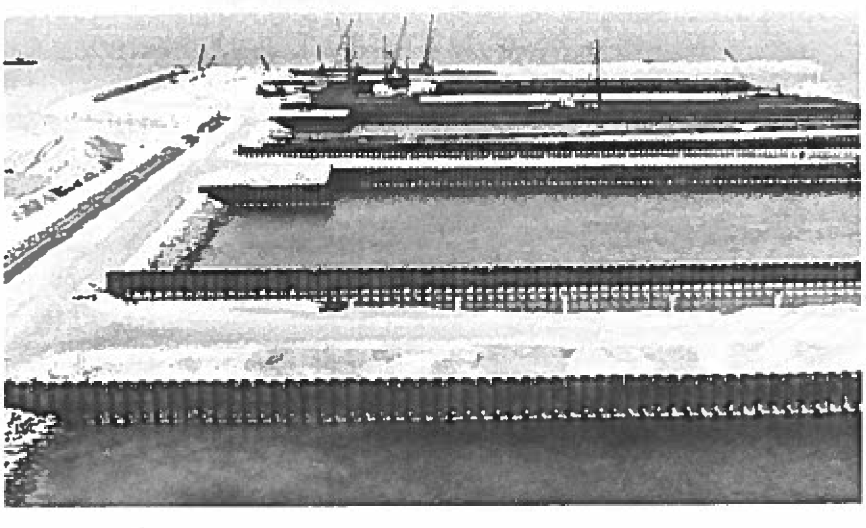
Cross-section of causeway.



Cross-section of a pier, 10 m water depth.

Nominated Subcontractors for Special Equipment

Special items such as communication facilities and cargo handling equipment have been included in the main contract as PC-sums and tendered during the course of the project eventually becoming goods supplied and installed by nominated subcontractors.
These items include: 24 nos 3 t fork lift trucks, 70 nos 15 t fork lift trucks, 2 nos 40 t fork lift trucks, 145 nos 40 t flat bed trailers and 32 nos terminal tractors.
Microwave link connection Ras al Ghar – Dammam.
SSB radio connection Ras al Ghar – Riyadh.



Ship terminal. The four fixed piers under construction, november 1977.

GEOTEKNISKE PROBLEMER VED ET HAVNEBYGGERI I SAUDI ARABIEN

Referat ved civ.ing. C. Bæk-Madsen af foredrag i Dansk Geoteknisk Forening af civ.ing. Torben Ernst (HSS) og civ.ing. K. Bennick (COWI).

MØDEREFERAT

Geotekniske problemer ved et havnebyggeri i Saudi Arabien

var emnet for sæsonens 2. møde i DGF torsdag den 14.december 1978.

Efter Krebs Ovesens indledning og præsentation af aftenens to foredragsholdere: Civilingeniør Torben Ernst fra J. Hostrup Schultz og O. Sørensen samt civilingeniør K. Bennick fra Cowiconsult,blev foredraget påbegyndt, idet dette blev fremført som en dialog mellem de to foredragsholdere, der gav en redegørelse for hændelsesforløbet for havnens bygning, hvor geoteknikken kom til at spille en afgørende rolle. Foredraget blev støttet af et omfattende materiale af overheads og lysbilleder.

Indledningsvis omtalte Torben Ernst, at fremstillingsformen knyttet til det faktiske hændelsesforløb med geoteknikkens tilknytning til kontraktforhold, tid m.v. måske fik sagen til at virke uoverskuelig, men mere realistisk.

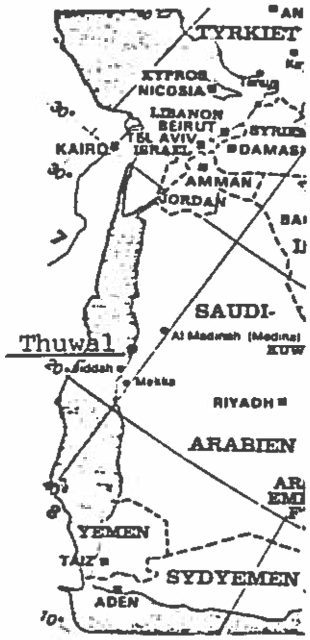
Af hensyn til referatets omfang har undertegnede tilladt sig at gå lidt på tværs af dette og ikke holde sig strengt til rækkefølgen i fremstillingen.

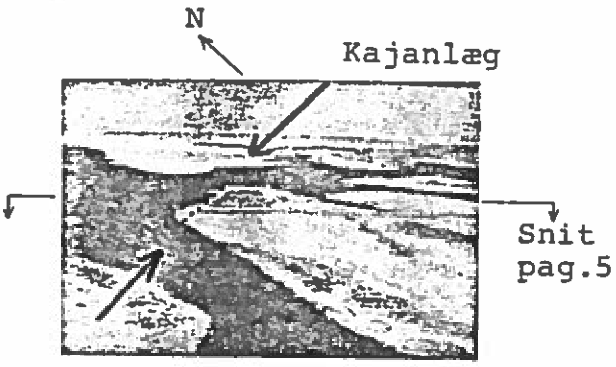
Den aktuelle havn, Quadeema, er beliggende ved Thuwal på Røde Havs kysten ca. 90 km nord for Jeddah med Ministry of Public Works and Housing som bygherre. I juni 1976 indledtes forhandlinger med Dansk Hydraulisk Institut om projektering, som sammen med Dangroup og Knud E. Hansen og bistand fra professor Lundgren og Geoteknisk Institut m.v. påtog sig opgaven.

Havnens beliggenhed blev udvalgt mellem 3 muligheder efter orienterende opmålinger.

Beliggenhed og omtrentlig tidsplan fremgår af nedenstående:

Tidsplan	1976	1977	1978
DHI kontrakt			
Valg af sted			
Boringer			
" ,suppl.			
Kontrakt entrepr.			
Anlægsarbejder			
Suppl. boringer			
Stensøjler			





Havnedata:

Havnen omfatter oplagsplads, værksteder, beboelse for 1400 mand, adgangsveje samt 725 m kaj langs revforkant, suppleret med en 40 x 180 m flydende stålponton mod nord og en fast pier mod syd. Vanddybde 12 m mod syd og 10 m mod nord.

Kajen skulle udformes som en stålspunsvæg placeret langs et koralrevs forkant. Der findes det aktuelle sted større sammenhængende koralsystemer og ved Quadeema findes et system af parallelle rev, hvor de første bundundersøgelser viste en ikke massiv klippe, med øverst en løs masse og løse sandforekomster under de øvre koraller. Efter dannelse af korallerne sker der gennem dyrelivet en tilfyldning mellem disse og endelig sker der en cementeringsproces i varierende omfang.

De første undersøgelser omfattede 25 spulesonderinger med en 3 HK pumpe gennem 3/4" jernrør og 15 bundprøver optaget med grab samt ca. 18 km seismisk bundprofilering. Herudover udførte Raymond International de første 10 boringer og 46 sonderinger ned til 0,4 á 4 m dybde med Atlas Copco mejsel. Laboratorieforsøg blev udført i Athen og på DGI. Undersøgelserne viste meget løst lejret koralsand med SPT-værdier ~ 0 med underliggende lag af blødt silt og ler med SPT ~ 0→40. De seismiske undersøgelser viste ligeledes løse, bløde aflejringer med lydhastigheder lig lydhastigheden i vand ned til kote ÷ 15 á ÷ 26.

Havbunden mellem revene bestod af marine sedimenter af karbonater og silikater. De fundne jordarter kunne groft opdeles i to lagserier:

Øverst: 1,5-6 m løst kalksand med skønnet $\gamma_d \sim 9$ kN/m^3 og $\varphi \sim 16^o$ med mulighed for liquefaction (porevandstrykket kan ved rystelser i løst lejrede, finkornede sedimenter hurtigt vokse til en størrelse, der medfører, at de effektive spændinger bliver nul og jordmassen dermed flydende).

Nederst: Lagserie fra kote ÷ 16 til ÷ 22 af sand, silt og ler med skønnede SPT-værdier på 0→20 og $\varphi \sim 16\text{-}30^o$ i de øverste 3 m og herunder $\varphi \sim 30\text{-}36^o$, samt koralrev med flad klippe foroven og underliggende løstlejret kalksand med $\varphi \sim 16\text{-}32^o$.

Seismisk aktivitet for nærliggende jordskælv måtte forudses og rapport over de første undersøgelser konkluderer supplerende vingeforsøg med norsk vinge, nøjere fastlæggelse af lerlag med dybere boringer, komprimering af det løse sand og yderligere oplysninger om seismiciteten.

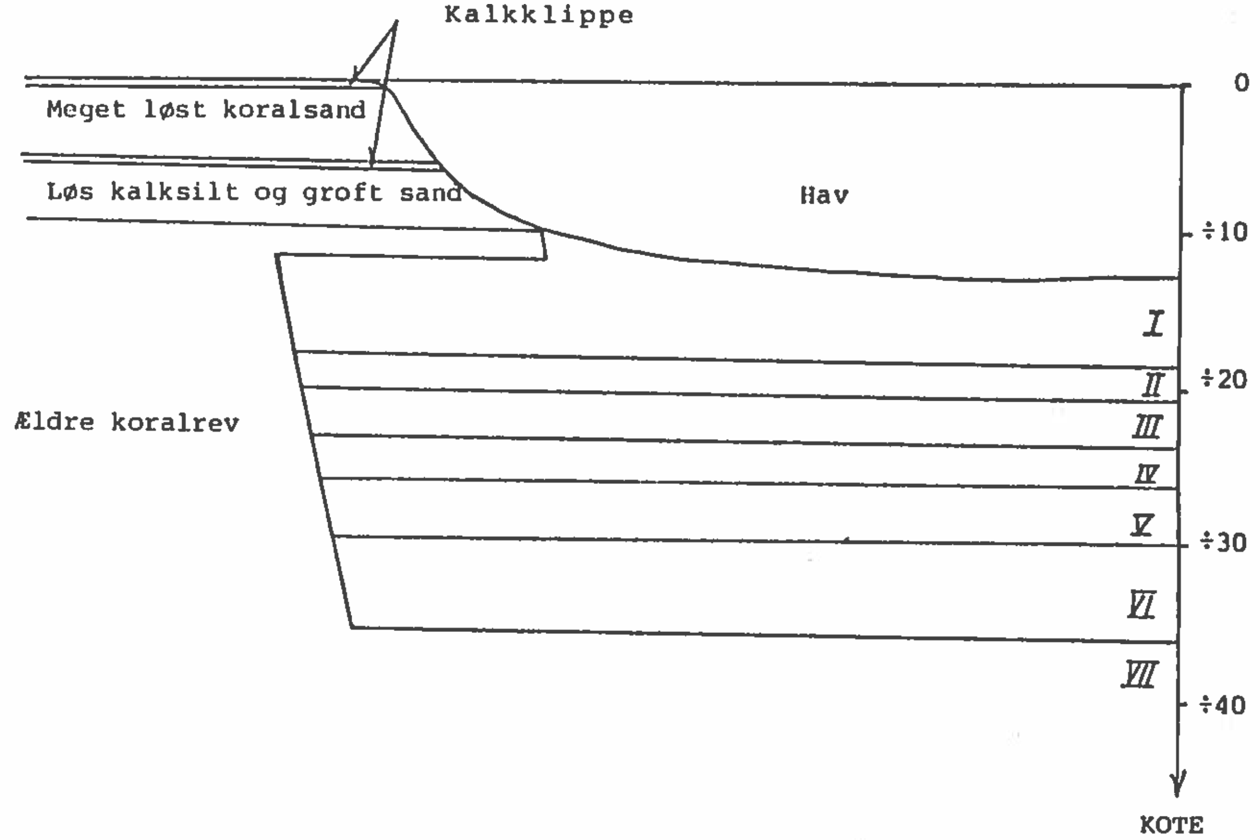
Rapporten indeholdt også vurderinger af kajernes stabilitet ved undersøgelse af brudcirkler og glidning; disse vurderinger viste, at sandet måtte komprimeres i en bredde af ca. 100 m foran væggen, eventuelt udskiftes. Administrative og budgetmæssige forhold bevirkede imidlertid, at kun et stærkt begrænset program af supplerende boringer kunne gennemføres til fastlæggelse af jordforbedring og/eller udskiftning; hvorfor licitationsmaterialet blev ændret, så vibro kompaktering og jordbundsundersøgelser kom til at indgå. Vibro-forsøgene skulle vise, om vibrering alene eller med tilsættelse af stenmaterialer kunne gennemføres. Dette specialarbejde skulle udføres af godkendte firmaer, hvoraf på forhånd følgende var godkendt:

Cementation, London; GKN Keller, Frankfurt og Held und Franke, München.

Entreprenør blev det koreanske firma Dong Ah.

De supplerende forundersøgelser ved årsskiftet 1976-77 blev igen udført af Raymond International. De norske vingeforsøg var meget vanskelige at udføre, og der blev i alt kun udført 6 vingeboringer ved 4 skylleboringer og 4 geotekniske boringer til ca. 24 m u.t. Laboratorieforsøg blev udført i felten.

Rapporten over disse undersøgelser viste i store træk en opdeling af bunden i hovedformationer, som kunne underopdeles i zoner af sammenlignelige styrker og kalkindhold. Hovedopdelingen, der fremgår af det angivne tværsnit (pag. 5), blev kun revideret i mindre omfang ved senere undersøgelser.



I Meget løs kalksand og -silt.

II Meget løs til middelfast svagt cementeret let kalkholdig sand og siltet ler med organiske bestanddele,

III Overvejende meget løs til fast let cementeret finsand med organiske bestanddele.

IV Overvejende meget bløde til stive let cementerede lerer.

V Overvejende meget bløde til meget stive let cementerede let kalkholdige lerer med organiske bestanddele.

VI Overvejende bløde til meget stive lettere cementerede kalkholdige lerer.

VII Meget svag koralkalk og hård kalkslam.

Sammenligninger mellem vingestyrke, SPT-styrke, kalkindhold og vandindhold i forhold til jordart, dybder m.v. gav indicier for at opstille styrkeparametre med angivelse af nedre værdier og gennemsnitsværdier. Konkluderende er der tale om meget svage jorder ned til zone V̲ i kote ca. ÷ 26.

Projektkontrol viste med de seneste resultater større udskiftninger end tidligere beregnet og med igangværende kontrakforhandlinger og fastlåst kontraktbeløb blev den faste pier erstattet med en flydende ponton, hvilket gav den nødvendige besparelse.

Dong Ah startede ultimo marts 1977 med de nødvendige tilkørselsveje og supplerende bundundersøgelser omfattende 13 boringer til søs og 9 på revene; ialt 490 boremetre samt vibroforsøg i 185 punkter med dynamiske sonderinger før og efter for kontrol. De samlede undersøgelser viste herefter, at bundforhold i rev og havbund bestod af en serie slappe aflejringer af karbonater og silikater helt ned til kote ÷ 33 m overlejrende ældre koralkalk, hvor zone IV, V og VI hver indledes med en udpræget svaghedsevne. Den øverste koralplades alder er ca. 2000 år, mens det ældre underliggende koralrev er mere end 40.000 år og sandsynligvis ca. 70.000 år. Forkastninger, vekslende vandspejl (ned til kote ÷ 80), temperaturer og vækstbetingelser for korallerne har udformet lagene, som de træffes i dag med meget lave friktionsvinkler i de øvre lag ($\varphi \sim 15$ á 16^{o}).

Forsøgene med vibrokomprimering blev udført af Dong Ah med et joint venture af Cementation og Keller. Metoden omfatter nedspuling af en cylinderformet vibrator, hvor komprimeringsfasen begynder fra den ønskede dybde med vibration opefter i små trin uden spulning og eventuelt samtidig tilførsel af sten fra et udlagt stentæppe på havbunden. Forsøgene gav kun ringe forbedring af jorden mellem stensøjlerne ved komprimeringen, men stensøjlerne betyder i sig selv en definitiv forbedring af jorden.

Projektet revideredes derfor til en mindre udskiftning af bunden med stenmaterialer til kote ÷ 17 á ÷ 19 m og konstruktion af stensøjler til kote ÷ 32 á ÷ 34 m gennem et 5 m tykt stentæppe. Stensøjlerne skulle fungere som "armering" af de svage jordlag, hvorved stensøjlernes relativt større friktionsvinkel og dermed større modstand mod overklipning gav det nødvendige ekstra bidrag til stabiliteten.

Supplerende vurderinger og stabilitetsberegninger gav et krav om en minimumdiameter af søjlerne på 1 m og en indre friktionsvinkelt på $\varphi \sim 38^{o}$ i stensøjlerne.

Sideløbende foretog professor Lundgren undersøgelser af liquefaction som følge af jordskælv og disse resultater blev inddraget i stabilitetsvurderingerne, hvor der er regnet med zonebrud på aktiv og passiv-siden og med glidning af det mellemliggende legeme med det aktive jordtryk udregnet proportionsmæssigt i forhold til stensøjlernes volumen. Disse beregninger viste netop sikkerhed mod brud.

De nøje udarbejdede detailspecifikationer for udførelse af stensøjlerne indeholdt også detaljerede krav til tilsynet med udførelse af søjlerne. Dong Ah's anmodning om at anvende et joint venture mellem Foundation Technique og Soil Mechanics blev accepteret og en 60 x 20 m forankret ponton med 5 vibratorer blev anvendt til forstærkninger af et ca. 600 x 70 m område. Udførelsesproblemer med utilstrækkelig nedtrængningsdybde og -søjlediameter gav væsentlige kontroverser med entreprenøren, der mente, at fejlvurdering af vibroforsøg og afvigende bundforhold var årsagen, mens DHI/Dangroup mente, der var tale om for kort konstruktionstid og for høje løft under komprimeringen.

En fornyet gennemgang af beregninger fra begge parter gav ikke anledning til overensstemmelse. Med visse ændringer i forudsætningerne viste de fornyede beregninger dog, at søjlernes diameter kunne reduceres lidt. Slutresultatet blev en øgning på 70% af det totale antal søjlemetre på grund af for lille søjlediamter. Totalt blev udført 170.000 lbm stensøjler til 63 mill. kr. mod projekteret 100.000 lbm til 56 mill. kr.

Ref. C. Bæk-Madsen
1978-12-18

SEISMICITY IN A RED COASTAL AREA

by

professor Helge Lundgreen — Institute of Hydrodynamics and Hydraulic Engineering, Technical University of Denmark

professor Erling Bondesen — Section for Geology, Roskilde University Centre

H. LUNDGREN, Professor of Marine Civil Engineering, Techn. Univ. Denmark, Lyngby, Denmark
ERLING BONDESEN, Professor of Geology, Roskilde University Centre, Roskilde, Denmark

Seismicity in a Red Sea Coastal Area
Sismicité dans un Terrain Côtier de la Mer Rouge

SYNOPSIS In connection with the design by DANGROUP of Port Quadeema on the Saudi Arabian Red Sea coast north of Jeddah (Fig. 3), the authors undertook the earthquake engineering studies required by the presence of extremely soft layers surrounding the quay walls. - A main tectonic feature in the Red Sea area is the spreading of ocean floor associated with the continuous opening of the rift along the axis of the Sea. The positions of transform faults was compiled from existing geophysical investigations, and the distribution of main epicentres correlated with the kinematics of the African and Arabian plates. The frequency of earthquakes decreases from south to north. - The attenuation of acceleration from a potential future epicentre near the rift to the port site was determined by means of Californian curves. Finally, the risk of liquefaction was based on the SPT-values, N. - The risk of weak local tectonic events, which could not have been recorded by seismographic stations 2000 km away, was also considered in relation to the present concepts of the tectonic development of the Red Sea region and the local geology. - It was concluded that the soft soils should be vibro-compacted to N > 7 at elev. - 15 m before quay wall construction.

1. GENERAL GEOLOGICAL BACKGROUND

Fig. 1 shows the present concepts of the movements of the plates that have relation to the Red Sea as a major geotectonic feature. For the last decades the Red Sea has been subject to numerous analyses as to its initiation and development. Various models have been proposed to explain the observed geological and geophysical features, mainly in terms of plate tectonics and the initiation of sea floor spreading (Whiteman 1968, Coleman 1974, Le Pichon and Francheteau 1978, Cochran 1983).

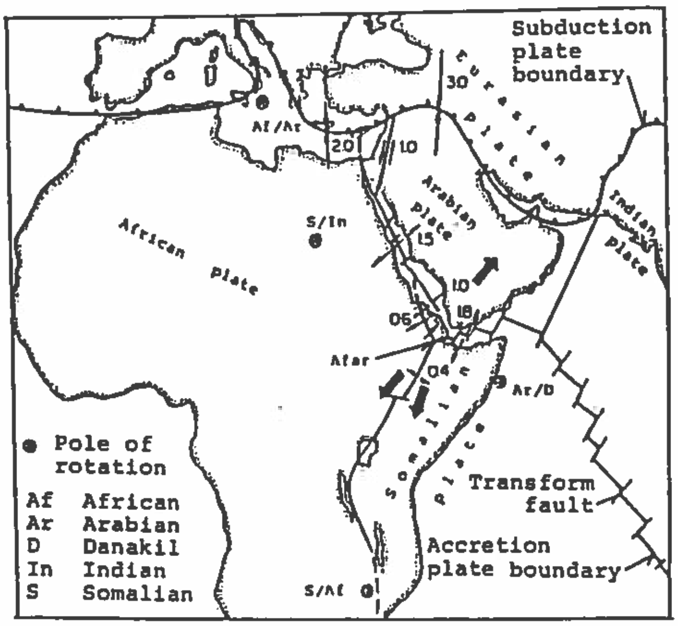


Fig. 1
Movements (thick arrows) of African, Arabian and Somalian plates relative to the Afar triple junction. The figures indicate relative plate movements in cm/a

The development started possibly in the early Tertiary (Eocene). Not until the Miocene (about 20 M.y.), however, the present elongate depression was formed, in which large amounts of evaporites (up to 4 km thick) were precipitated, cf. Fig. 2. Marginally to the basin coarse clastic sediments were deposited and, at the same time, volcanism took place in the bordering shield areas (Coleman et al 1975). In the Pliocene (about 5 M.y.) the Red Sea became connected to the Indian Ocean, as demonstrated by deposition of marine oozes and marginal clastics. The Pleistocene and recent development is featured by the formation of reef building in the coastal areas and deep sea sedimentation and volcanism in the axial zone.

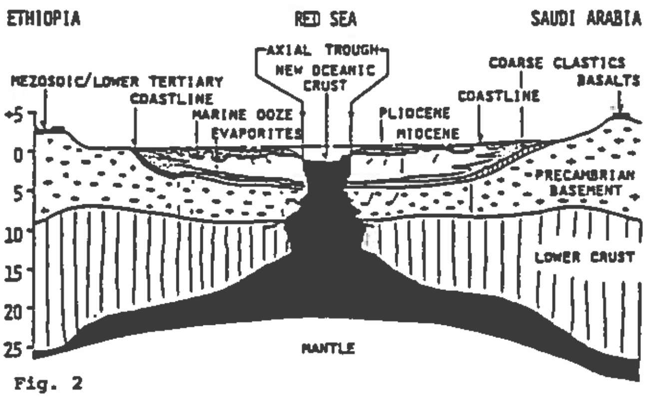


Fig. 2
Cross section of southern Red Sea with main geological units (modified after Coleman et al)

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The structural process has traditionally been interpreted as a graben development related to a large scale crustal culmination in the sense of Cloos, 1953. No observable major normal faulting is present, however, along the main part of the Red Sea. Indeed, the Gulf of Suez is controlled by a true graben structure, probably as a result of the tension created by the 105 km horizontal relative movement along the complex sinistral wrench fault through the Gulf of Aqaba and the Dead Sea. Also, the southern termination of the Red Sea in the Afar triangle is fault bounded.

Because of the lack of major block faulting along the Red Sea, it is now generally accepted that the main structure of the Red Sea has developed as a result of a large scale stretching and thinning of the continental crust, accompanied by the down flexure of an elongate basin, rising of mantle material from below (Fig. 2), and large swells on the African and Arabian sides. In the Pliocene the thinning resulted in the formation of an axial rift, starting from the south (Cochran 1983). Magnetic and gravity data confirms that active sea floor spreading today occurs as far north as about 23° N, cf. Fig. 3 (Girdler 1969, Allan 1970).

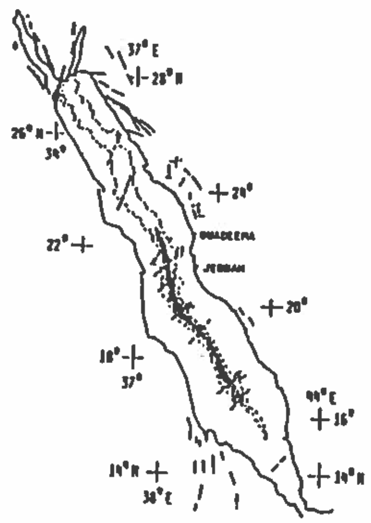

Fig. 3
Red Sea with 500 fathoms contour (---) and major faults. Large parts of the area of active sea floor spreading (hatched) are more than 1000 fathoms deep

To this corresponds a general rising of the coastal regions south of about 25° N, whereas the northern part is either static or sinking. However, local deviations from this general pattern are widespread, possibly due to salt tectonics.

2. SEISMIC ACTIVITY

Fig. 4 shows the epicentres of recorded major earthquakes before 1981. It should be noted that the three nearest seismographical stations are about 2000 km away from Jeddah, for which reason the minimum main body wave magnitude recorded is MB = 4.0.

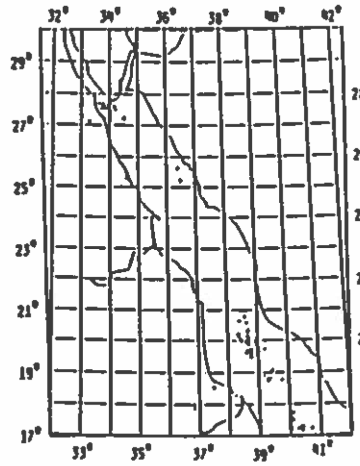

Fig. 4
Epicentres in the Red Sea

The earthquakes near the southern tip of the Sinai peninsula are of no interest for the Quadeema site, because they are related to the block faulting in the Gulf of Suez and to the wrench fault of the Gulf of Aqaba.

The three earthquakes between latitudes 26° and 25° are near the present coastlines and may be assumed to be related to major faults in the continental crust.

The many earthquakes along the axial zone south of 21° must mainly be due to movements along the transform faults (cf. Fig. 3), and to less extent to magmatic activity in the spreading sea floor zone. Thus, the seismic activity is compatible with the tectonic model discussed above (Fairhead and Girdler 1970).

3. SEISMICITY

The seismicity of interest for the Quadeema site cannot be directly calculated from the total number of earthquakes. For example, between 1967-03-10 and 1967-09-21 a total of 37 shocks were recorded, all associated with the main shock of 1967-03-13, 19.22 GMT with MB = 5.7. These shocks are counted as only one (genuine) event.

Defined in this manner, only 11 events have taken place near the axial zone north of latitude 17° over a period of 31 years. These events are plotted in the left hand side of Fig. 5 as a frequency distribution curve N per year, accumulated from north towards south. Thus, the stepped curve represents the frequency of the total number of events north of the latitudes written as abscissae. The vertical lines of the steps represent the latitudes of the epicentres of the main shocks.

The stepped curve is approximated by the dash-dotted line. The steep slope of this curve is clearly correlated with the relative movement of the two plates and the opening of the rift from the south.

By extrapolation of the dash-dotted line to 22°, it is seen that a frequency N = 0.01 per year is on the conservative side for all events north of this latitude. Fig. 3 shows that magnetic

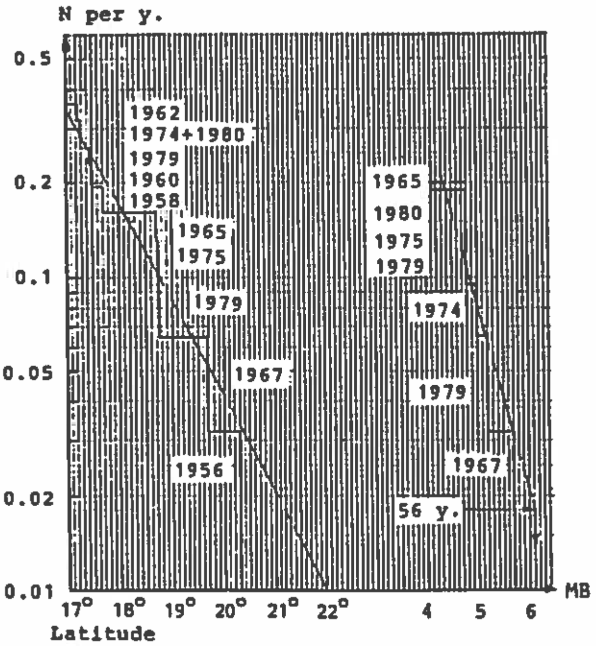

Fig. 5
Rift seismicity of the Red Sea. Distribution curves for frequency N (decreasing towards north) and for body magnitude MB

records have revealed a transform fault crossing the rift at 22°. Though no earthquakes from this fault have been recorded so far, it is a potential source, to which the value N = 0.01 should be assigned, because the frequency pertaining to the fault at 24° will be very much smaller. (The fault at 21.3° is also a potential source of earthquakes. These need not be considered, however, because of the longer distance to Quadeema.)

Accepting a risk of 5% for an assumed 100 year life of structures at Quadeema, the period to be considered is 2000 years, corresponding to 20 events at 22°.

In order to estimate the magnitude of the worst out of 20 events at 22°, the following approximate reasoning is applied: The 11 events between 17° and 21° required 31 years. 20 events would therefore require $(20/11) \cdot 31 = 56$ years. The worst of these events would have a probability of $1/56 = 0.018$ per year. In the right hand side of Fig. 5 the distribution curve of MB-values has been plotted for the 7 (out of 11) events for which the body magnitudes have been determined. The dash-dotted line gives by extrapolation a conservative value of MB = 6.1 as design earthquake.

4. SHAKING AT QUADEEMA

The transform fault nearest Quadeema is believed to cross the talweg of the central trough approximately at 22° (Coleman 1973). In order to be on the safe side, the potential epicentre has been assumed to be situated 25 km NE of the crossing point, i.e. with a distance of 100 km from Quadeema.

Only in the western United States a sufficient amount of strong motion data is available for the determination of the attenuation of the acceleration with distance and the dependence of the acceleration on local soil conditions.

The depth to bedrock being unknown at Quadeema, the surface acceleration has been determined from the curves given by Seed et al (1976) for deep cohesionless soil conditions. Including a correction for the magnitude of the earthquake, the maximum horizontal acceleration has been found to be 0.04 g.

Because of the short construction time available, a steel sheet pile wall had to be driven in front of a coral reef through very loose layers of sand, silty sand and sandy clay, in some cases with SPT-values as small as N = 0-1 through a layer of thickness 7 m.

It was decided to apply vibro-compaction of certain strata of vital influence on the stability of the quay wall. The necessary compaction was determined by various stability criteria, among which the earthquake considerations were also important.

Since small local earthquakes could not have been recorded by the distant seismological stations, it was necessary to carefully consider the local geology (see below). In this connection it was of utmost importance that the older theory of a graben structure could be discarded with because of the work by R. G. Coleman and others.

5. LOCAL GEOLOGY

The geology of the Quadeema area has been deduced mainly from geotechnical borings, information from the consulting engineers, 1:500,000 Geological map sheet (U.S. Geological Survey), aerial photographs and Erts satellite photos.

The area is possibly underlain by Miocene evaporites and/or coarse clastics capped by Pliocene marine ooze, all of unknown thickness and in unknown depths.

The area is a dissected shallow watered coast of numerous coral reef islands and bars intermittent with areas and lagoons of muddy, silty and sandy carbonates and silicic sediments. The subsoil in the seabed consists of alternating formations of marine sediments of carbonates and silicates, reflecting changes in sea level and climate possibly related to glacial-pluvial periods. These formations can be divided in zones or horizons of comparable strength, which also have comparable contents of carbonates and moisture. In the deepest parts around elev. - 35 m cemented carbonate siltstones and marlstones occur. Interfingering the formations of marine clastic sediments there are coral reef structures of solid limestone. These show discontinuous growth and erosion resulting in cemented coral rubble, likewise reflecting changing water level. The coastal zone rises to a level of about + 3 m, which is succeeded inland by a gravel plain with windblown sand at a level of about + 12 m. This plain, dissected by numerous wadis, rises inland for a distance of 5 km, where it terminates in the pediments of low table mountains. These consist of Precambrian basement gneisses capped by Tertiary sandstones and Mio-Pliocene basalts. This terrain is dissected by linear valley features, which appear to be fault controlled and of which some of E-W and NE trend may be post Pliocene as they displace the basalt lavas.

There is no sign of the existence of major coast parallel normal faults. Such might be covered by the gravel plain. On the other hand, this seems unlikely as promontory table mountains both to the north and to the south reach far west and come close to the coast, thus hardly leaving space for such major features.

On aerial photographs the coastal zone of reefs and recent marine sediments shows numerous linear features, bounding islands or cutting the solid coral limestone exposures (Fig. 6). One frequent system appears to be parallel to the

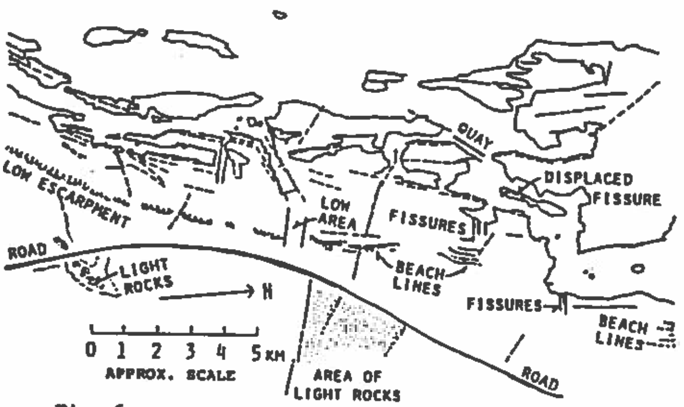

Fig. 6
Lineaments (---) found by interpretation of aerial photographs

general N-S trend of the coast, and other less prominent systems are more east-westerly directed or trend north-east. Displacements of up to 0.3 m related to the N-S system, as well as open tension cracks, have been observed.

The age of these features is uncertain. However, as the coral reef limestones possibly are Pleistocene or late Pleistocene and in their surface have been subjected to erosion related to a lower sea level, their age could be anything less than 100,000 years. Other features, which apparently can be followed into areas of soft unconsolidated sediments and which in the coral reef rocks vertically displaces the present surface, could be very young if not recent.

There is thus some reason to believe that the area as shown by the local geology has been subjected to movements of Pleistocene and possibly also recent age. These movements can hardly be directly related to earthquake generating faults. They are therefore regarded as secondary effects. Finally, it should not be excluded that the movements could be caused by salt diapirism.

ACKNOWLEDGEMENTS

Danish Hydraulic Institute was responsible for the design of Port Quadeema for the Saudi Arabian Ministry of Public Works and Housing. The authors had valuable consultations with Professors *H. B. Seed* and *J. Lysmer* of University of California, Berkeley and with Mr. *R. G. Coleman* of U.S. Geological Survey. They also had inspiring cooperation with many engineers from the firms *Cowiconsult*, *Portconsult* and *Rambøll & Hannemann*, as well as leading staff from *Danish Geotechnical Institute*, which played an important role in defining problems and carrying out laboratory tests and analyses.

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NEW TANKER TERMINAL AT BIZERTA, TUNISIA

by

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R&H

October 1983

NEW TANKER TERMINAL AT BIZERTA, TUNISIA

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1. INTRODUCTION

In the beginning of 1981, the National Ports Authority of Tunisia, OPNT, foresaw that an increase of the crude oil import capacity of the Port of Bizerta would be necessary as a consequence of a planned extension of the oil refinery at Bizerta.

The Port of Bizerta is located almost at the northernmost point of Tunisia, exposed to waves from the Mediterranean in the quadrant from North to East.

The existing port facilities are mainly located in the canal connecting the large lagoon "Lac de Bizerte" with the sea. The mouth of this canal is protected by a northern and an eastern breakwater projecting from the coast, and a detached breakwater covering the entrance between the two other breakwaters. (see figure 1).

The scheduling for the oil refinery allowed a total period of about 3 1/2 years for establishing the new import facilities.

The refinery planned to use tankers up to 100 000 DWT or 150 000 DWT for the crude oil import.

The planning and design was divided into two phases: a feasibility study and preliminary design phase, covering site selection, selection of type of berth and selection of ship sizes for the future oil import, and a detailed design phase covering the detailed design, tendering and contracting for the new facility.

This article describes the first phase. The second phase is described in reference (6).

2. SELECTION OF LOCATION AND PRELIMINARY DESIGN OF BERTH

2.1 Initial planning

After a site reconnaissance and discussions with the client and local authorities five alternative terminal locations were chosen as a basis for the study. The five locations are shown in figure 1 and these five alternatives were considered to cover all reasonable possibilities for a new tanker terminal.

There are three principal types of terminal:
- fixed berth with bridge connexion to land (Alternatives 1 and 2)
- fixed berth connected to land by subsea pipelines(3,4)
- single point mooring buoy (SPM) located offshore and connected to land by subsea pipelines (5).

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The different locations can be characterised as:
- well protected, inside the existing outer harbour (1 and 3)
- protected from direct wave action, but exposed to waves that run over a breakwater (4)
- partly protected against direct wave action (2)
- unprotected, offshore location (5).

The alternatives have thus very different service conditions (navigation manoeuvres and down times) and capital costs (dredging, structures and pipelines). Favourable service conditions are coupled with high capital costs and vice-versa, so a rational programme for choosing the best solution had to be used, and the programme adopted is described in the following.

2.2 Preliminary site investigations

A programme for preliminary site investigations was prepared in order to provide parameters that could not be established from existing information.

The programme consisted of a general bathymetric survey and a soil investigation covering the alternative locations of the terminal. Furthermore a general picture of the currents in the area was established by float tracking. This, together with sediment sampling partially gave the basis for the evaluation of sediment transportation.

Bathymetric survey

The bathymetric survey was carried out by echo-sounding and resulted in a bathymetric chart with grid size approx. 25 x 50 m.

Preliminary soil investigation

The preliminary soil investigations consisted of one boring at each alternative location and one boring supplemented by a number of wash borings in the area to be dredged. Some difficulty was encountered in getting the borings deep enough, partly due to the continual wave movements of the floating barge which was used. The investigations resulted in preliminary design parameters for the soil.

Current measurements

A few float trackings showed that both the ebb current and the flood current would have significant influence on the manouevering of vessels at all potential pier locations. The float trackings were executed during springtide and extrapolations to the extreme spring and neap situations were done on the basis of the study referred to in section 2.4. The max. current velocity measured in the manoeuvre zone was 1.23 m/s during spring ebb tide and 0.36 m/sec during spring flood time. See figures 8 and 9.

Evaluation of sediment transport

The sediment transport pattern was estimated by general inspection of the nearby coastal area and sediment sampling using a handheld sea bed sampler at 19 points throughout the outer harbour and outside the harbour. On this basis, together with the current pattern it was concluded for the alternatives 1,3 and 4 that sedimentation would not occur in the dredged basin. For the alternatives 2a, 2b and 2c moderate general sedimentation together with some stronger local sedimentation in the near coastal part of the dredged basin was to be expected. The rate of sedimentation was estimated to be 15 000 - 20 000 m3/year in a basin dredged to -18 or -20.5 m.

2.3 Wave analyses and movements of the moored ship

Due to the lack of field data on waves a comprehensive wave study was conducted at the Danish Hydraulic Institute. The study was done on a mathematical model and resulted in a description of the wave conditions outside the harbour and at the alternative pier locations. The model included a wind wave hindcasting routine, a wave refraction routine and a wave diffraction routine.

The hindcast was based on 10 storms selected from storms that had occurred in the period 1971 to 1981.

The resulting long term wave statistics outside the harbour and at the alternative pier locations are shown in fig. 2.

The corresponding peak periods in the wave energy spectrum T_p are shown in the following table

Ho (m)	2,0-2,5	3,0-3,5	4,25-4,75	5,25-6,00	6,25-7,25
Tp (sec)	7-10	9-11	10-12	12-14	13-15

Peak periods in wave energy spectrum.

The wave study indicated that seiches with periods 100 to 140 sec. could exist in the harbour affecting the ship movements for the alternatives 1 and 3.

Based on experiences from earlier model tests it was concluded that the ship movements would be acceptably small for all alternatives if significant wave heights did not exceed 2.0 m outside the harbour.

The down time due to movements of moored ships is described further in section 2.6.

2.4 Tide and current analyses

Although the tide in the Mediterranean is generally of low amplitude the actual tidal levels are of direct importance in the choice of dredging depths and of indirect importance because of the tidal currents that they give rise to.

According to tide tables (1) the following astronomical levels are valid at Bizerta:

Tidelevel	MLWS	MLWN	MWL	MHWN	MHWS
Height above datum	0.12	0.18	0.24	0.30	0.37

Tide levels, Bizerta

The speed of the current in Chenal de Bizerte is a function of the difference in waterlevels between Lac de Bizerte and the outer harbour. A mathematical treatment of this tidal inlet problem gave the following maximum current velocities, U_{max} in the mouth of Chenal de Bizerte as a function of the tidal range, H_o:

H_o (m)	0.12	0.15	0.20	0.25	0.30	0.35
Umax (m/sec)	0.76	0.88	1.08	1.32	1.40	1.60

Current velocities in Chenal de Bizerte.

As the currents in the outer harbour and outside the harbour are controlled by the current in the canal the extrapolations of current velocities as measured during site investigations were done using the velocities shown in the table.

2.5 Preliminary manoeuvre evaluations and dredging layout

In view of the strong currents the navigation aspects became very important for the selection of the location and the orientation of the pier.

Preliminary manoeuvres were evaluated in close cooperation with experienced pilots. For each alternative the arrivals and departures including the use of tugs were described. The restrictions because of waves and currents were estimated assuming that manoeuvering with tug assistance was impossible with significant wave heights larger than 2 m. As regards currents no general rule was used since the type of the manoeuvre and the available space are important factors.

It was concluded that no acceptable manoeuvres could be proposed for the two alternatives situated in the outer harbour (alternatives 1 and 3) because of currents. For the other alternatives the proposed manoeuvres for 100 000 DWT tankers generally required the use of 1 tug of 3000 HP and 2 tugs of 1000 HP. The proposed manoeuvres for the 150 000 DWT tanker generally required an extra 3000 HP tug. An example of the graphic representation of the arrival at alternative 2a of a 100 000 DWT tanker in an ebb current is shown in fig. 4. The down time aspects are described in section 2.6.

The area to be dredged was preliminarily laid out using the envelope of the proposed ship trajectories and including the necessary safety areas.

The preliminary dredging depths were estimated using recommendations from PIANC (2).

The required dredging depths were characterized by the coincidence of extreme low water and significant wave height 2.0 m.

No differentiation of the manoeuvering area was made at this preliminary stage and the necessary waterdepths were found to be 18.2 m for 100 000 DWT tankers and 20.9 m for 150 000 DWT tankers (supposing a sandy sea bed).

The required waterdepth for 100 000 DWT tankers is composed of the following parts:

Draught (typical)	14.8 m
Trim	0.3 m
Squat	0.1 m
Vertical ship movements	2.4 m
Sounding accuracy	0.1 m
Net underkeel clearance	0.5 m
Total depth	18.2 m

For a rocky sea bed the net underkeel clearance must be 1.0 m.

2.6 Traffic Analysis

The traffic analysis covered both a survey of oil tanker statistics and a comparison of the alternative berth locations with respect to ship waiting time.

The principal statistics of all existing 1083 oil tankers in the deadweight range 70 000 tonnes to 200 000 tonnes were obtained (3) and put onto computer files. It was seen that the tanker population was concentrated in the two ranges 80 000 DWT to 100 000 DWT and 130 000 DWT to 150 000 DWT, thus confirming the relevance of the basic design tanker sizes 100 000 DWT and 150 000 DWT mentioned earlier. Computer plots showed the dimensions length, beam and draught graphically as functions of deadweight for all 1083 ships.

Draughts, being the most important ship parameter in the design of the civil works, were examined more closely. It was found that the draughts of tankers built during the last decade were independent of the year of building, thus indicating the abscence of any design trend to give tankers radically different draughts in the future.

The distribution of tanker draughts was analyzed, giving results which are shown in figure 6. From the figure the number of tankers in a given range with draughts larger than a given draught can be found. (e.g. there are 27 tankers in the deadweight range 70 000 tonnes to 100 000 tonnes with draughts larger than 15.0 m). The figure was used in fixing design draughts which accomodated the majority of tankers but excluded atypical vessels. Results from the figure are used in the evaluation of the necessary dredged depth.

Ship waiting times at the alternative berth locations were compared using queueing theory. Firstly, downtime percentages were assessed on the basis of restrictions on arrival manoeuvres and restrictions on movements of the moored ship. Secondly, berth utilization factors were calculated from the given import quantity, the average oil quantity per tanker, the average service time and the downtime.

Various distributions of service times and arrival intervals were used in the application of queueing theory.

Erlang 2 distributions of arrival intervals and service times (E2/E2 system) usually give the best estimates of queueing times for specialized terminals (4). However experience with other bulk terminals had indicated that a hyperexponential distribution of arri-

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val intervals and a negative exponential distribution of service times (H/M system) could occur. A third system of distributions (M/E2), placed between the two first systems as regards evaluating queueing times was also considered. Operational data for the existing berth in the outer harbour for 50 000 DWT vessels was obtained, and it was found that the E2/E2 system fitted best. However, much of the existing traffic is in the form of a shuttle between Bizerta and the oil fields in Southern Tunisia using a single tanker. Queueing times for the new berth with its more random traffic pattern of import from further afield were therefore based on the M/E2 system. Figure 7 shows the results of the queueing analysis for the various queueing systems considered and various service times.

With downtime, utilization factor and queueing model fixed, it was possible to find ship waiting times for each alternative berth location. Ship waiting times for each berth location were converted to annual costs for ship delay, which were capitalized, and then combined with other costs to contribute to the economic comparison of the alternative locations.

The queueing analysis was also used to assess whether one berth was adequate for the design import quantity. It was concluded that one berth was adequate if the queueing model (M/E2) were applicable. A two berth solution could be relevant if the model (M/E2) was too optimistic or if the import quantity increased.

2.7 Preliminary design of fixed berth

The layout of the berth was based on OCIMF Recommendations (5) for placing fenders and mooring points. With 72 m between centres of fenders and with six mooring dolphins it was possible to cover the full range of tankers 30 000 DWT to 150 000 DWT. (Minimum tanker size 30 000 DWT was required for product export). A central platform 50 m x 30 m was found adequate for oil handling installations and a through road for access to a possible future second berth. See figure 3.

The berth foundations in the form of reinforced concrete caissons or piles were found to be suitable in the given soil conditions. A rough cost estimate for caissons was lower than that for piles and this estimate was used in the comparison of berth locations.

2.8 Selection of alternative

Having gone through the steps described above it was possible to select the best alternative on a purely rational, economic basis as all parameters in the choice are weighted with a cost figure. The relative values for the various parameters in the choice are shown in the table below (cost of 2a set to 100)

Alternative	1	2a	2b	2c	2d	3	4	5
Dredging	74	63	96	132	37	74	15	0
Structures	10	18	21	12	25	12	18	35
Pipelines	0	1	2	0	2	14	17	56
Ship waiting time	8	3	2	2	8	8	8	5
Manoeuvres (New tugs)	31	15	31	31	15	31	35	27
Total	123	100	152	177	87	139	93	123

Relative economic factors in the choice of alternative.

Alternatives 2d and then 4 were thus the most favourable. However, a later requirement was that the berth had to be able to be extended, thus eliminating 4, and making 2a more interesting. With the two berth solution 2a + 2d in mind, further analysis showed it more economic to start with berth 2a and extend with 2d rather than the other way round. Alternatives 2a and 2d were thus selected for detailed study, whilst 2a was to be designed in detail and constructed first.

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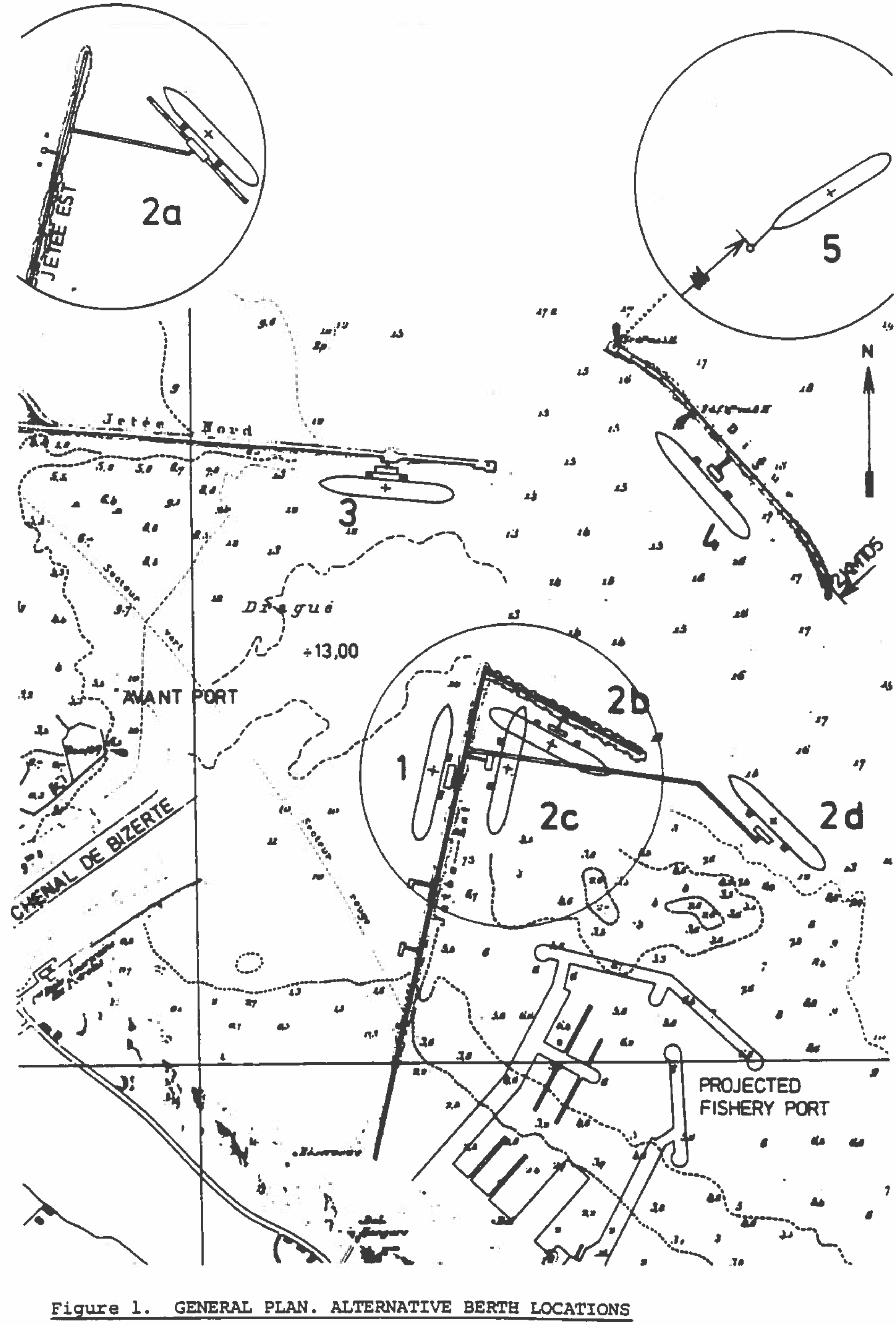


Figure 1. GENERAL PLAN. ALTERNATIVE BERTH LOCATIONS

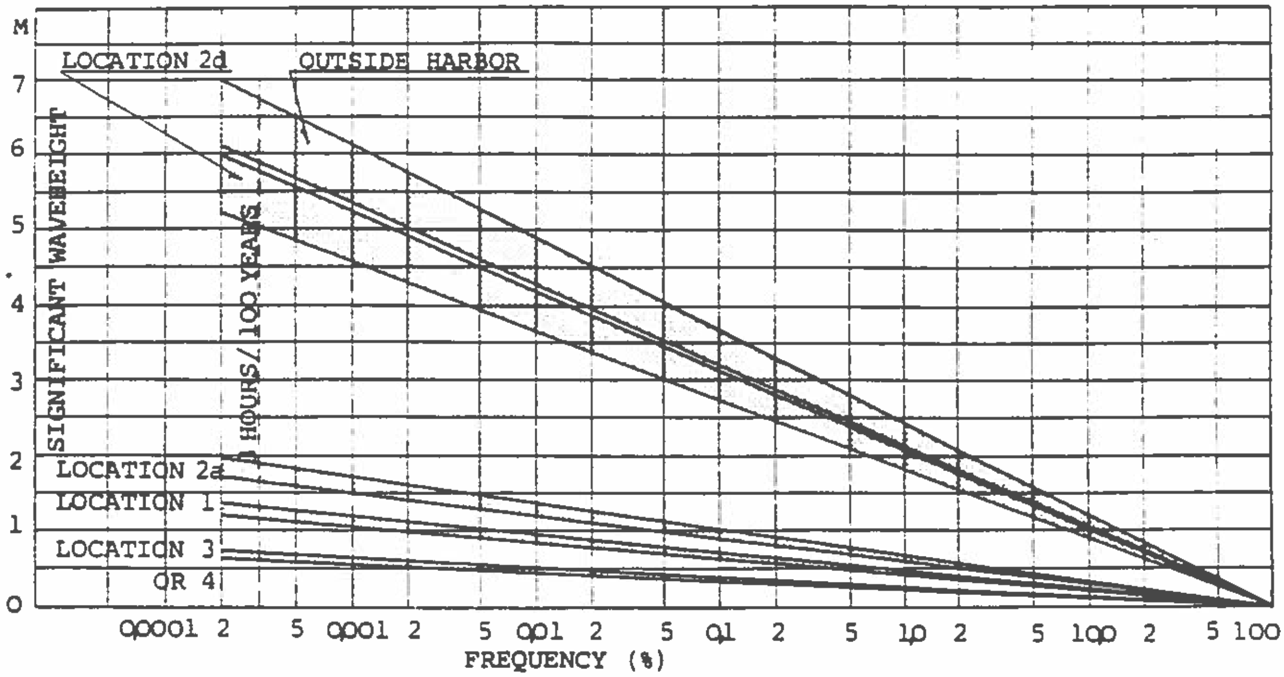


Figure 2. WAVE HEIGHT FREQUENCY (HINDCAST)

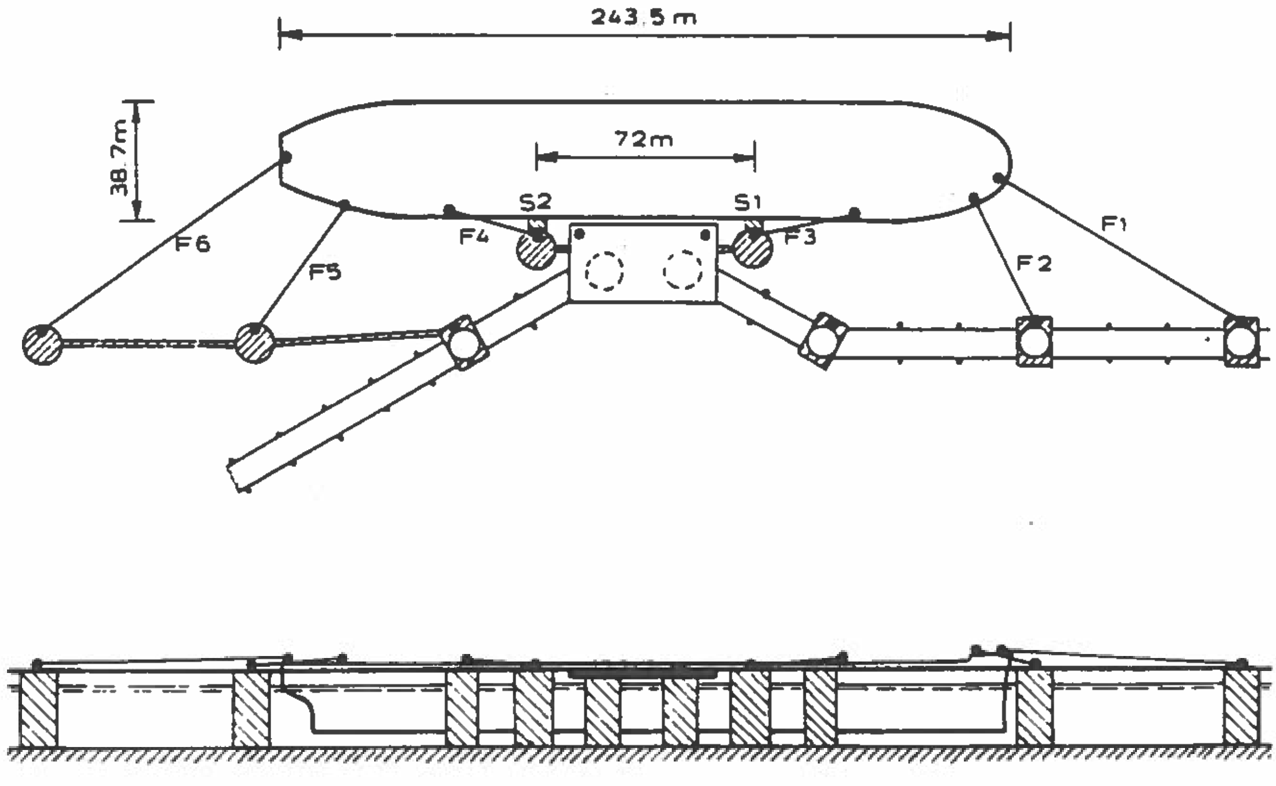


Figure 3. MOORING ARRANGEMENT. 100000 DWT TANKER

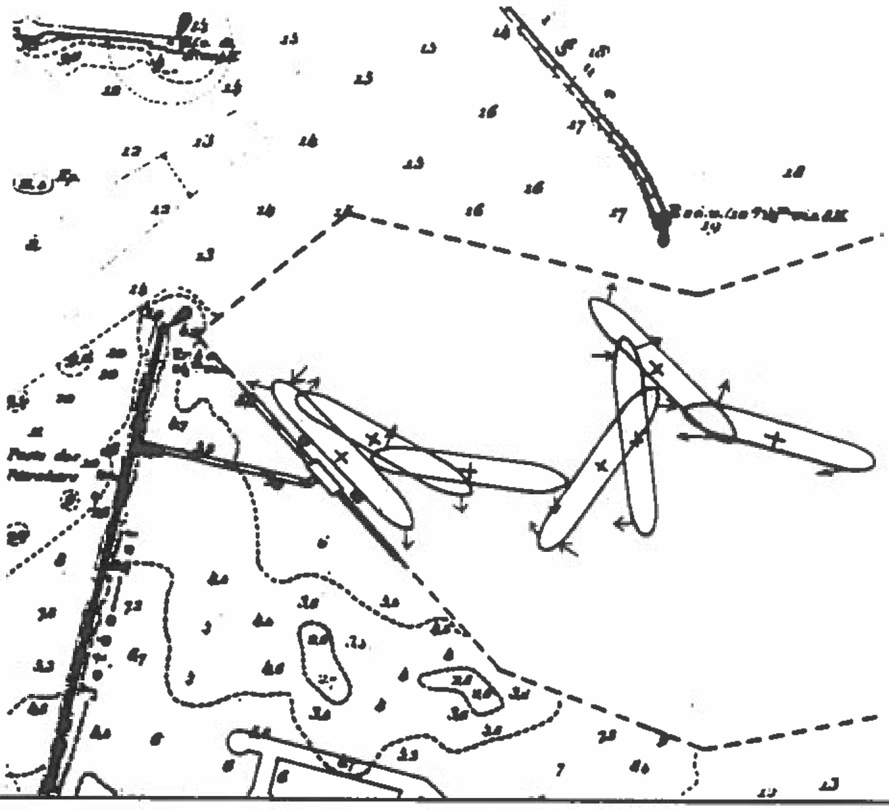


Figure 4.
ARRIVAL OF 100000 DWT
TANKER IN EBB CURRENTS
(GRAPHIC METHOD)

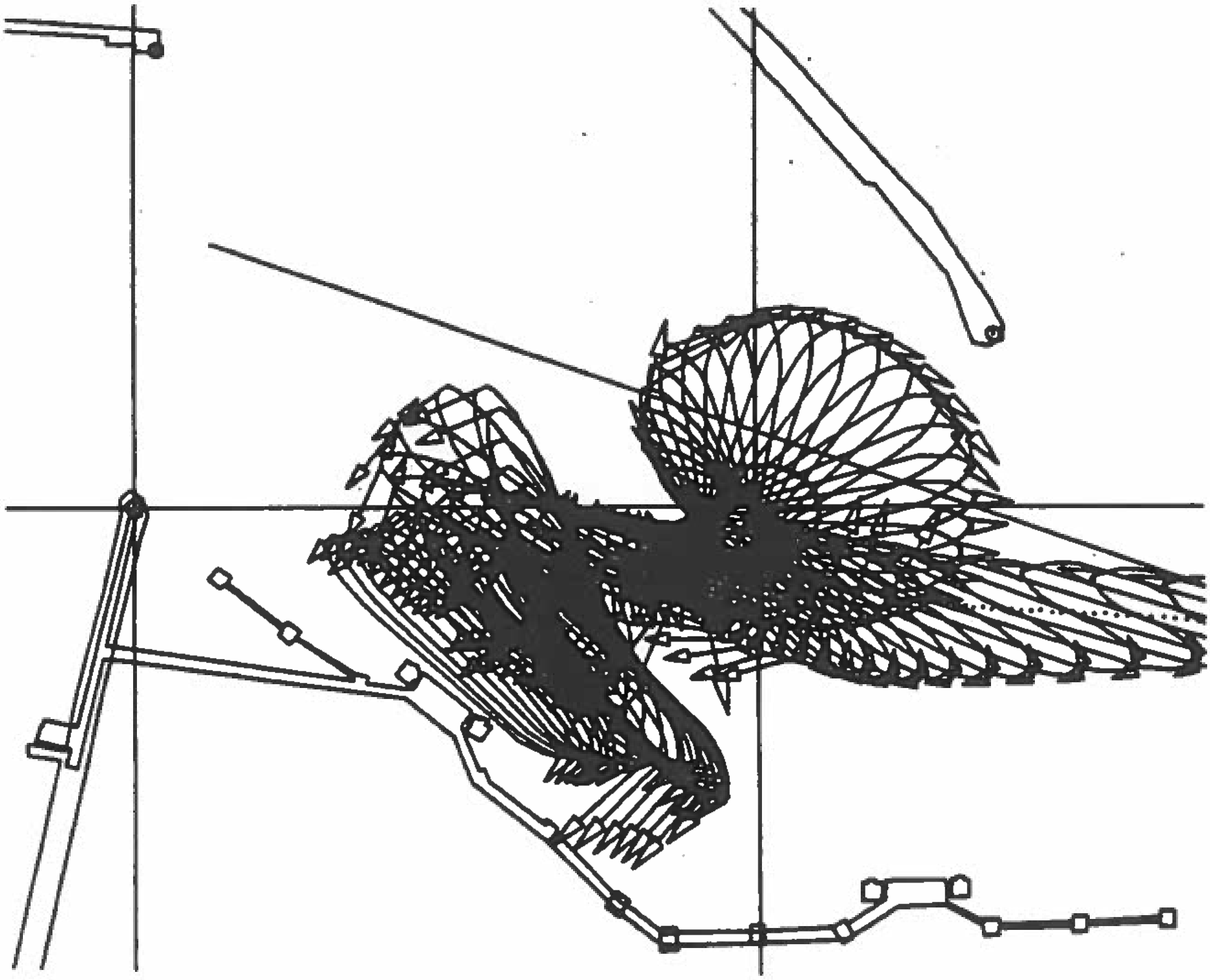
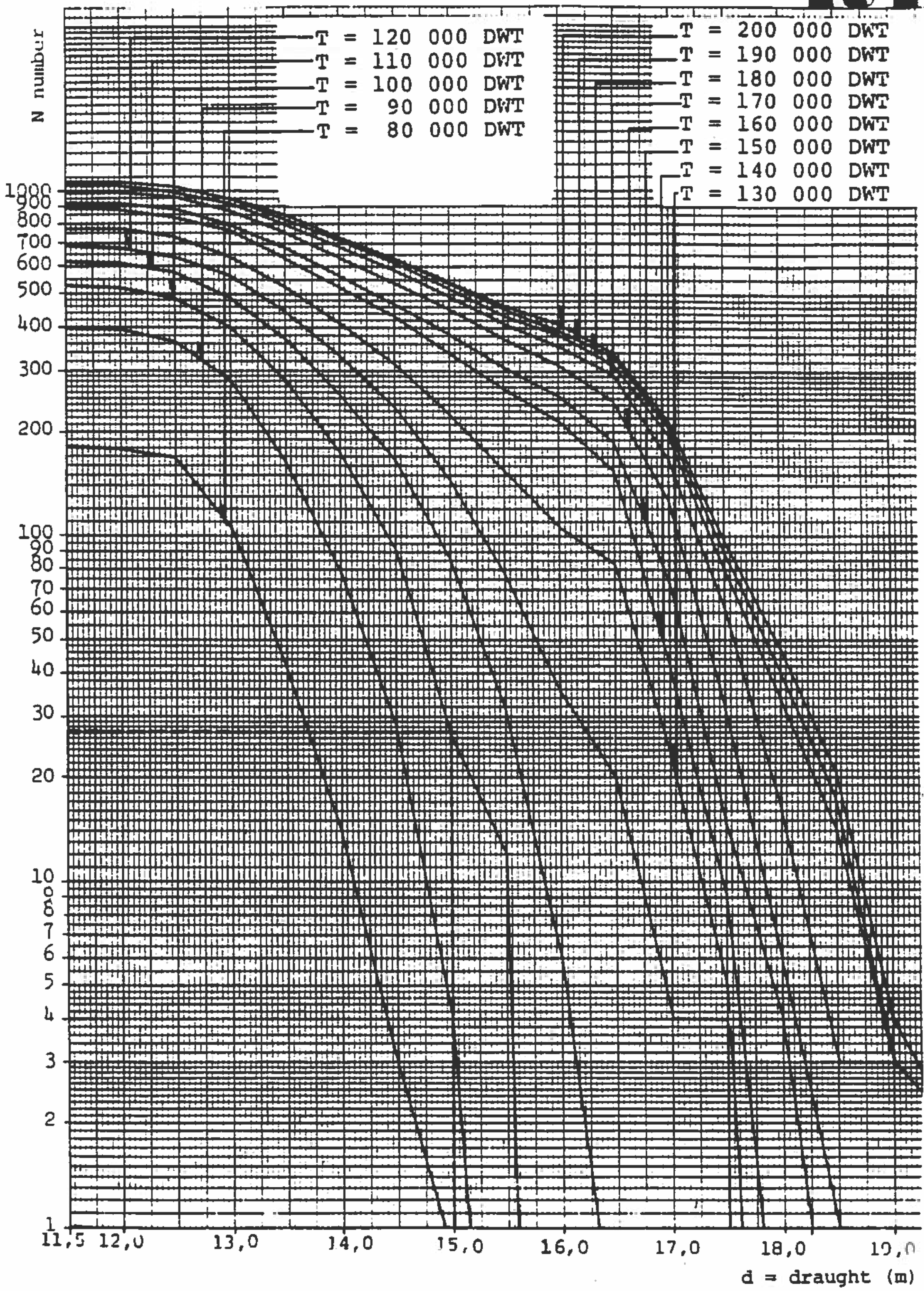


Figure 5. ARRIVAL OF 100000 DWT TANKER IN EBB CURRENT.
(COMPUTERIZED MANOEUVRE SIMULATION)



N gives number of tankers in the range
70000 DWT ⩽ tonnage < T with draught ⩾ d

Figure 6. TANKER DRAUGHT DISTRIBUTION

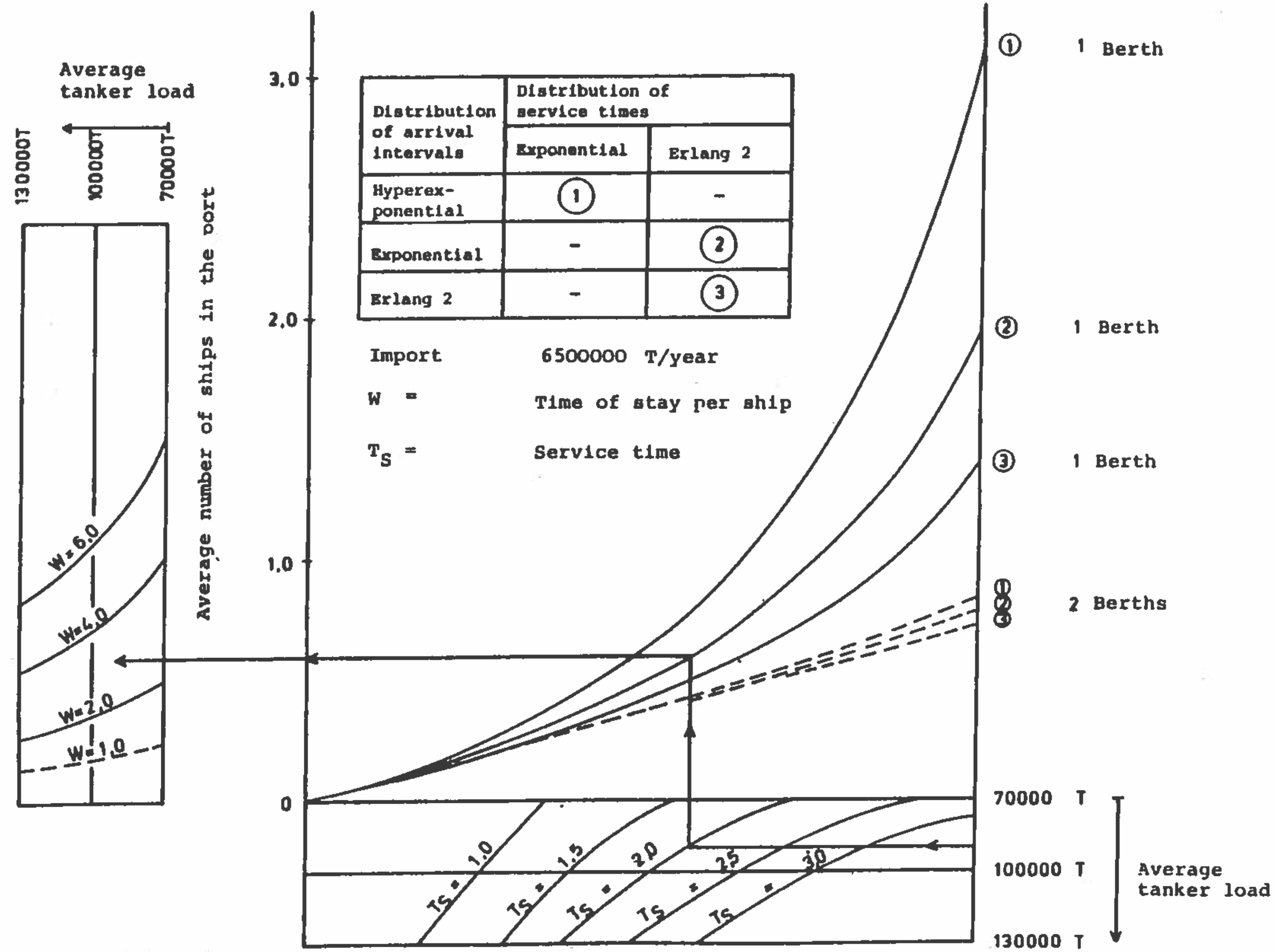


Figure 7. RESULTS OF QUEUEING ANALYSIS

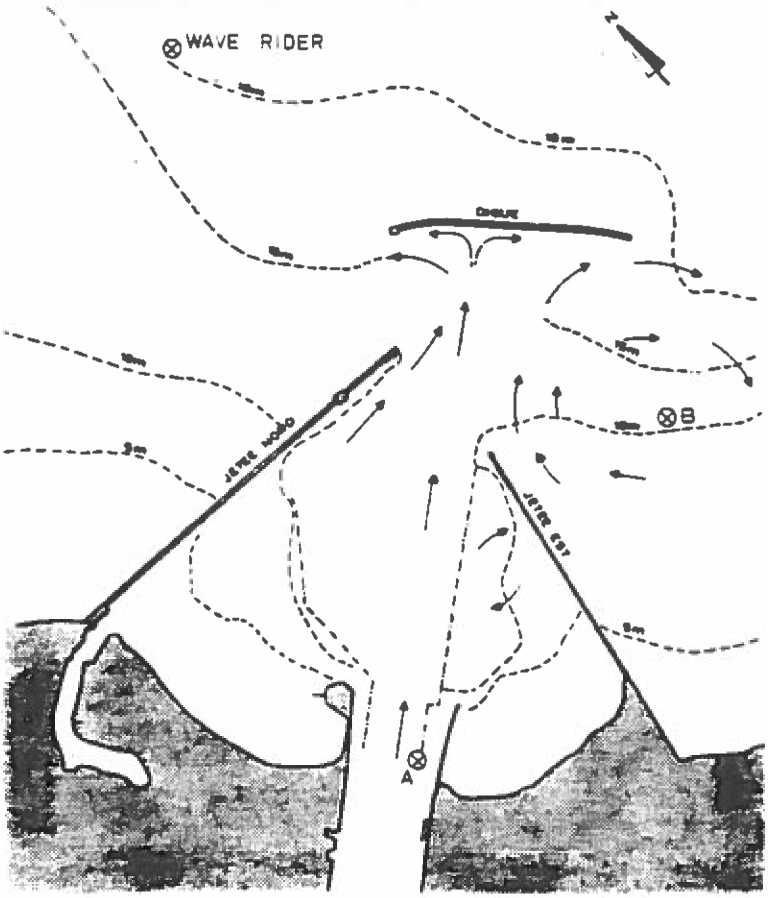


Figure 8.
EBB CURRENT PATTERN
(SITE INVESTIGATION)

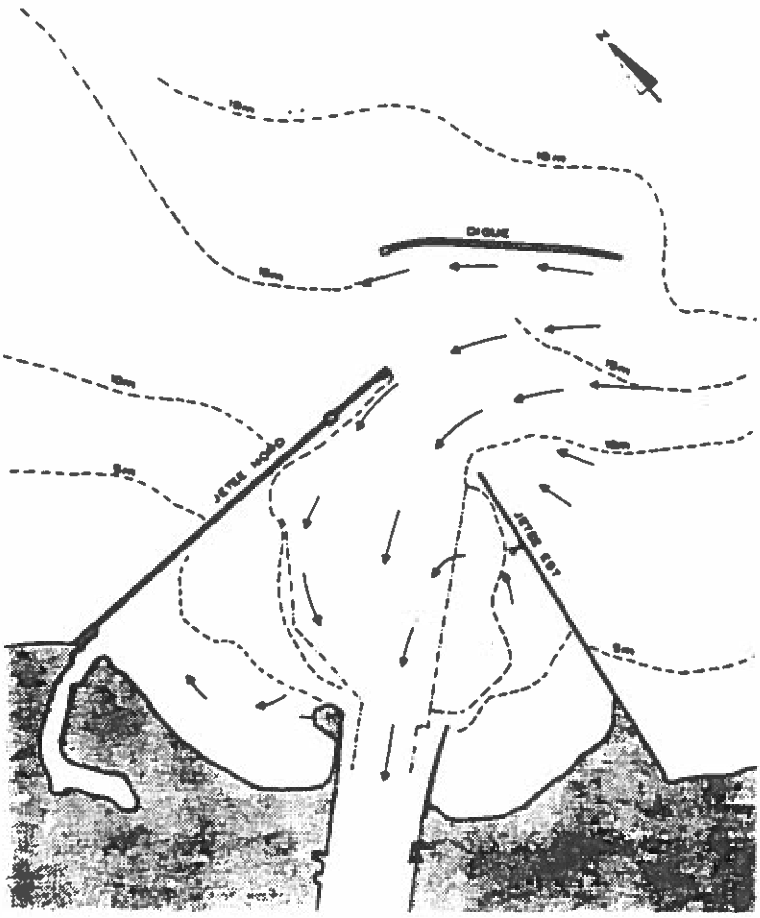


Figure 9.
FLOOD CURRENT PATTERN
(SITE INVESTIGATION)

TARTOUS - SYRIEN

ved

civ.ing. J. A. Jensen, Kampsax

F O R E D R A G

T A R T O U S - S Y R I E N

Projektet for Tartous havn er egentlig ganske ordinært, nærmest trivielt, uden særlige spændende problemer.

Det startede for 25 år siden, og dengang havde man ikke så mange sophisticatede metoder til rådighed. Derfor hviler afgørelserne - ud over en vurdering af anlægsomkostningerne - i høj grad på "det bedste skøn", som dog, anvendt under passende iagttagelse af sund fornuft, godt kan føre til tilfredsstillende resultater.

Eksemplet giver således en illustration af, hvordan et projekt har mødt tidens skiftende krav og løbende har kunnet tilpasses disse.

Figur 1 viser det østlige Middelhav.

I slutningen af 50'erne var der på Syriens kyst kun én kommerciel havn, nemlig Latakia, og den var endda kun en lægterhavn, idet skibene blev opankret uden for den åbne kyst, mens godset blev ført mellem skib og havnekajer med lægtere. Metoden var brugbar, for det østlige Middelhav er roligt hele året på nær december, januar og februar, men den er jo noget besværlig og tidskrævende og giver spild af gods, og da moderne skibe helst skal ekspederes hurtigt, var man indstillet på at gøre noget. Ved Latakia blev der derfor suppleret med en dækmole på dybt vand samt dybvandskajer, og ønsket opstod om at få endnu en havn længere mod syd, hvor valget faldt på Tartous, som kunne være alternativ havn for Beirut til betjening af den sydlige del af Syrien, samt transit fra Irak, Jordan og Saudi Arabien.

Figur 2 viser kystområdet ved Tartous.

Tartous ligger på en ret lige, typisk Middelhavskyst med vanddybder på ca. 20 m, to - tre kilometer ude og derfra stærkt affaldende bund til mange hundrede meters dybde.

Det er en klippekyst med kun beskedne sandstrande og terrænet stiger ganske kraftigt ind i landet. Langs strandlinien er der en klint, mere eller mindre stejl på 5 á 8 meters højde.

Byen Tartous ligger ud til vandet, men havde dengang ingen havn, kun en lille anløbspier for fiskerbåde, som var hjemmehørende på øen Ruad ca. 3 km mod sydvest.

Der var dog en stærkt tilsandet romersk havn og en moleruin fra en endnu ældre græsk havn.

På Figur 3 ses oplægget fra bygherren.

Man forestillede sig i første omgang en lægterhavn som i Latakia, med påfølgende successiv udbygning til en egentlig dybvandshavn.

Snart blev det dog klart, at lægterhavnen omgående burde suppleres med en dybvandskaj med mindst to anlægspladser.

Som man ser, forestillede bygherren sig dybvandshavnen udformet med kun én bølgebryder, hvorved havnen kunne udbygges successivt mod nord så at sige ubegrænset. Sådan er Beirut havn anlagt, og også Latakia havn blev i første omgang udbygget med kun én bølgebryder.

Imidlertid ligger disse havne begge i en havbugt, således at fremspringende land skærmer mod bølgeangreb og overflødiggør en sekundær bølgebryder.

Tartous derimod ligger på en lige kyststrækning, snarest endda på et fremspring, hvorfor det for enhver nyanlagt pier ville være nødvendigt at udforme dens yderside som en bølgebryder.

Vi foreslog derfor, som vist på Figur 4, at man i stedet byggede begge bølgebrydere fra starten, altså færdiggjorde havnens ydre rammer, og da det viste sig økonomisk overkommeligt, blev det vedtaget.

Lægterhavnen, som var vist placeret lige foran byens gamle kerne fra romertiden, flyttede vi op nord for bebyggelsen, således at man i hvert fald i første omgang undgik konflikt med den gamle by, ligesom adgangsforholdene her var langt gunstigere.

På dette tidspunkt havde man endnu ikke noget detaljeret kendskab til vanddybder og bundforhold. Efterhånden som disse forhold blev kendt, blev det klart, at hele havnen burde placeres længere mod nord, som vist på Figur 5.

Herved opnåede man 3 fordele:

1. Kortere vej mellem indsejling og det dybe vand.

2. Man kom fri af byen og kunne derfor opnå flere muligheder for adgangsveje og jernbaneforbindelser samt landarealer.

3. Takket være et knæk i kystlinien omtrent ud for Tartous, bliver havnen naturligt drejet 15 á 20^{o} med uret, hvorved havnemundingen drejes mere bort fra den hyppigste retning af større bølgeangreb.

Roen i havnen blev undersøgt ved modelforsøg (i Holland), men uheldigvis ikke på den endelige bølgebryderplacering.

Da sagen hastede, var penge bevilget og forsøg gennemført allerede inden markundersøgelser m.v. havde ledt frem til den nordlige løsning.

Imidlertid havde modelforsøgene vist en god overensstemmelse med de ved standardkurver konstruerede resultater, hvorfor den endelige planløsning kun blev undersøgt teoretisk.

Figur 6 og 7 viser de teoretiske kurver for bølgehøjder for farligste bølgeretning og de tilsvarende forsøgsresultater.

På grundlag af forsøgene blev havnepiererne drejet og forsynet med bølgeabsorberende ender for bedre at give læ langs kajerne og dæmpe uroen i yderbassinet.

Yderbassinet var desuden skånet for bølgerefleksion, idet bølgebrydernes bagsider bestod af stenkastninger.

Netop dette forhold er sikkert hovedårsagen til, at bølgeuroen i havnen kunne bedømmes ret sikkert ad teoretisk vej.

I den endelige planløsning, Figur 8, er den separate lægterhavn udgået og erstattet af lægterkajer langs den inderste del af havnebassinerne.

I øvrigt tænktes havnen udbygget med 3 stk. 175 meter brede pierer med varehuse, adgangsveje, jernbane, åbne pladser etc.

Området mellen den sydligste pier og hovedbølgebryderen har en uregulær facon. Det kan dog sagtens anvendes til udbygning med anlæg for massegodshåndtering, værftsanlæg og lignende.

I første udbygning, Figur 9, byggedes kun den sydligste pier med kaj kun på sin sydside. Større skibe betjentes ved dybvandskajerne eller de ankredes op langs bølgebryderens inderside og varerne ekspederedes med lægter via lægterkajen, som naturligvis desuden kan betjene mindre skibe. Havnen blev taget i brug i 1964.

Efter nogle år - sidst i 60'erne - byggedes syd for Pier A en massegodspier for eksport af fosfatmalm, Figur 10, og kort derefter udbyggedes Pier A's nordside med kaj, samtidig med at skråningen langs Pier A's yderende erstattedes af en kaj, uden at de derfra reflekterede bølger gav gener af betydning. Denne udbygning ses på Figur 11.

Man burde da samtidig have bygget Pier B's yderende som bølgebryder, men det undlod man, og resultatet blev da også ubehagelig stærk bølgeuro ved den yderste anlægsplads på Pier A's nordside.

Først i midten af 70'erne blev vi anmodet om at projektere den endelige udbygning af havnen. Resultatet ses på Figur 12.

Vi er nu i containertrafikkens tidsalder, delvis i form af ro-ro, og som man ser, bliver havnen nu indrettet i overensstemmelse hermed.

Udbygningen med varehuse er for en stor del erstattet af åbne arealer samt en stor hal for pakning af containere, og der er indrettet flere ro-ro ramper i bassinhjørnerne.

I det skæve område mod syd er yderligere projekteret et kajanlæg for svovl og kunstgødning, mens et planlagt værft endnu lader vente på sig.

Nordbassinet er marinehavn.

Arealmæssigt ser det således ud:

Forhavn	700.000 m^2
Havnebassiner	450.000 m^2
Landarealer	1,450.000 m^2
	2,600.000 m^2
Marinehavn	400.000 m^2
Uudnyttet areal i syd	150.000 m^2
	3,150.000 m^2 ∿ 3.2 km^2

hertil kommer allerede erhvervet ekstra landareal på cirka 400.000 m^2.

Nu kunne man stille sig selv det spørgsmål, om havnens planløsning var blevet væsentlig anderledes, dersom alle forhold vi nu har kendskab til om moderne godshåndtering havde været bekendt ved den oprindelige projektering.

Jeg tror det egentlig ikke, forudsat naturligvis, at der ikke opstår behov for en havn større end de lagte rammer.

Havnens generelle proportioner virker harmoniske: Mundingen nær dybt vand, passende forbassin, gode manøvreringsforhold, passende landarealer og gode adgangsforhold fra baglandet. Det lidt uregelmæssigt formede areal mod syd kan sagtens udnyttes til særlige formål og er i øvrigt opnået gratis, idet bølgebryderen ikke var blevet væsentligt billigere af at være ført ind vinkelret på kysten.

BUNDFORHOLD

Som nævnt før, er kysten klippefyldt, og havbunden består af kalkklippe med overlejring af ler eller sand i fra nul til ca. 4 m's tykkelse.

Uddybning i kalk har kun været nødvendig i de indre dele af bassinerne, i svajecirklen og i indsejlingsrenden. Den har kunnet ske med en stærk spandkædemaskine eller "rock-cutting dredger."

Fundering af kajer har stort set overalt kunnet ske direkte eller næsten direkte på klippegrunden, blot med en pude af stenfyld som mellemlag.

KONSTRUKTIONER

Bølgebrydere

Det klippefyldte område bød på muligheder for at åbne stenbrud mindre end 10 km fra havneområdet.

Stenen var hård kalksten, som egnede sig til både betontilslag og stenfyld. Til brug for molebyggeri kunne der produceres en rimelig procentdel blokke på 8 - 15 tons vægt.

Når dertil kom, at dybderne var maksimalt 13 m og havbunden var stabil, faldt valget naturligt på bølgebrydere af stenfyld med beskyttelseslag af naturlige klippeblokke.

Profilet blev dimensioneret efter Iribarrens formel og modificeret ved modelforsøg.

"30 års bølgen" på 7.0 m signifikant højde blev lagt til grund svarende til 1% skade.

Ved alle forsøg blev bølgeangrebet øget, til der opnåedes fuldt udviklet sammenbrud, så eventuelle uhensigtsmæssige brudforløb kunne afsløres og afbødes ved ændring af opbygningen.

Det viste sig bl.a., at man over højvandslinien kan gøre skråningen stejlere end længere nede, hvor Iribarrens og i øvrigt også

Hudsons formel syntes at passe pænt. Der gør sig ved Tartous det særlige forhold gældende, at indkommende bølger, før de når bølgebryderen, må passere vanddybder på omkring 12 m, hvilket betyder, at alle bølger på over ca. 8 m's højde brydes. Her ligger altså en øvre grænse for ikke alene signifikantbølger, men også for maksimumsbølger.

Dette er formodentlig forklaringen på, at denne havn var den eneste i det østlige Middelhav, som ikke led skader i den kraftige storm i januar 1968. Der var her formentlig tale om en ca. 80 års storm, med signifikante bølgehøjder omkring 9 m på dybt vand.

Molehovedet for hovedbølgebryderen udførtes med klippeskråning hele vejen rundt, mens den sekundære bølgebryders hoved for at lette navigationen udførtes med lodrette vægge til fuld dybde.

Kajer

Da der skal funderes på kalkgrund er åbne kajer på jernbetonpæle ikke nogen realistisk løsning. Pælene kan ikke rammes ned i kalken og kan ikke optage træk.

Vanddybderne på op mod 13.0 m samt korrosionsfaren gjorde, at stålspunsvægge ansås for uhensigtsmæssige.

Caissoner af jernbeton kunne være en løsning, som f.eks. var brugt ved Latakia. Imidlertid ønskedes der anvendt lokal uerfaren arbejdskraft, og armeringsjern var desuden importvare, hvorfor man søgte andre løsninger.

Valget faldt da på kajer af uarmerede betonblokke med blokvægt på op til 60 tons, som vi tidligere med held havde anvendt i tyrkiske havne.

Det er en robust konstruktion af stor holdbarhed, som desuden udmærker sig ved:

1) Materialerne er alle indenlandske.
2) Grej til støbning og udlægning er enkelt.
3) Arbejdsprocedurer og kontrol er enkle.

Blokkene oplægges i skråt forbandt, hvorved opnås, at man automatisk får de lodrette, eller næsten lodrette, fuger til at slutte tættest muligt. Fugerne er plane uden nogen form for fortanding, så eventuelle differenssætninger kan forløbe langs med dem. Selvfølgelig bliver en sådan blokmur ikke sandtæt, hvorfor der må bagfyldes med klippefyld, som også giver mindre jordtryk, end sand ville have gjort, takket være den højere skræntvinkel.

Der opnås endvidere en god dræning bag muren til sikring mod vandovertryk.

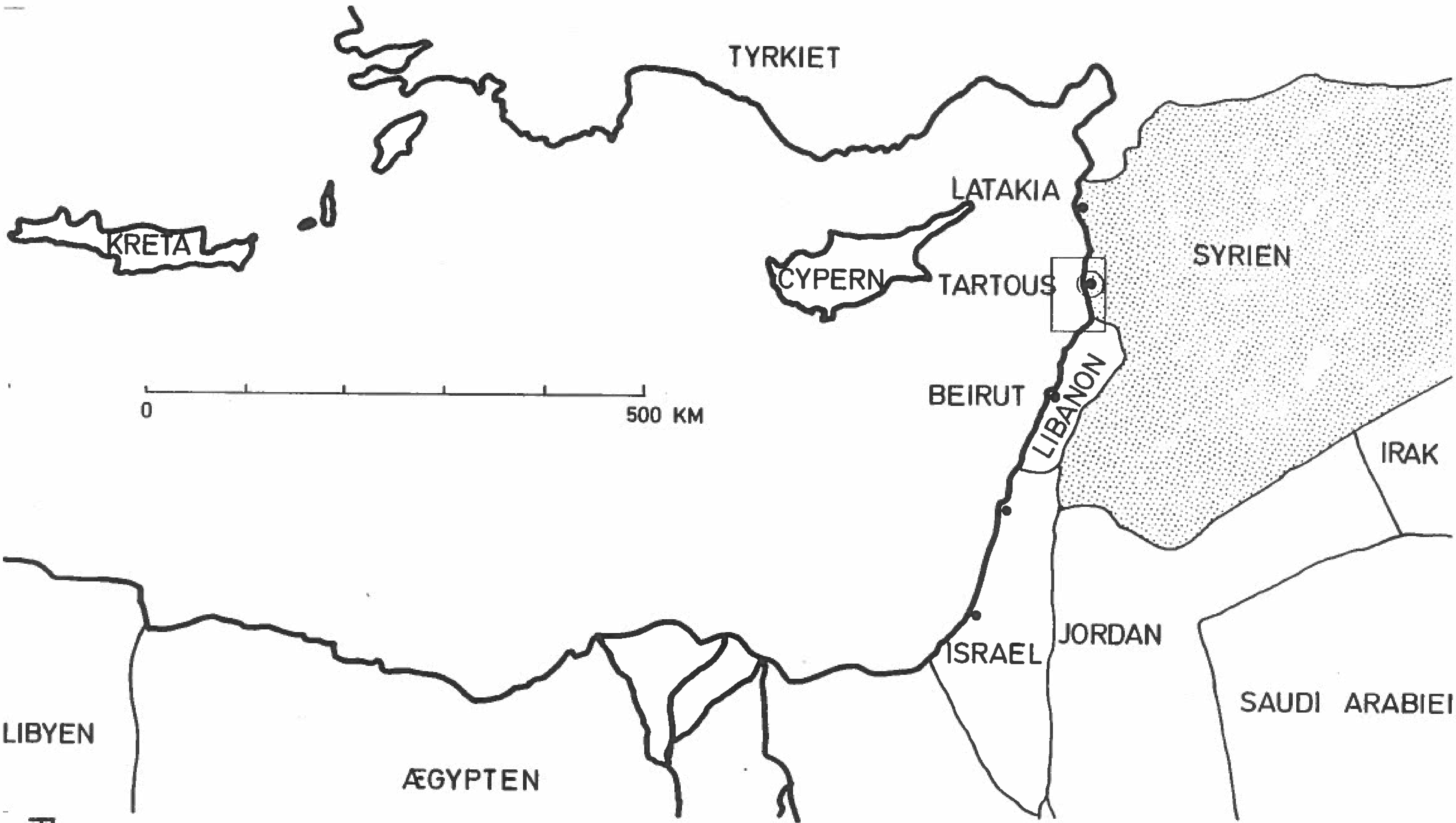

TYRKIET
LATAKIA
KRETA
CYPERN
TARTOUS
SYRIEN
0
500 KM
BEIRUT
LIBANON
IRAK
JORDAN
ISRAEL
SAUDI ARABIEI
LIBYEN
ÆGYPTEN

FIG. 1

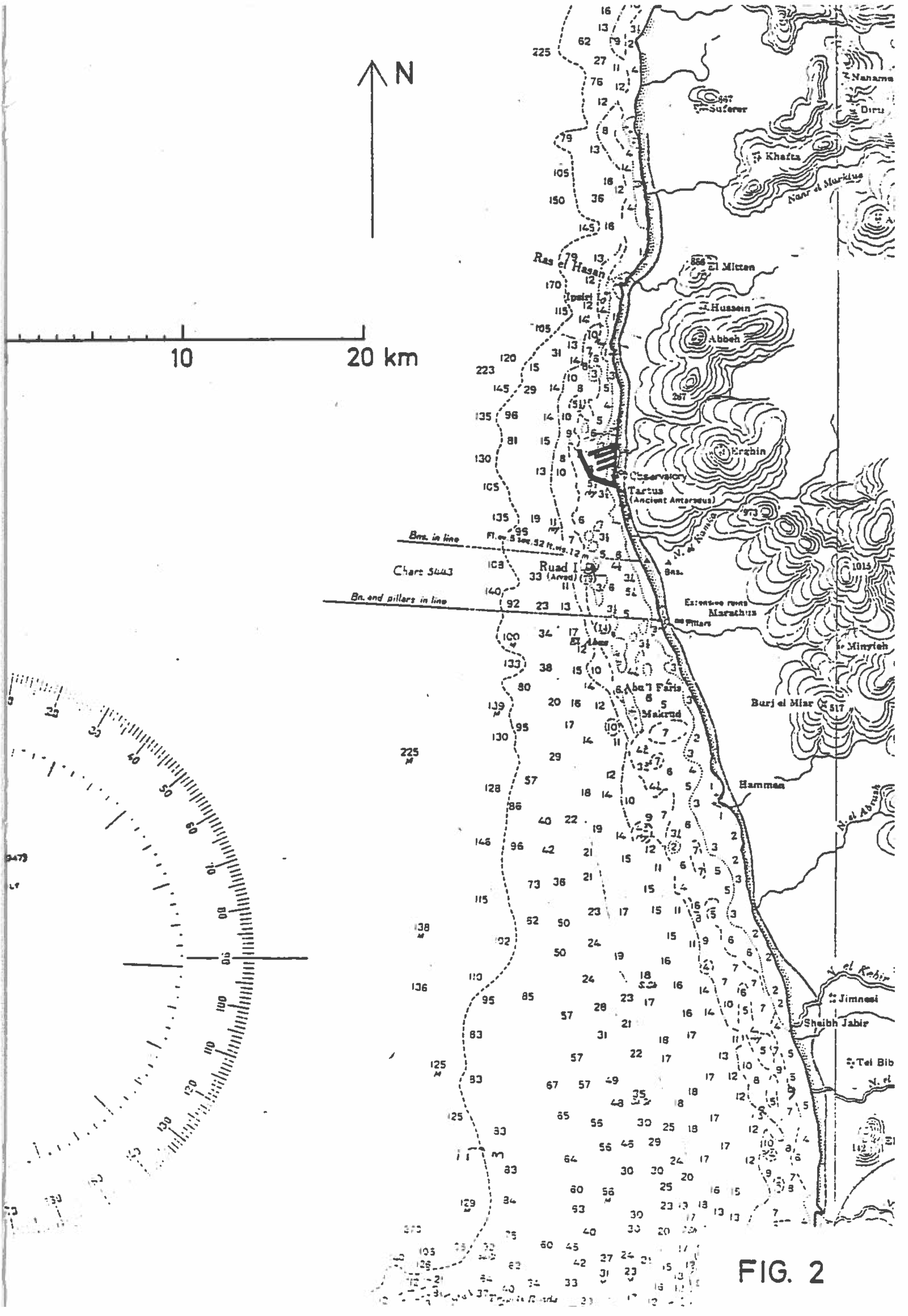
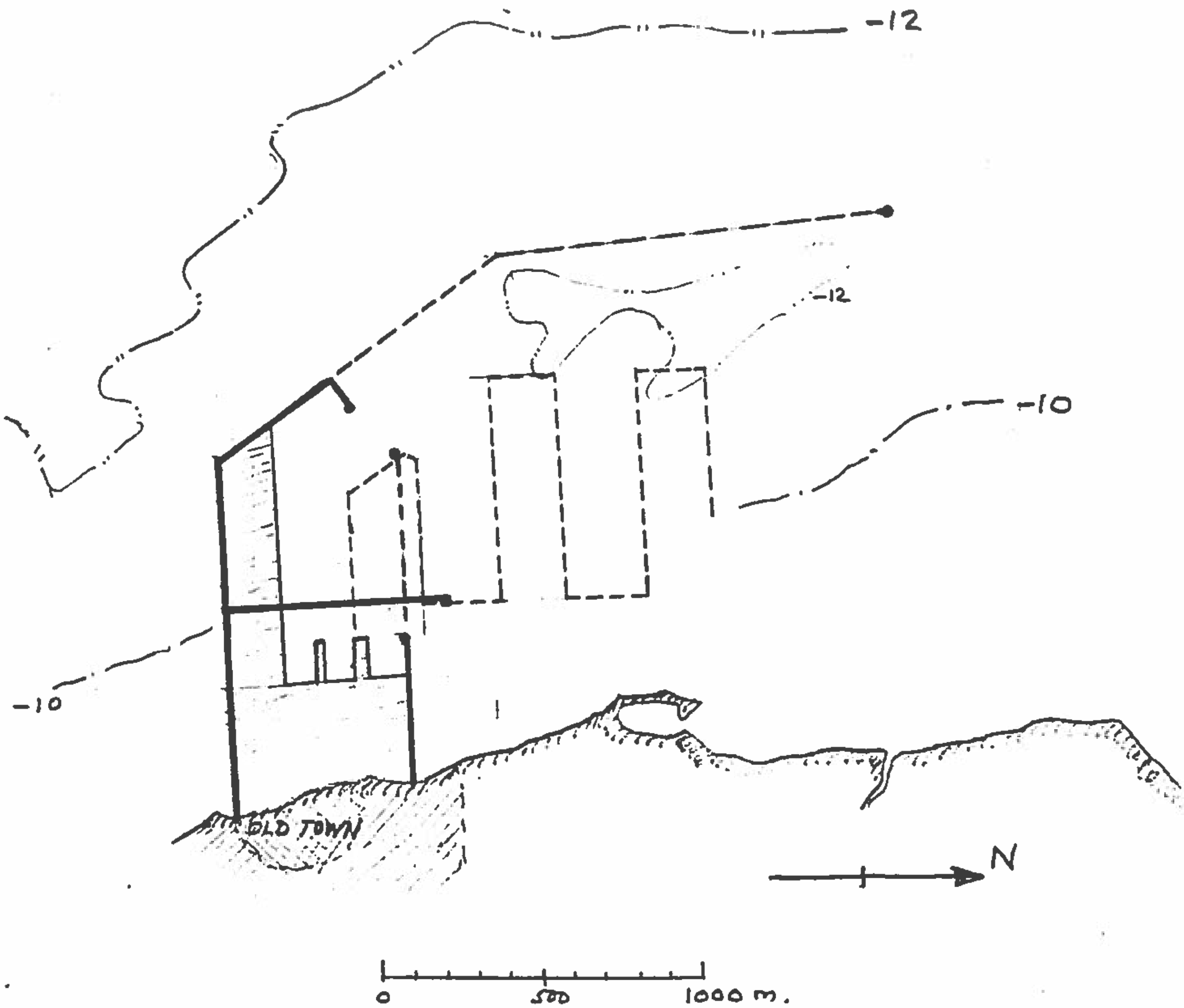

N
10
20 km
Ras el Hasan
Ipsiri
Suferer
Khafta
Nahr el Markius
el Mitten
Hussein
Abbeh
Erghin
Observatory
Tartus
(Ancient Antaradus)
Bns. in line
Ruad I.
(Arvad)
Chart 5443
Bn. and pillars in line
Extensive ruins
Marathus
Pillars
El Abas
Abu'l Faris
Makrud
Burj el Miar
Hammam
N. el Abrash
N. el Kebir
Jimnesi
Sheikh Jabir

FIG. 2



BYGHERRES OPLÆG, 1958

FIG. 3

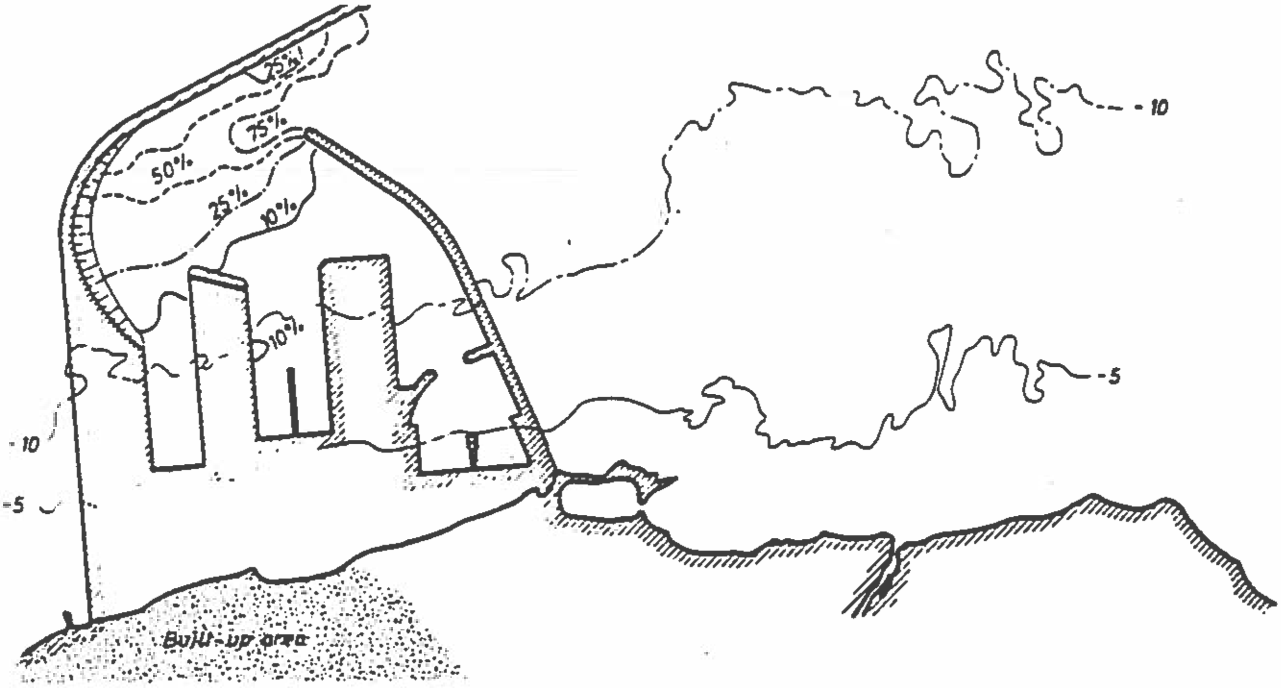
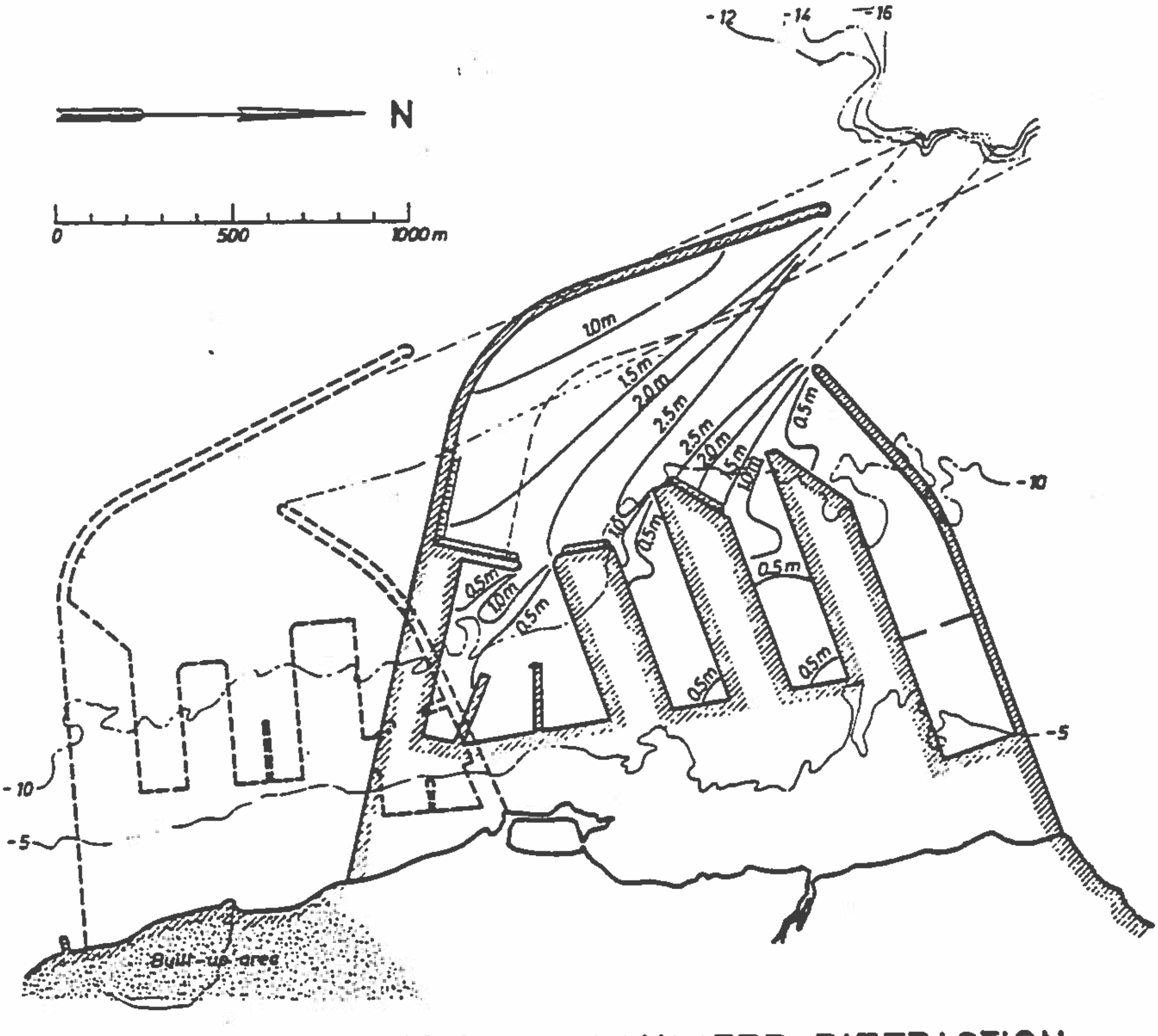
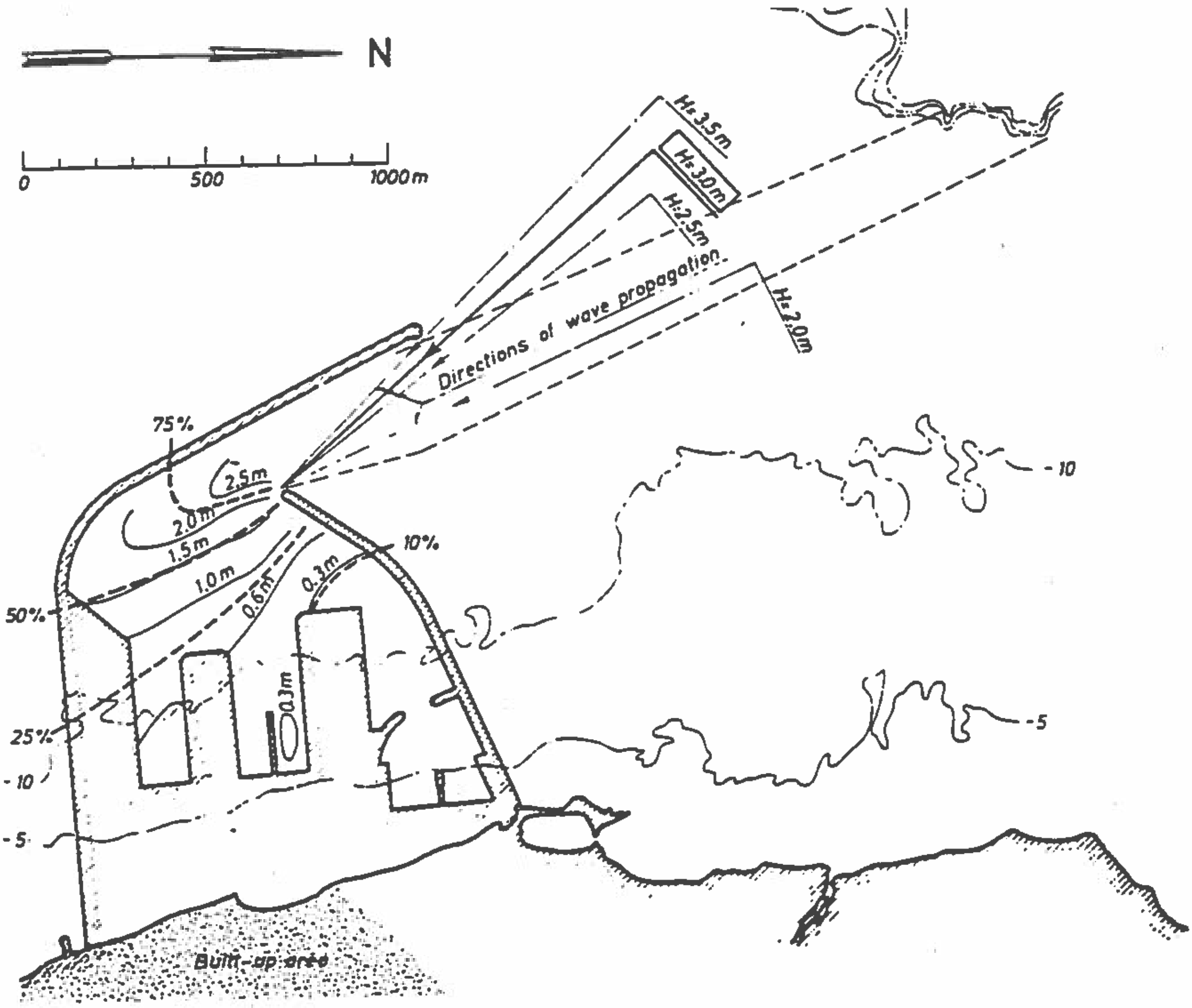


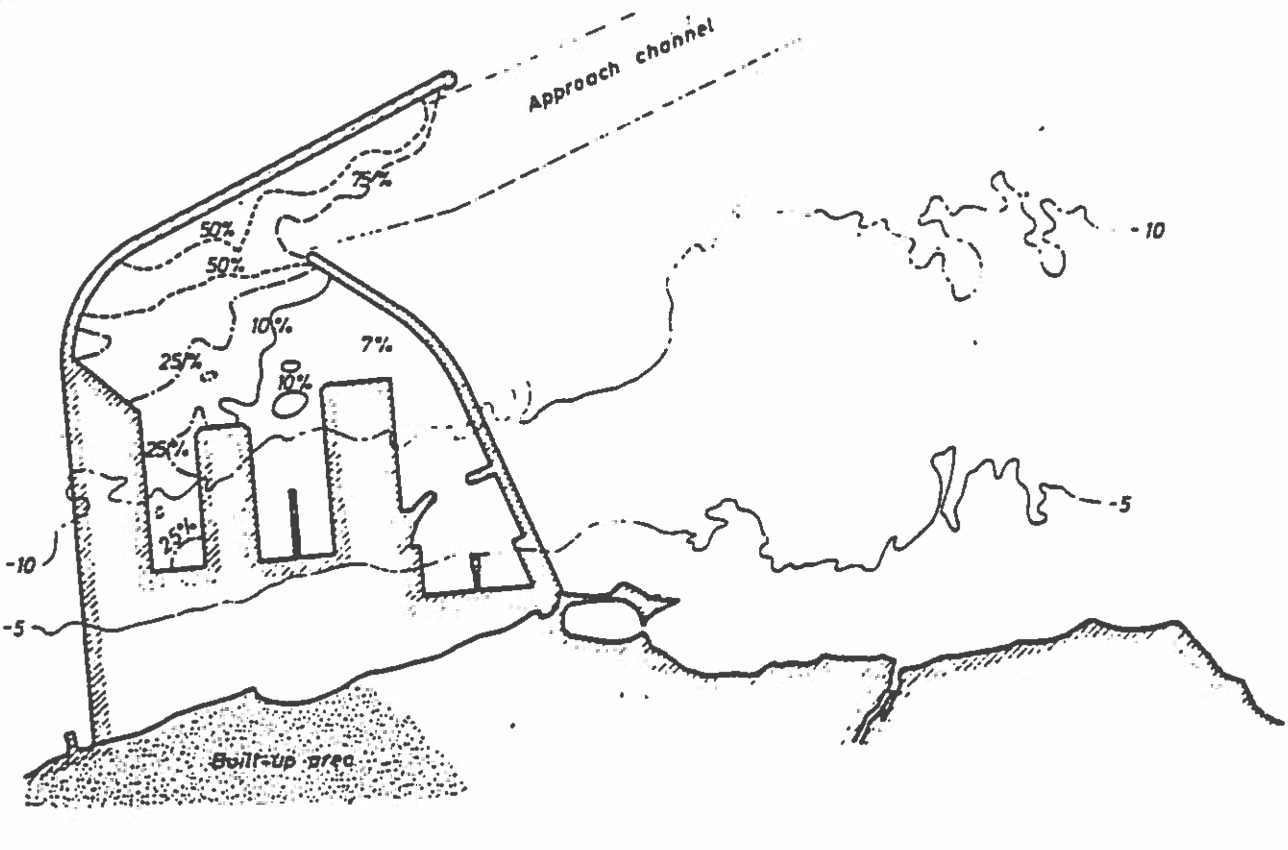
FIG. 4. MODIFIED LAYOUT – MODEL TEST RESULTS



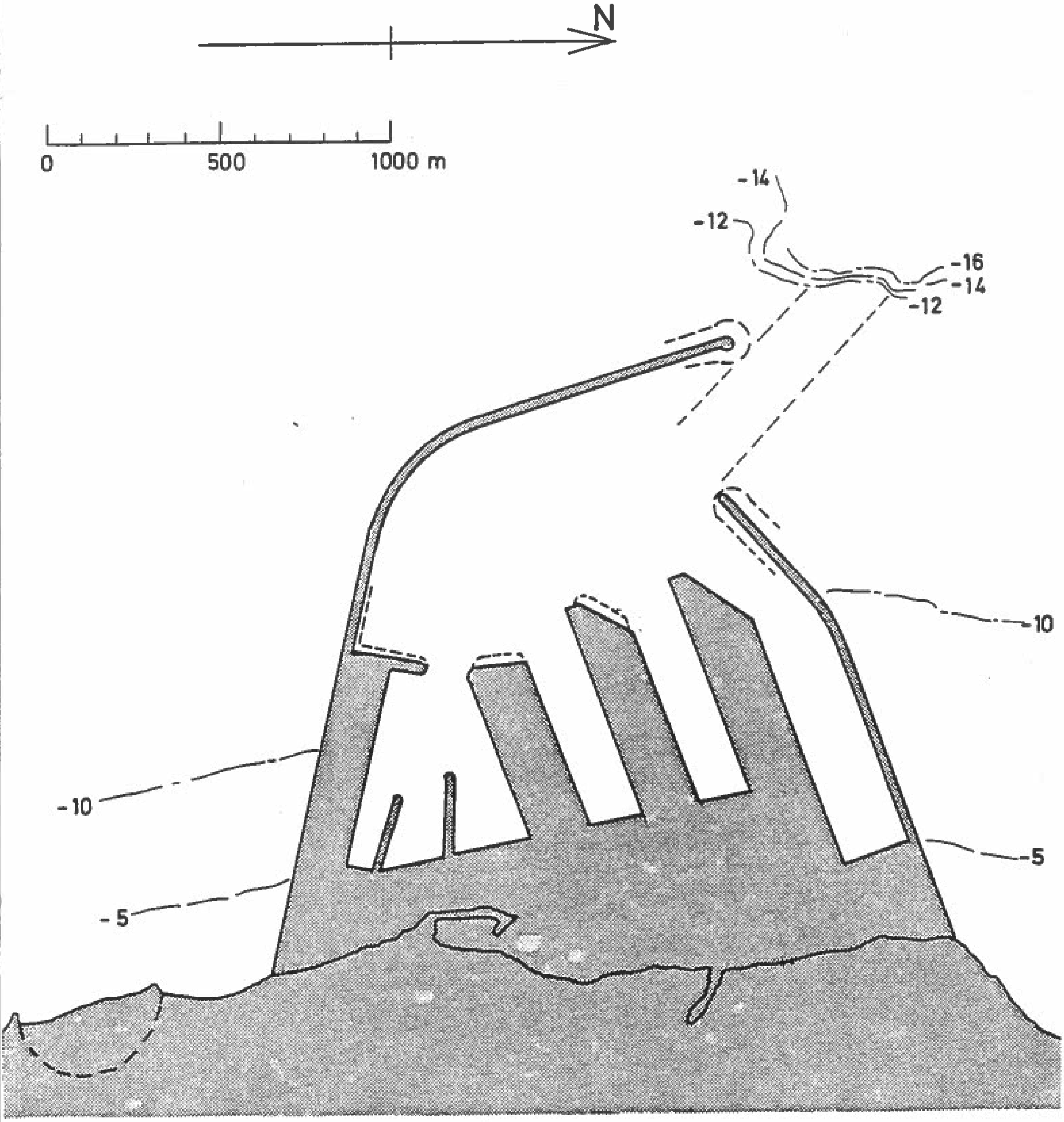
** FIG. 5. NEW LOCATION – CALCULATED DIFFRACTION



** FIG. 6. CALCULATED DIFFRACTION

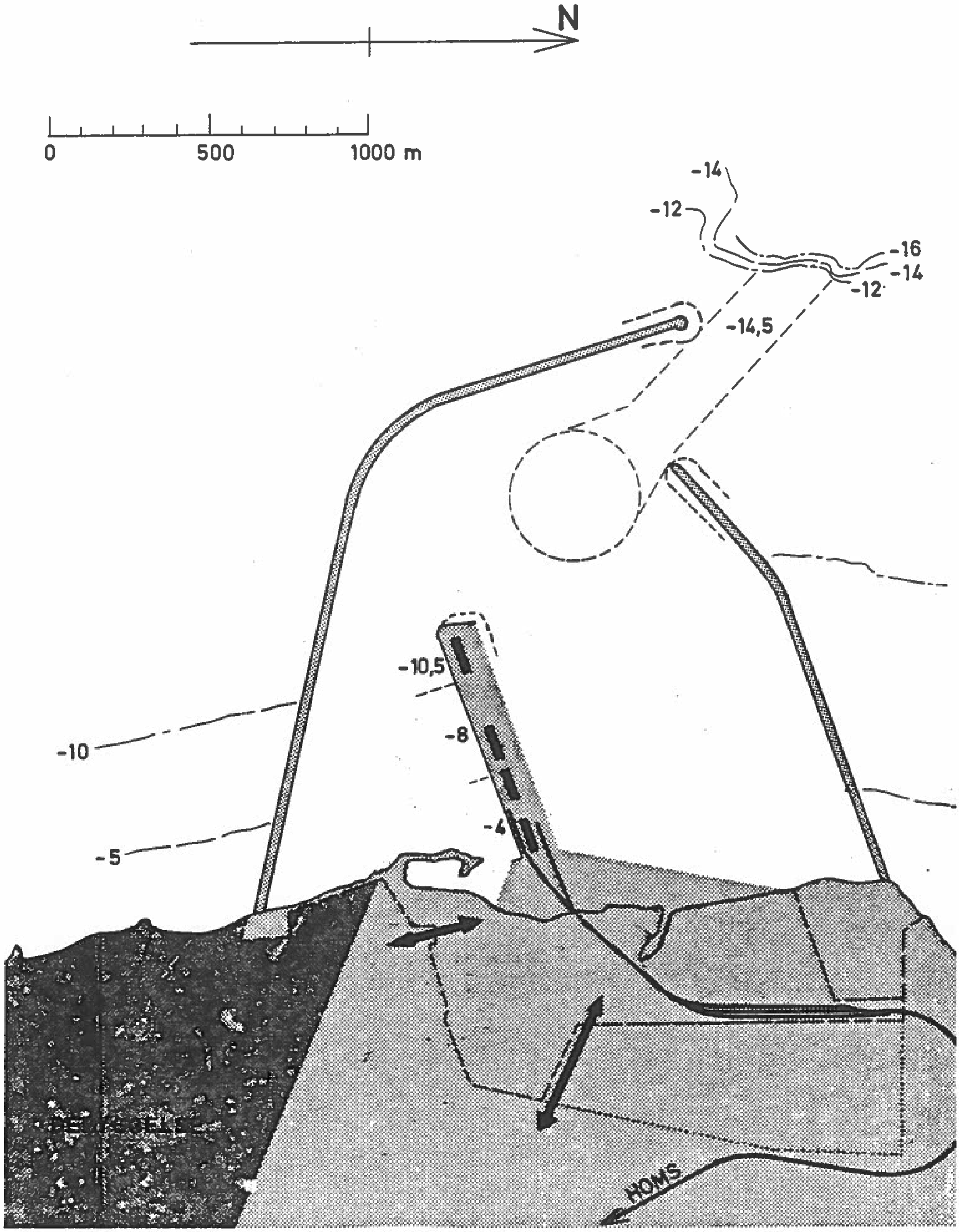


** FIG. 7. MODEL TEST RESULTS



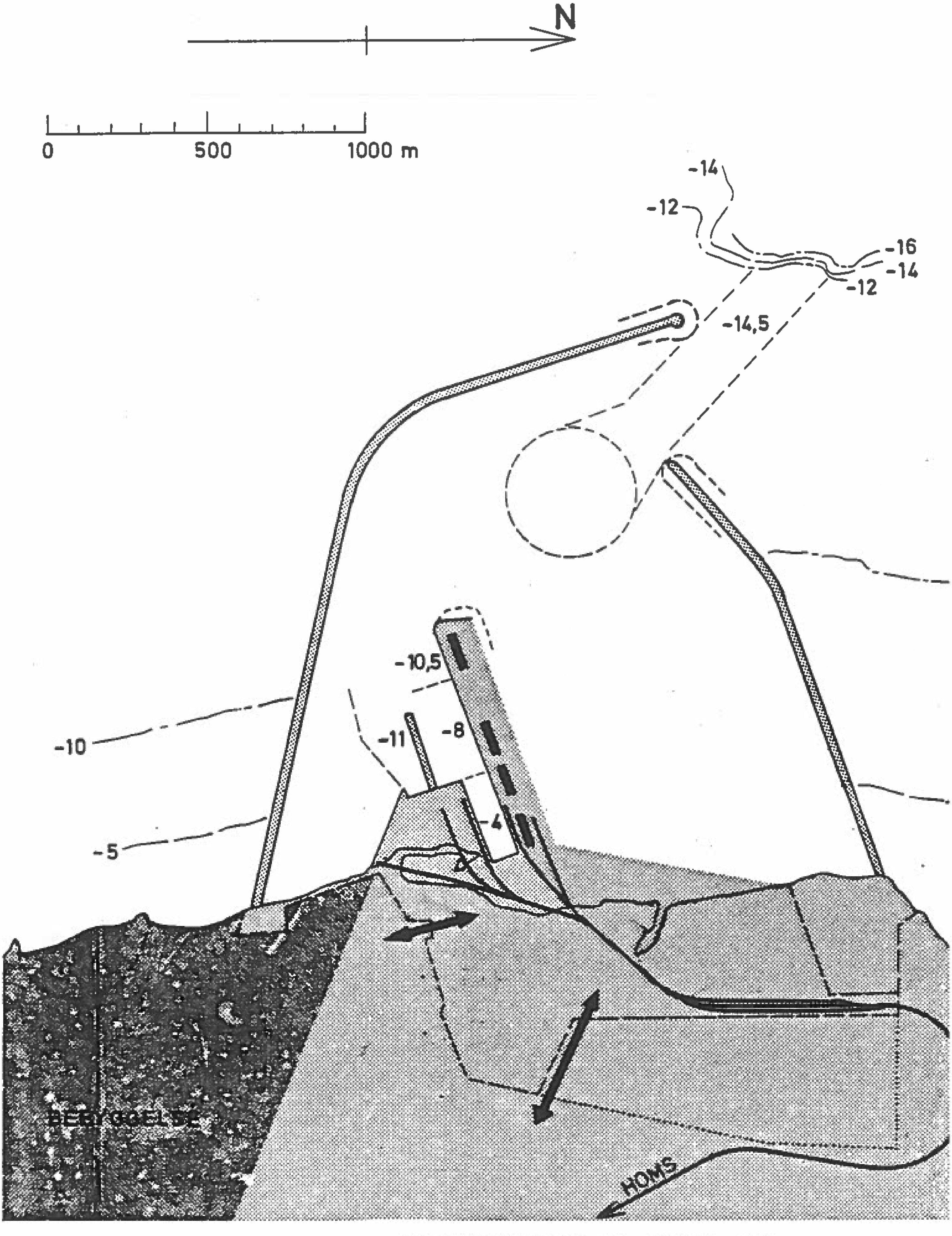
PLANLØSNING, 1960

FIG. 8



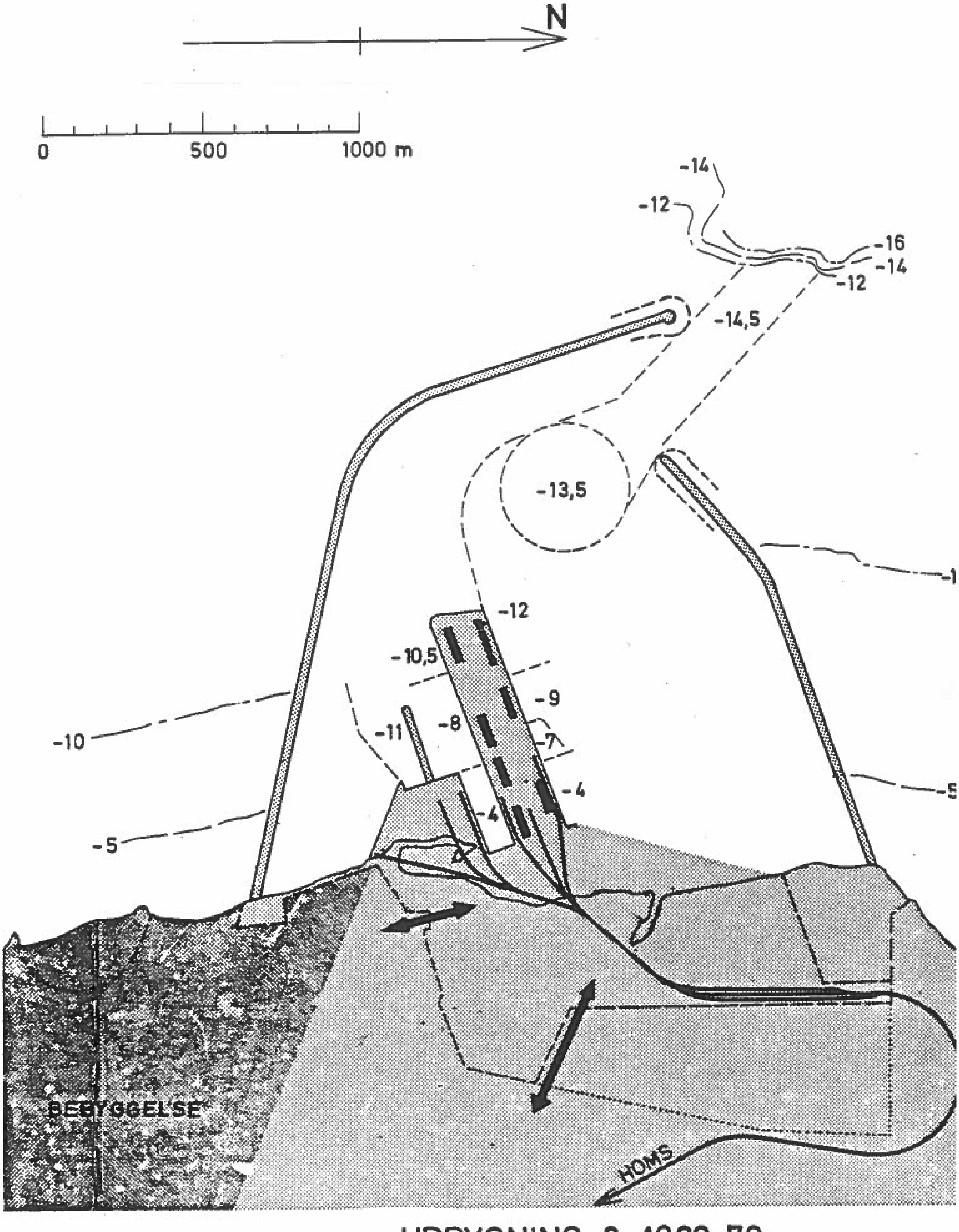
UDBYGNING 1, 1964

FIG. 9



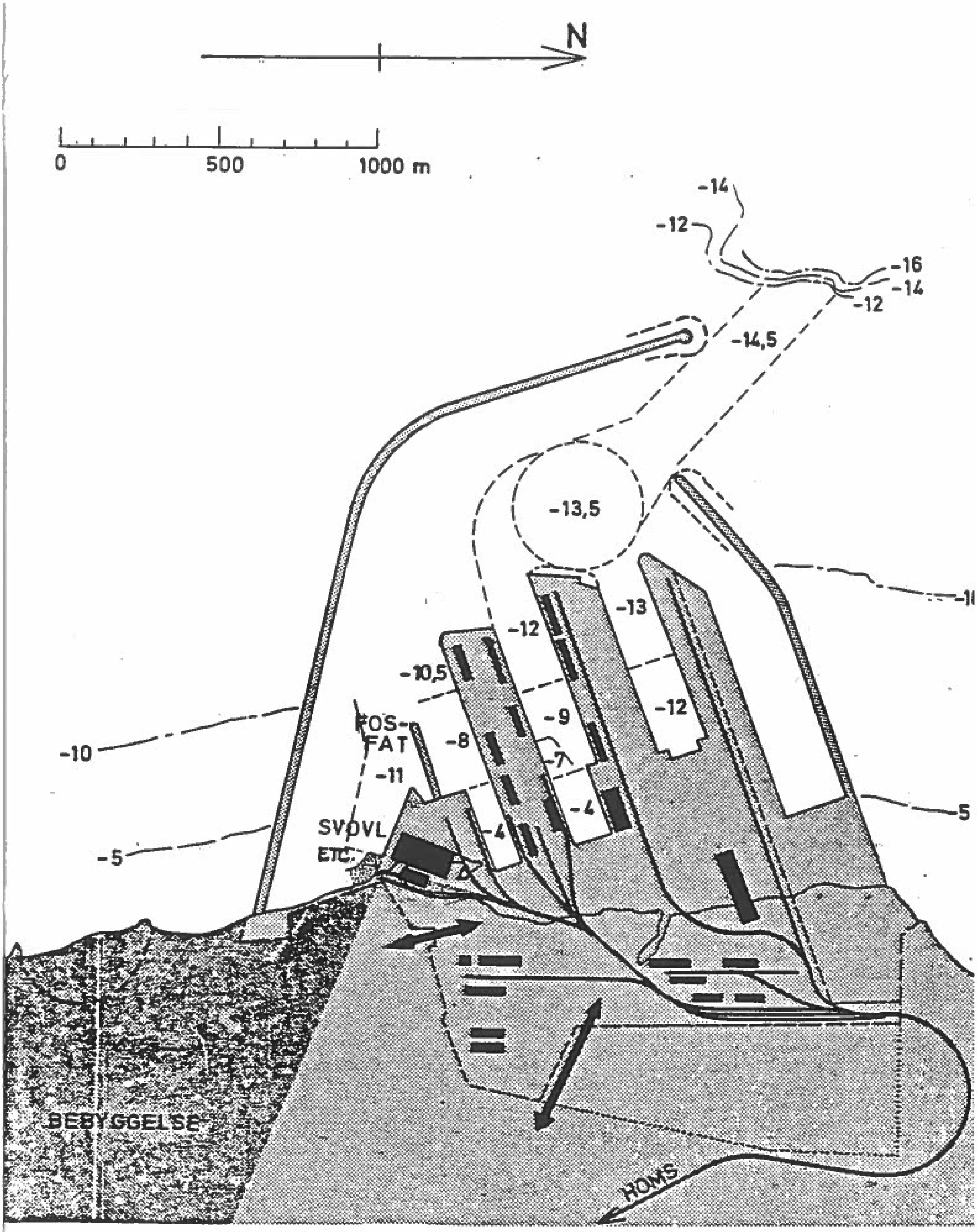
UDBYGNING 2, 1968-69

FIG. 10



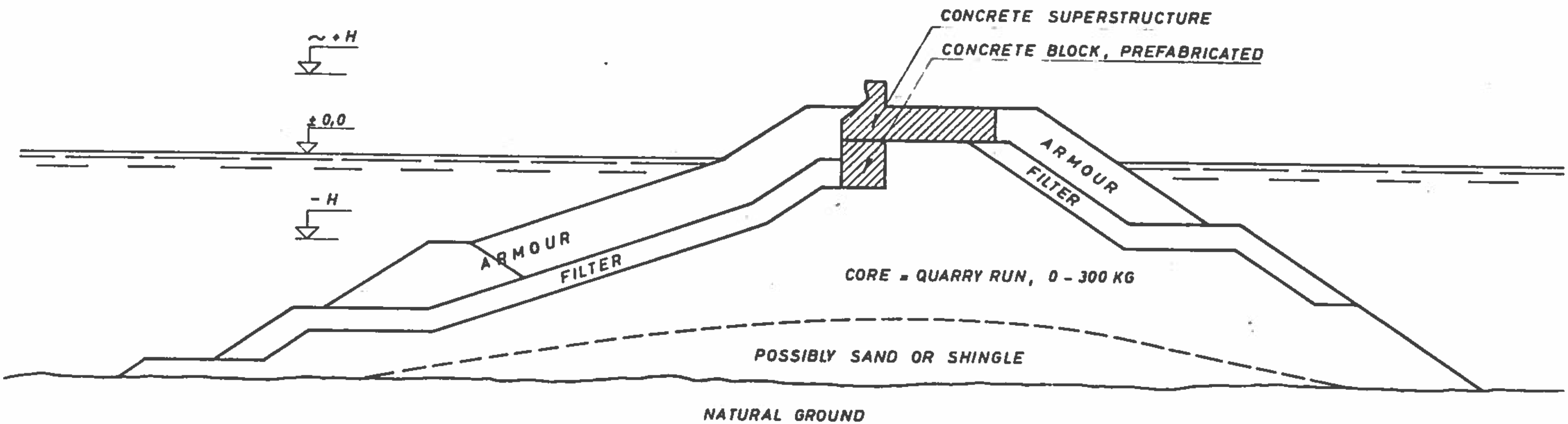
UDBYGNING 3, 1969-70

FIG. 11



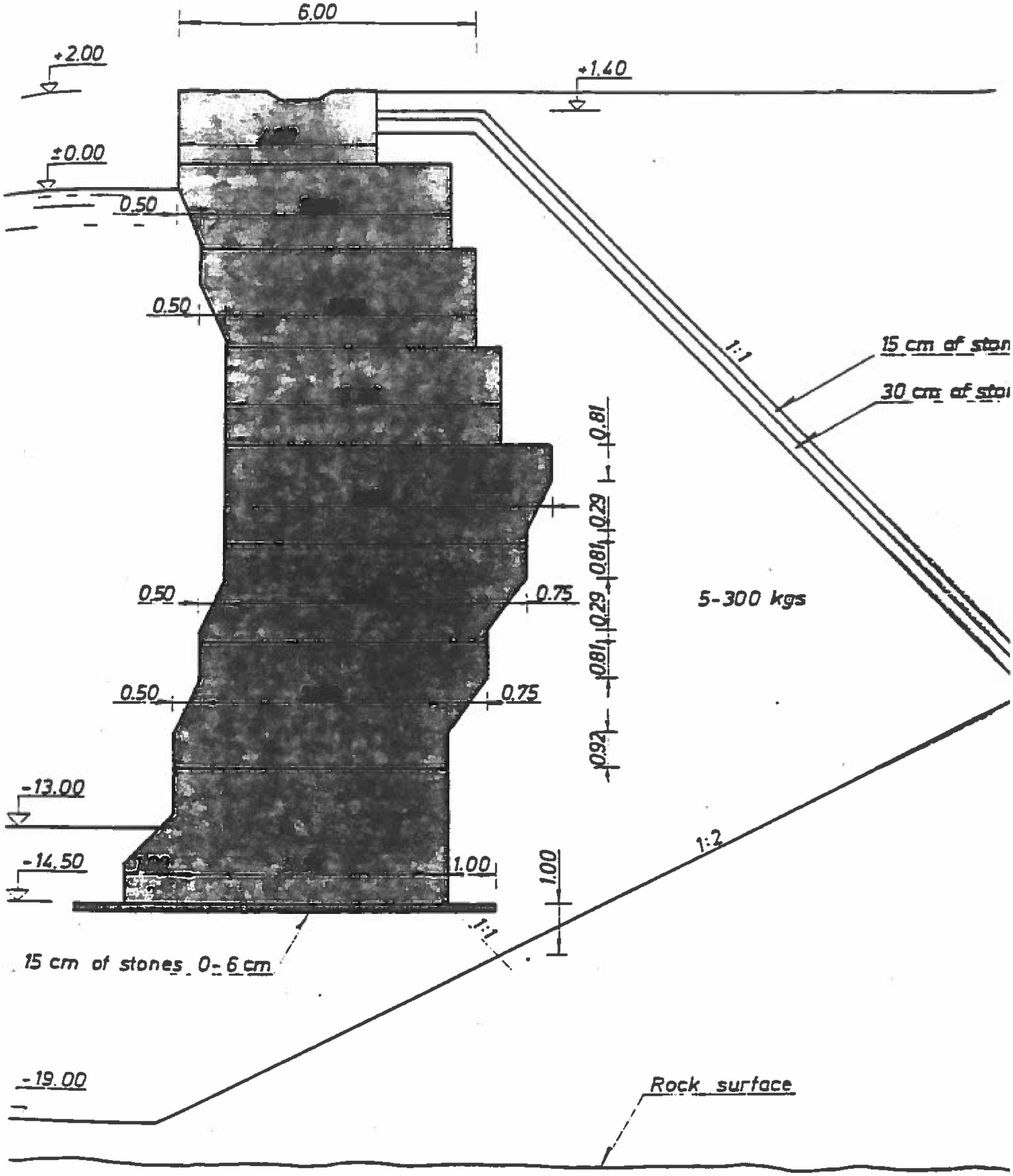
UDBYGNING 4, 1984

FIG. 12



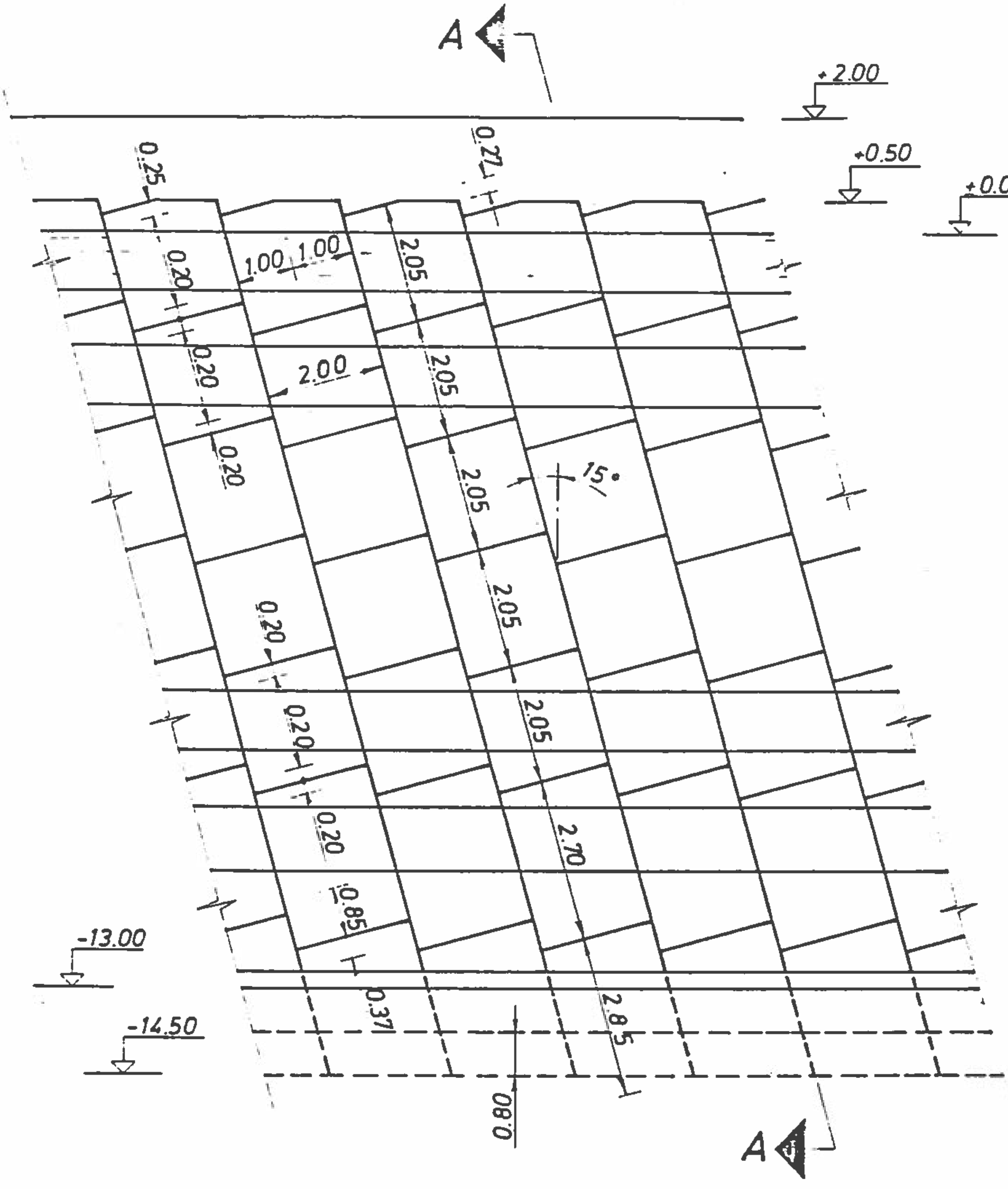
RUBBLE MOUND BREAKWATER.

FIG. 13



6.00
+2.00
+1.40
±0.00
0.50
0.50
1:1
15 cm of ston
30 cm of sto
0.81
0.29
0.81
0.29
0.81
0.92
0.50
0.75
5-300 kgs
0.50
0.75
-13.00
-14.50
1.00
1.00
1:2
1:1
15 cm of stones 0-6 cm
-19.00
Rock surface

FIG. 14



ELEVATION OF NORTH SIDE PIER B

1:100

FIG. 15

PUERTO DE CARBONERAS (SPAIN)

ved

civ.ing. Aksel Smidt-Petersen
(Christiani & Nielsen A/S)

DVS Seminar 25.10.83

Nye havne

Emnet for dette seminar er kriterier for valg af byggested, udformning og konstruktionstyper for nye havne. I det følgende eksempel, en havn for import af kul i Carboneras i Spanien, var beliggenheden fastlagt således at kun de to sidste parametre, udformningen og konstruktionstypen var frie parametre.

Carboneras er en lille fiskerby i provinsen Almería på Spaniens middelhavskyst, Fig. 1.

Man har her planlagt at bygge et kulfyret kraftværk mellem byen Carboneras og en eksisterende havn for udskibning af cement, Fig. 2. Cementhavnen kan tage skibe op til 40.000 DWT, mens kultrafikken går på langt større skibe. Af grunde som vi ikke er blevet indviet i, ønskede man ikke at udvide den eksisterende havn, men ville bygge en ny havn.

Bygherren er PUCARSA, et aktieselskab indenfor ENDESA, det spanske ELSAM eller ELKRAFT, oprettet alene med det formål at bygge og drive havnen. Man regnede med en havn, som i første udbygning skulle betjene et kraftværk med 2 enheder à 550 MW og en import på 3 mill. tons kul, som ville ankomme på massegodsskibe på 60.000 - 150.000 DWT.

I maj 1981 da C&N kom ind i billedet som rådgivende, var der allerede udskrevet en licitation baseret på et udbudsprojekt udarbejdet af det spanske rådgivende firma Carmoa. Det bestod af en temmelig løst skitseret planløsning, Fig. 3, og nogle få typiske tværsnit i dækmolen, se Fig. 4. Udbudsprojektet var, som man ser, en vinkelformet dækmole med en kaj orienteret nord-syd. Dækmolen er en stenkastningsmole med bølgeskærm af beton op til +13,5 m. Det forventedes, at de indbudte entreprenører eller grupper af entreprenører selv udarbejdede tilbudsprojekter som også kunne være alternativer til det udbudte.

I erkendelse af, at det ved denne udbudsform kunne være vanskeligt at foretage en fornuftig sammenligning af de indkomne tilbud valgte bygherren at lade C&N udarbejde et mere detaljeret udbudsprojekt som sammen med specifikationer og konditioner blev tilstillet de bydende med opfordring til også at give tilbud på det.

Foruden at lave et mere detaljeret udbudsmateriale havde vi naturligvis stillet bygherren i udsigt, at vi kunne lave et mere økonomisk projekt end det udbudte, hvilket vi så gik i gang med. Vi havde til rådighed ca. 2 måneder, således at der kun var tid til skrivebordsarbejde og ingen af vore påstande kunne nå at blive verificeret ved forsøg.

For at udføre denne opgave søgte vi og fik hjælp hos DHI og havde også af formelle grunde assistance fra det spanske rådgivende firma INITEC.

Idet vi går tilbage til Carmoa's udbudsprojekt, kan vi lige nævne de elementer af projektet, som skulle analyseres og detaljeres og hvor der var mulighed for forbedringer. Carmoa, Fig. 3, Fig. 4.

Havnens beliggenhed lå fast, der var ekspropriation i gang, man var således ikke interesseret i at undersøge en alternativ beliggenhed lige syd for Carboneras, som ellers nok kunne se ud til at byde på fordele.

Derefter kunne vi arbejde med følgende elementer:

a) Havnebassinets størrelse og form
b) Beliggenhed og orientering af dækmole
c) Dækmolens tværsnit
d) Udformning af kajer etc.

Før vi omtaler disse mere detaljeret skal de ydre betingelser omtales lidt nærmere:

Den nye havn ligger på en kyst stort set orienteret nord-syd, Fig. 2, og er derfor udsat for vind og bølger fra retningerne NØ til SØ, her er det naturligvis især NØ og Ø, der tæller, idet det fri stræk fra SØ er ubetydeligt.

DHI analyserede de eksisterende observationer, som var modtaget fra bygherren og korrelerede dem med deres egne erfaringer fra området og kom frem til en såkaldt 100 års design-bølge på H_s = 7,1 m. DHI havde kort før analyseret den storm, der ødelagde bølgebryderen i Arzew i Algeriet, Fig. 5; det ses, at Carboneras ligger i et udsat hjørne af Middelhavet. Fig. 6 viser bølgehyppigheden for retningerne NØ, Ø og SØ, 60°, 90° og 135°. Det ses, at NØ er den farligste retning. Der er intet tidevand.

Havnebassinets størrelse og form

Her var Carmoa gået ud fra den berømte manøvreellipse, se Fig. 3, som man finder i PIANC publikationer og som så vidt jeg har forstået, egentlig er udledt for noget helt andet, nemlig retningsændringer ved sejlads i en sejlrende. Den løsning, som vi kom frem til, Fig. 7, var baseret på en analyse af de tillægnings- og fralægningsmanøvrer, som vi regnede med ville forekomme i havnen, Fig. 8. Vi regnede med, at skibene altid gik ind og fortøjede med forstavnen ind i havnen og forlod havnen ved en ud-af-garagen manøvre, som altid foregik med skibet i ballast, d.v.s. man kunne reducere dybden i den indre del af svajebassinet til -12 m. Derudover regnede vi med dybden -20 m svarende til 150.000 og -8 m ved oliekajen, hvor 5000 DWT tankere skal kunne lægge til.

Analyserne af skibsmanøvrerne blev foretaget i samarbejde med lods Jan E. Roll, Stigsnæs.

Dækmolens beliggenhed og orientering

Ved at arrangere svajebassinet som vist opnåede vi at kunne "klemme" dækmolen ind på mindre vanddybder, hvilket skulle give besparelser, sammenlign Fig. 3 Carmoa og Fig. 7 C&N.

Dækmolens tværsnit - Fig. 9

Som sagt skulle dækmolen beregnes for bølgehøjder på op til H_s = 7,1 m, hvilket i forbindelse med den betydelige vanddybde, op til 35 m, førte til en ganske alvorlig konstruktion. Det anbefalede profil blev foreslået på grundlag af følgende overvejelser:

1. De tilgængelige stenbrud kunne højst levere blokke på 8 tons, altså måtte dæklaget udføres af betonelementer under en eller anden form.

2. Med erfaringerne fra den senere tids ødelæggelser af dækmoler med dolos'er ville man ikke anbefale disse, selv om de har en god stabilitetsfaktor.

3. Man antog, at profilet ville blive så bredt, at man ikke kunne nå dets yderste punkter fra en landbaseret kran, men måtte sænke dæklagets

elementer fra en pram i bølgebevægelse med deraf følgende risiko for brud i tilfælde af tetrapoder eller tilsvarende "multipods". Vi anbefalede derfor kubiske betonblokke som de eneste, der kunne modstå disse påvirkninger.

4. Det af Carmoa foreslåede profil med bølgeskærm, som jo er almindeligt i Middelhavet, blev også overvejet. Det har den fordel, at man kan komme ud at reparere mindre skader fra land. Det er så til gengæld katastrofalt dårligt i tilfælde af større skader, som man så det ved ødelæggelsen af Arzew-bølgebryderen i Algēriet.

5. Vi endte med at foreslå profilet Fig. 9. Da man her er afskåret fra at foretage reparationer fra land er profilet projekteret med et meget lavt skadeskriterium. 0,5-2%, således at skader skulle være meget sjældne.

6. Ved at lægge kronen så højt som +13,5 m undgik man overskyl, således at bagsidens dæklag kunne udføres af relativt små blokke.

Konstruktioner inden i havnen

Lossekajen som bærer lossekranen, en tingest på ca. 1000 tons, kunne ifølge udbudsprojektet udføres som pælekonstruktion, på sænkekasser eller på spunsvægsceller uden at der var givet nogen detaljer af hvordan disse konstruktioner skulle udføres.

Da der var angivet en meget kort byggetid for projektet, valgte vi at foreslå en pæleløsning ud fra den betragtning, at en sådan kunne udføres samtidig med dækmolen. Hvis man valgte sænkekasser skønnede vi, at der ville gå for lang tid, før der var så meget læ i havnen, at kasserne kunne sættes med sikkerhed. Vi havde erfaring for, at pælekajer kunne udføres i åbent hav, når dækkoten var over højeste bølge i konstruktionsfasen, Fig. 9. Dækkoten blev sat til +7,0 m, hvilket jo ville være rigeligt for den færdige konstruktion, men dog acceptabelt. Den samme konstruktion foresloges for adgangsbroen, som bærer en 6 m bred vejbane og kultransportøren, som er en båndtransportør. Som det ses på planen Fig. 7 forløber adgangsbroen frit inde i bassinet, uafhængigt af dækmolen.

Også fortøjningsdalberne og oliekajen foresloges udført som pælekonstruktion.

Bugserbådskajen, Fig. 7, som har 6,5 m vanddybde, foresloges bygget af rektangulære præfabrikerede betonblokke.

Licitation

Ved licitationen kom der tilbud fra 4 grupper af spanske entreprenører. Tilbudene var meget vel præsenterede, der var lagt et stort arbejde i udarbejdelsen af alternativer, en enkelt havde udarbejdet 7 alternativer ud over C&N's og Carmoa's projekter. Bygherren valgte at skrive kontrakt med en gruppe af entreprenører, AGREDA bestående af

Dragados y Construcciones
Entrecanales y Távora
Auxini

om at udføre et projekt, der var sammensat af den af C&N foreslåede dækmole og en lossekaj bestående af rektangulære sænkekasser. I overensstemmelse med tilbudet var detailprojekteringen inkluderet i kontrakten.

Revideret projekt

Efter at aftale var indgået med entreprenøren bestemte bygherren sig for at reducere projektet, idet man nu kun regnede med en kulimport på 1,8 mill. tons pr. år svarende til 1 enhed på 550 MW. C&N og DHI fik i febr. 82 til opgave at udarbejde det reviderede forprojekt, som nu var baseret på:

(a) Dækmole, plan og tværsnit som foreslået af C&N
(b) 1 lossekaj for 70.000 DWT massegods skibe med mulighed for udvidelse til 120.000 DWT. Kajen udføres af sænkekasser efter entreprenørens forslag
(c) Adgangsvej og -bro med kultransportør
(d) Kølevandsindtag for kraftværket

Det reviderede projekt omfattede revision af havnens udformning i overensstemmelse med de ændrede betingelser og ikke mindst modelforsøg til verifikation af alle dimensioner. Det skulle danne grundlag for entreprenørens detailprojektering. I sin endelige udformning kom det til at se således ud. Fig. 10.

Arbejdet på byggepladsen skulle starte straks, så vi måtte hurtigt vælge et startpunkt for dækmolen, således at entreprenøren kunne komme igang. Startpunktet blev valgt således at man, hvis det skulle blive nødvendigt, kunne forlænge dækmolen med 100 m og stadig bevare en indsejlingsåbning på ca. 270 m, som skulle være tilstrækkelig for en 120.000 DWT.

Laboratorieforsøg - Fig. 12

DHI lagde ud med 3-dimensionale forsøg til bestemmelse af de kystnære sektioner af dækmolen. På grund af det skrå bølgeindfald kunne disse sektioner ikke prøves i bølgerende. Det var den første del af et omfattende program, der så således ud:

Dækmole: kystnære profiler, 3 dim. model
- : hovedsektioner, bølgerende
- : molehoved, 3 dim. model
Uroforsøg: skibsbevægelser, 3 dim. model
- : fenderkræfter, 3 dim. model
- : fortøjningskræfter, 3 dim. model
- : bølgebryder i bassin, 3 dim. model

Forsøgene udførtes med forskellige typer af uregelmæssige bølger registrerede under storme i Middelhavet, incl. en måling af en storm ved Carboneras i foråret 82.

På basis af forsøgene kunne vi bestemme de endelige profiler for dækmolen, her skal kun beskrives et enkelt af dem, Fig. 11. Forsøgene viste, at med den valgte geometri: hældning 1:2, topkote +10,0 m var 58 t blokke på forsiden tilstrækkelige. Mellem +10,0 m og +13,5 m kunne vi reducere fra 18 t blokke til type I, d.v.s. 2-4 t sten, som viste sig at være de største man kunne fremskaffe. Banketten ved foden af dæklaget viste sig at være udsat, det ser ud som om den vandrette overside af banketten er "unaturlig", i forsøgene blev den høvlet af, så den fik en skrå overside. vistnok på grund af den tilbageløbende bølge. For at sikre banketten indførte vi endnu en banket af type I sten foran den banket, der støtter hoveddæklaget. Molehovedet blev prøvet i 3 dim. model, det viste sig, at de 80 t blokke var tilstrækkelige.

Uroforsøgene som udførtes i en 30 x 33 m model i Hørsholm var meget omfattende, Fig. 12. For de 2 skibstyper 70.000 DWT og 120.000 DWT med forskellige former for fortøjning (med eller uden nylonstropper) med forskellige fender-typer bestemtes bevægelser og kræfter for bølger fra 3 forskellige retninger. Af de mange resultater kan nævnes, at de største kræfter og bevægelser opstår som følge af lang-periodiske bølger, der anslår havnebassinets egenfrekvens og vi måtte konstatere, at der var vejrsituationer, hvor skibsbevægelserne blev så store, at losning var umulig og at der derudover var ekstreme vejrsituationer, hvor bevægelser og kræfter blev så store, at skibet ikke kunne forblive ved kaj. De valgte grænser for skibets bevægelser var: 1,5 m, 1,0 m og 0,5 m for henholdsvis langsgående, tværgående og op/ned bevægelser, se Fig. 13. I skemaet Fig. 13 er også angivet hyppigheden for disse grænsers overskridelser, hvilket altså er lig med "downtime" for losningen. Tallene svarer til 100% belægning på kajen, hvilket naturligvis ikke bør forekomme i praksis.

I samme skema er angivet hvor hyppigt den tilladelige fortøjningskraft overskrides, ca. 8 timer pr. år. Den valgte kraft svarer til 75% af brudstyrken af 2 stk. 4 tommer (omkreds) stålwire, og det er springene, som først når denne grænse, hvilket vil sige, at det er de langsgående påvirkninger, der er de farligste. Denne kraft, 66 t, optræder ved en bølgehøjde udenfor havnen på H_s = 3,5 m. Man kunne foreskrive at lægge nogle flere trosser ud og holde skibet på den måde. Imidlertid viser forsøgene, at trossekræfterne vokser mere end proportionalt med bølgehøjden så vi valgte at fortolke resultaterne derhen, at der kan forekomme situationer, hvor skibet ikke kan blive ved kaj, men må søge "tilflugt" til åben sø. Da dette kan forekomme med så kort varsel, at der ikke er tid til at hidkalde bugserassistance, må man anbefale, at skibet altid ligger fortøjet med forstavnen udad, hvilket betyder, at det må svaje ved ankomsten, altså i lastet tilstand. Dette medfører så, at hele svajebassinet må uddybes til fuld dybde, som nu på grund af de mindre skibe kun er -15 m. Fig. 10.

For at bestemme de nødvendige dimensioner af bassinet fik vi hos Skibsteknisk Laboratorium i Hjortekær udført en forsøgsrække på deres simuleringsanlæg, hvor de i en matematisk model kan simulere manøvrer med skibe udsat for de samme fysiske betingelser som i virkeligheden. Ved forsøgene ser man skibets bevægelse på en skærm, samtidig kan man optegne øjebliksbilleder med faste tidsmellemrum, således at hele forløbet er registreret; resultatet er ofte meget spændende figurer. Fig. 14. I dette tilfælde ses en manøvresituation, hvor man tager en 70.000 DWT til kaj udelukkende ved hjælp af spil placeret på land, d.v.s. på 2 fortøjningsdalber og på en dalbe på kysten.

Da der ikke er havn med bugserbåd i nærheden, foreslog vi bygherren at lade installere sådanne spil, han har dog endnu ikke accepteret ideen, bl.a. fordi han regner med uvilje hos skibsførerne. For øjeblikket ser det ud til at bygherren vil lade installere 1 spil, på en duc dalbe nær stranden. Med det kan man i en nødssituation få et skib fra kaj uden bugserbåde og dermed få det ud af havnen.

Tilbage står at nævne, at bygherren valgte entreprenørens sænkekasseløsning for lossekajen i stedet for pælekaj. Fig. 15.

Som et nyt krav var kommet, at vi skulle skaffe plads til et indtog for kraftværkets kølevand. Selv om det blev gemt af vejen i det fjerneste hjørne, viste DHI's analyser, at der var fare for at sand opstemmet af skibenes skruer ville kunne komme ind i indløbet, så vi måtte lægge en sekundær dækmole foran. Med den beliggenhed kulpladsen har, viste det sig at man kunne anbringe kultransportøren oven på den sekundære dækmole.

Havnen er under udførelse og forventes færdig maj 1984.

Man har udlagt dækmolen op til +3,5 og alle betonblokke på ydersiden. Dækmolen færdiggøres nu fra molehovedet mod land.

Kajens caissoner bliver støbt med en hastighed af 1 hver 10. dag.

A. Smith-Peh

44°
42°
40°
38°
36°
34°
SPAIN
BARCELONA
VALENCIA
ALICANTE
CARTAGENA
CARBONERAS
ALMERIA
MALAGA
GIBRALTAR
MELILLA
ORAN
ARZEW
ALGIERS
SKIKDA
TUNIS
100 0 100 200 300
4°
6°
8°
10°
0°
2°

FIG. 1

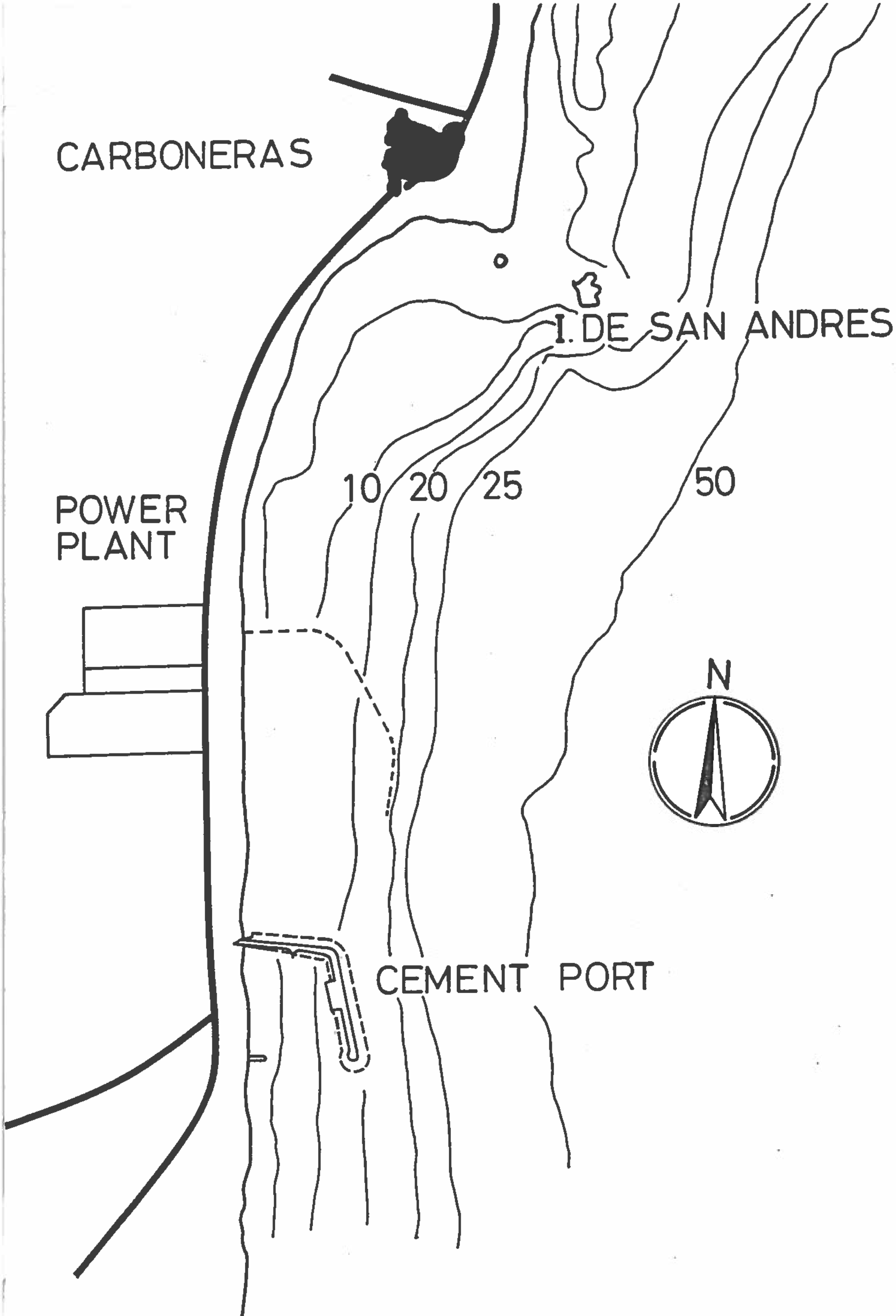
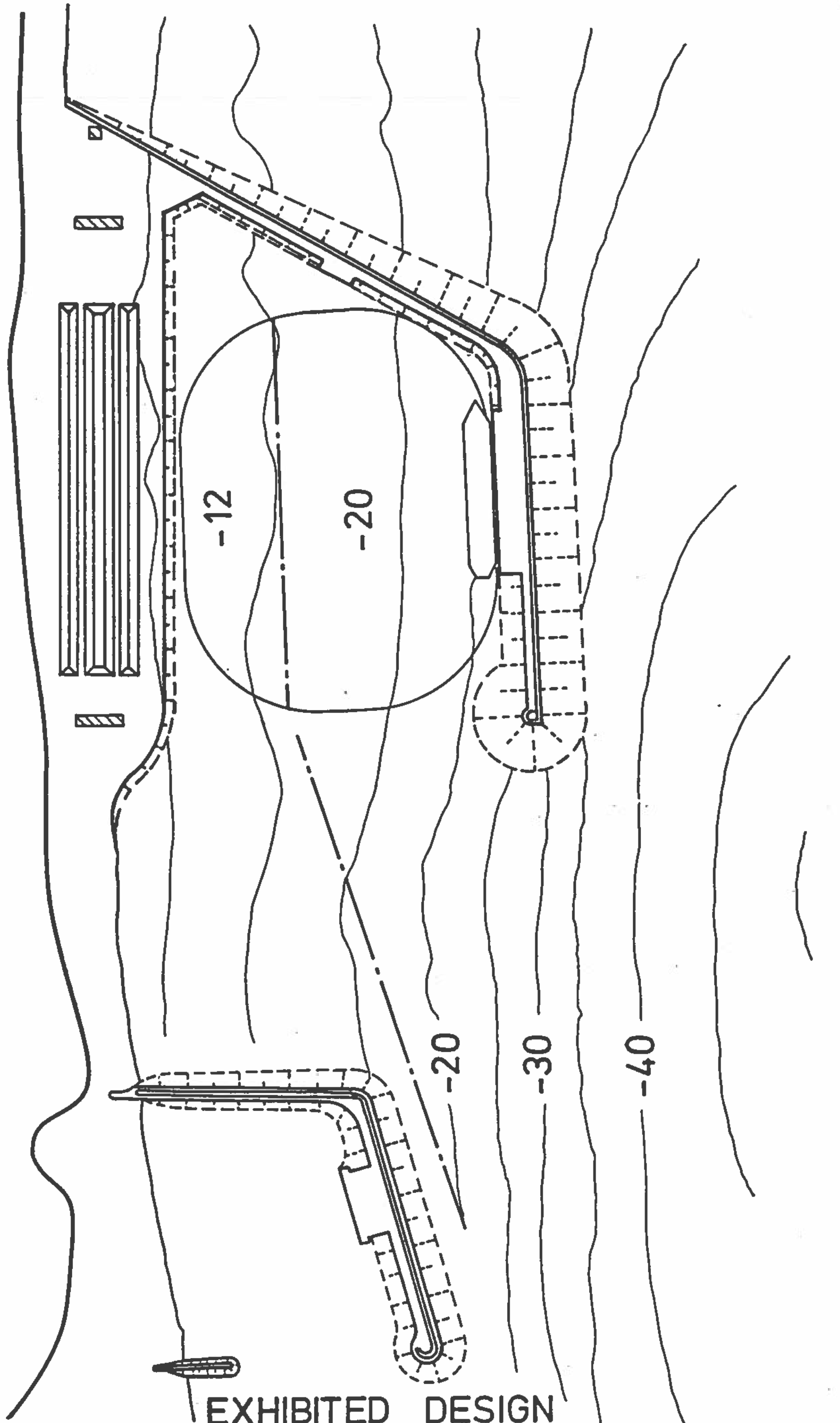

CARBONERAS
I. DE SAN ANDRES
POWER PLANT
10
20
25
50
N
CEMENT PORT

FIG. 2



EXHIBITED DESIGN

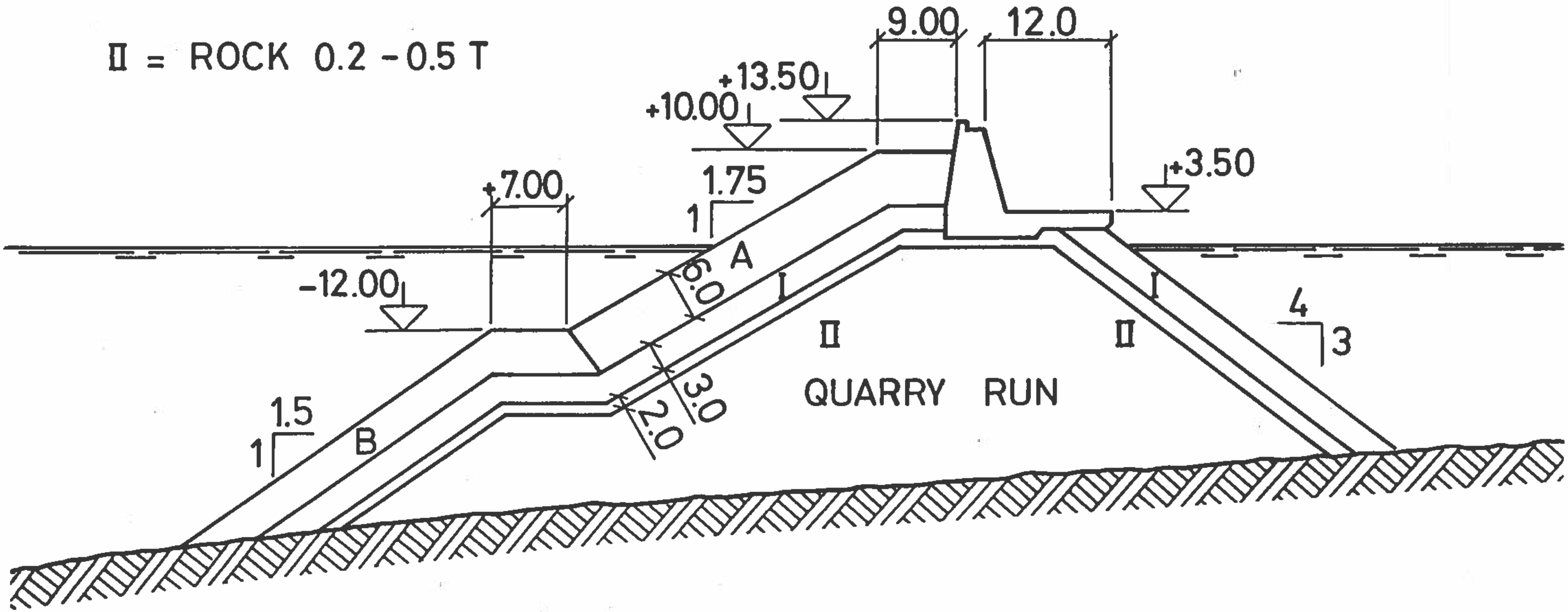
FIG. 3

A = BLOCK 60 T

B = BLOCK 25 T

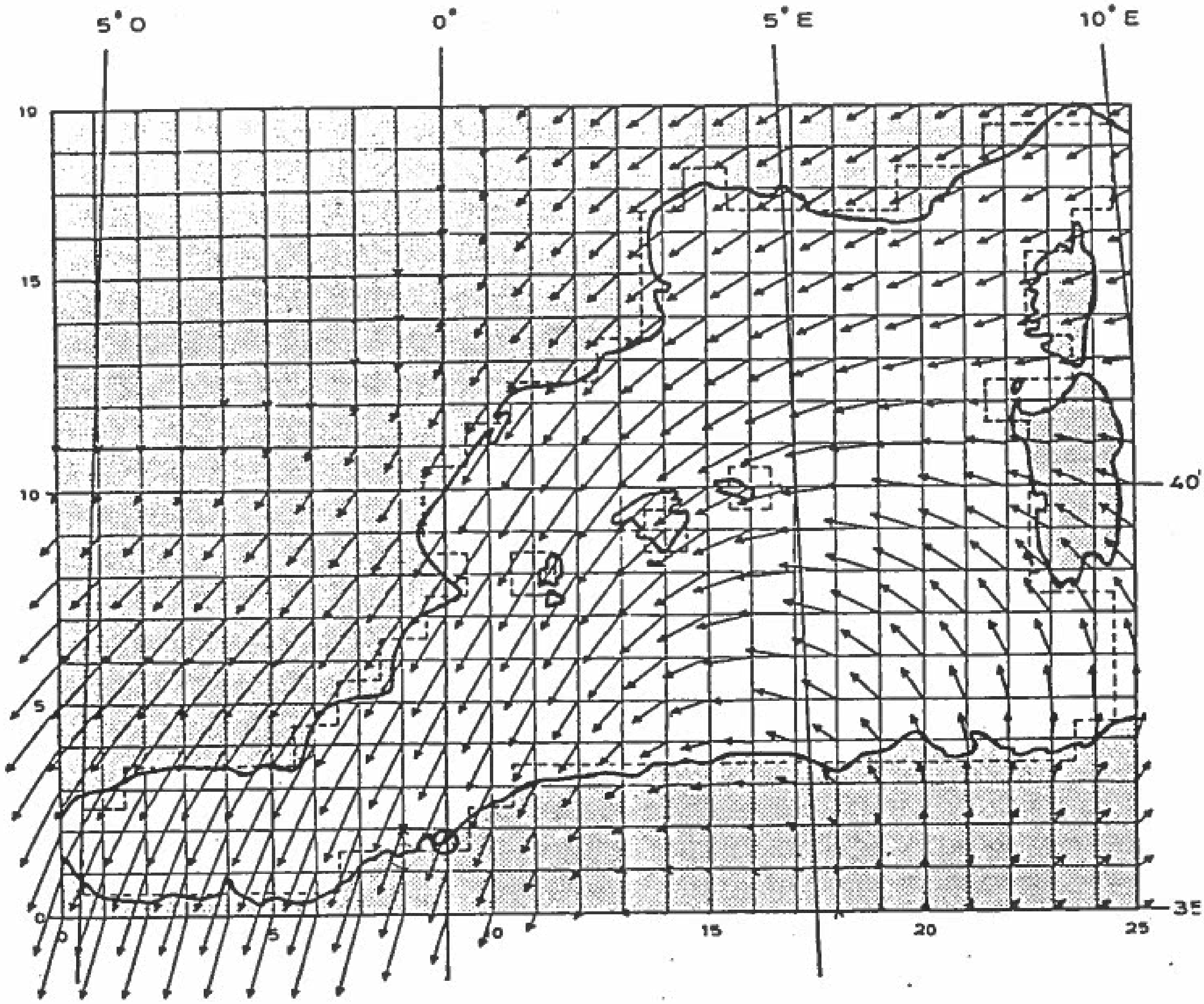
I = ROCK 2.5 - 4 T

II = ROCK 0.2 - 0.5 T



EXHIBITED DESIGN

FIG. 4



STORM DESTROYING ARZEW BREAKWATER
28-12-1980

FIG. 5

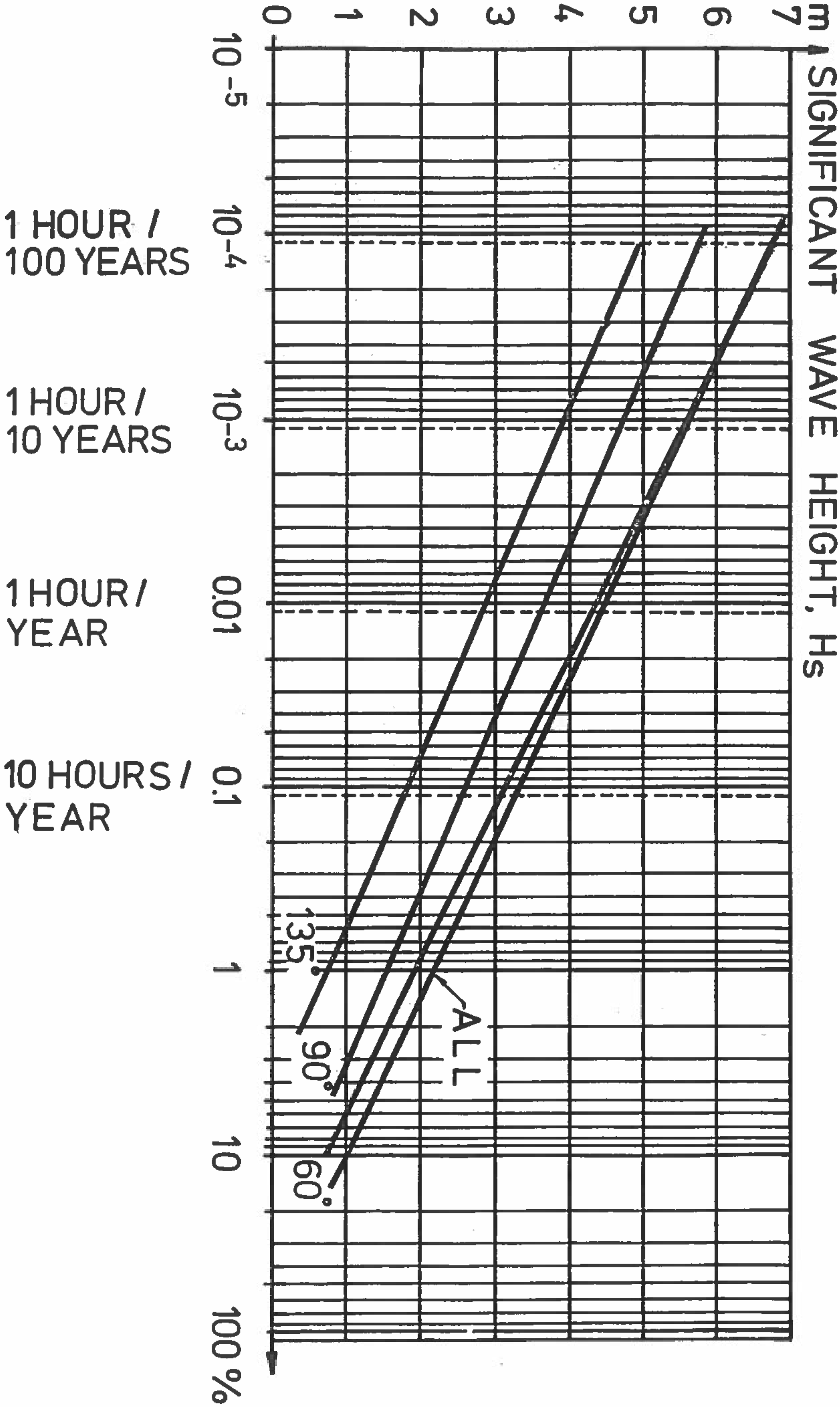
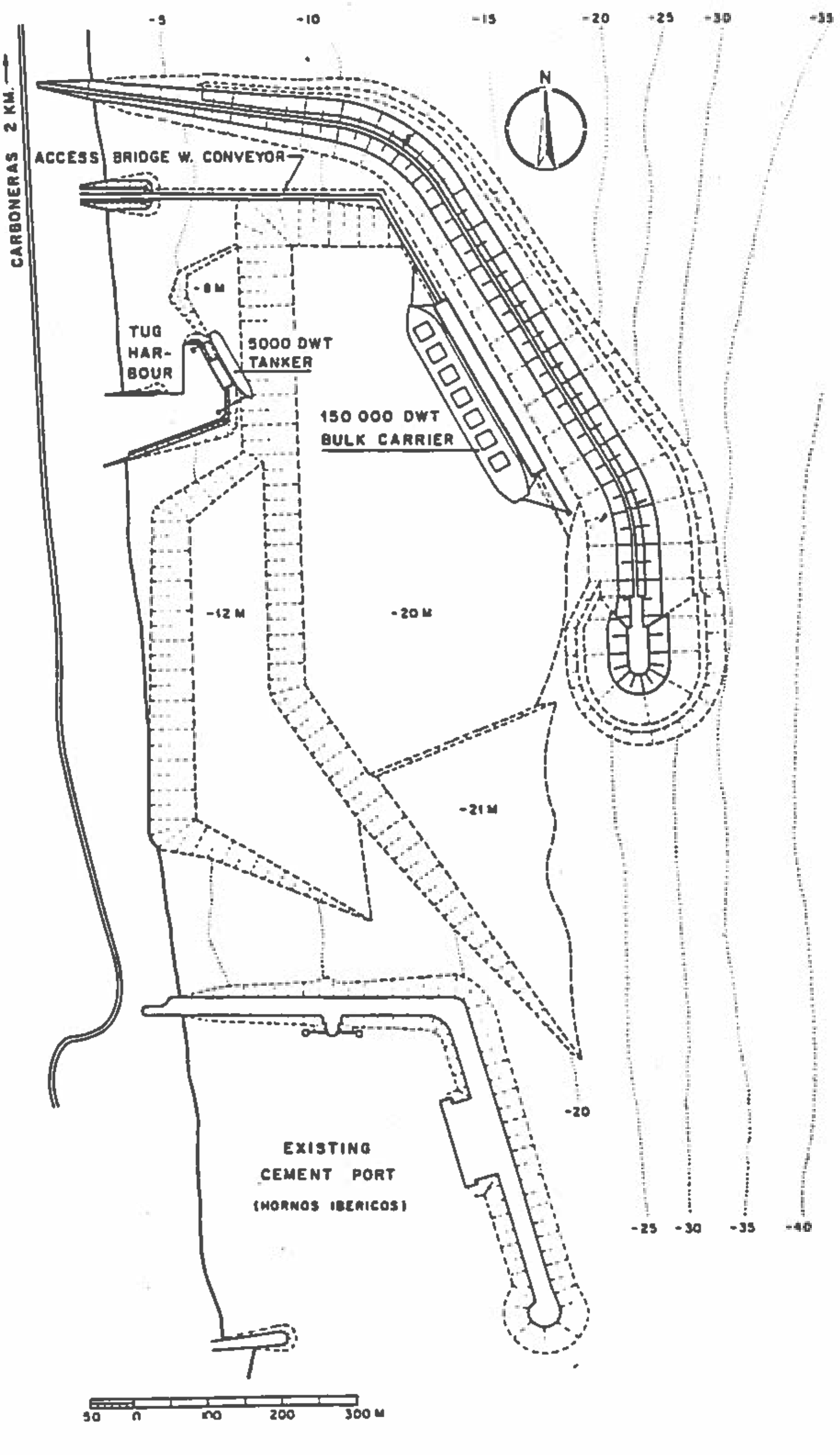
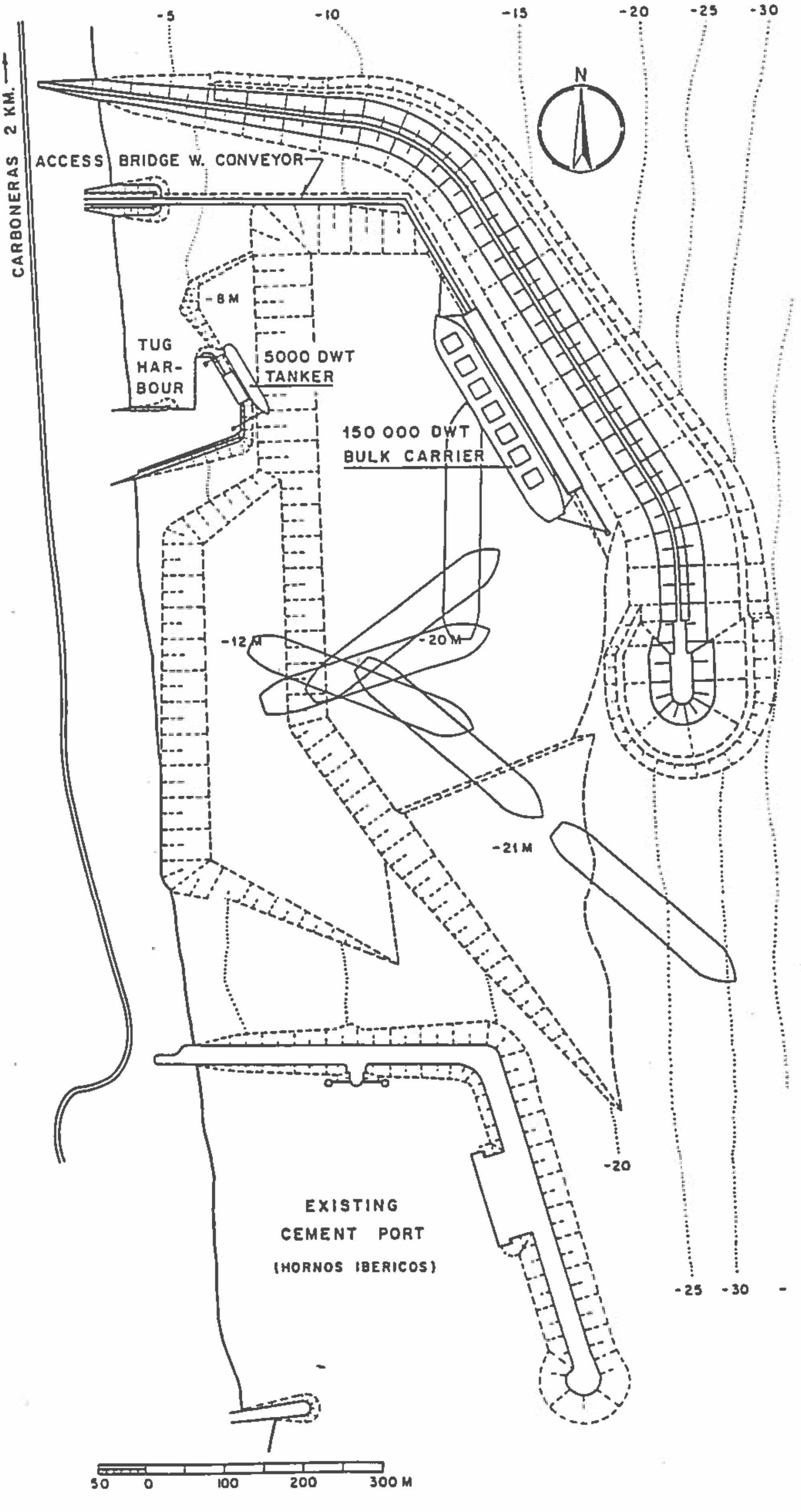

SIGNIFICANT WAVE HEIGHT, Hs
m
0
1
2
3
4
5
6
7
10−5
10−4
10−3
0.01
0.1
1
10
100%
1 HOUR / 100 YEARS
1 HOUR / 10 YEARS
1 HOUR / YEAR
10 HOURS / YEAR
135°
90°
60°
ALL

FIG. 6

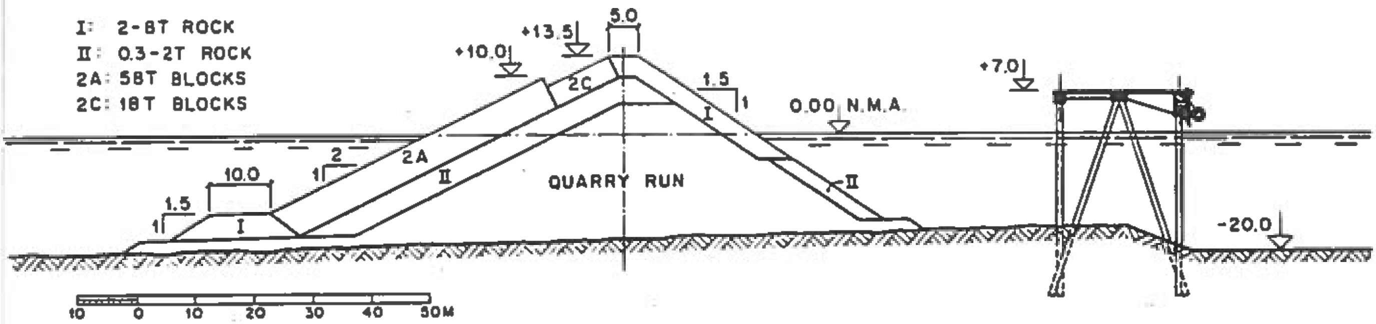


PRELIMINARY DESIGN, GENERAL LAYOUT FIG. 7



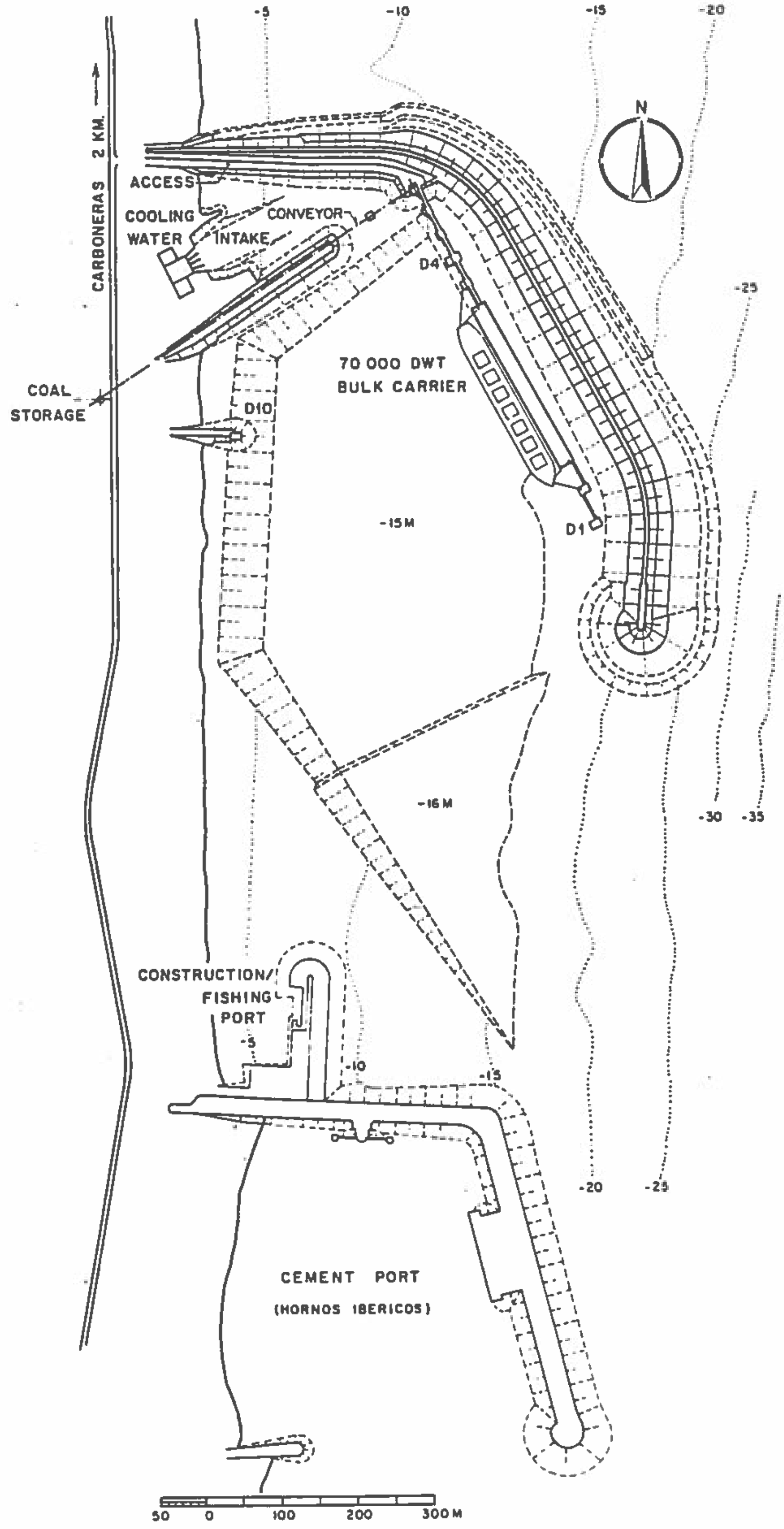
FRALÆGNINGSMANØVRE

FIG. 8



TENDER PROJECT

FIG. 9



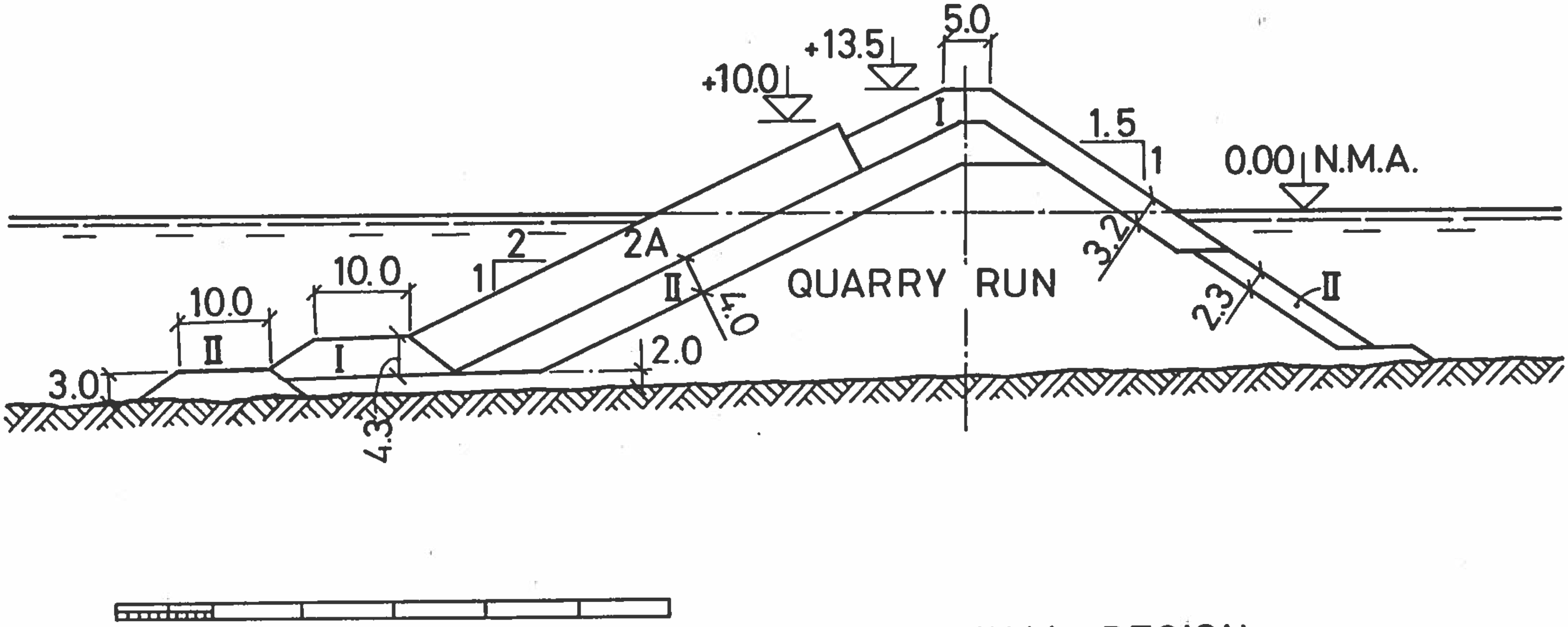
SCHEME DESIGN, GENERAL LAYOUT

FIG. 10

I: 2-4T ROCK

II: 0.2-2T ROCK

2A: 58T BLOCKS



FINAL DESIGN

FIG. 11

DHI FORSØG

DÆKMOLE,	KYSTNÆRE PROFILER,	3 DIM.MODEL	1:35
DÆKMOLE,	HOVEDSEKTIONER,	BØLGERENDE	1:40.6,1:60
DÆKMOLE,	MOLEHOVED,	3 DIM.MODEL	1:70
UROFORSØG,	SKIBS BEVÆGELSER,	3 DIM.MODEL	1:70
UROFORSØG,	FENDERKRÆFTER,	3 DIM.MODEL	1:70
UROFORSØG,	FORTØJNINGSKRÆFTER,	3 DIM.MODEL	1:70
UROFORSØG,	BØLGEHØJDER I BASSIN	3 DIM.MODEL	1:70

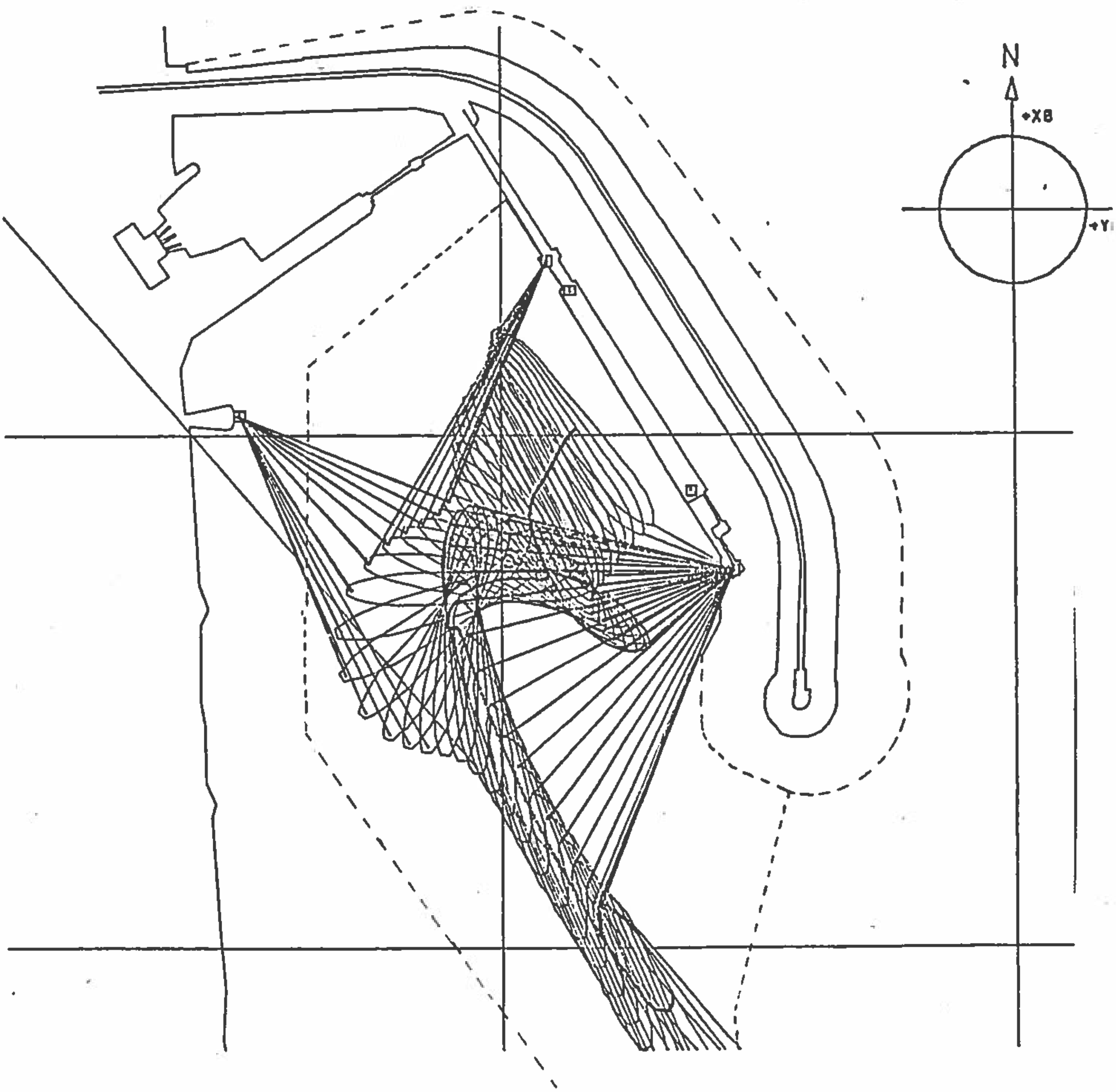
FIG. 12

VERIFICATION CASE
70000 DWT
EVALUATION OF OPERATIONAL DOWN-TIME

	MOTIONS			MOORING FORCE
	SURGE	TRANSVERSAL BOW	VERTICAL BOW	SPRING LINE
	1.5 m	1.0 m	0.5 m	66 TONS
DOWN-TIME HOURS/YEAR	20	22	8	8
CORRESPON-DING H_sm	2.9	2.8	3.5	3.5

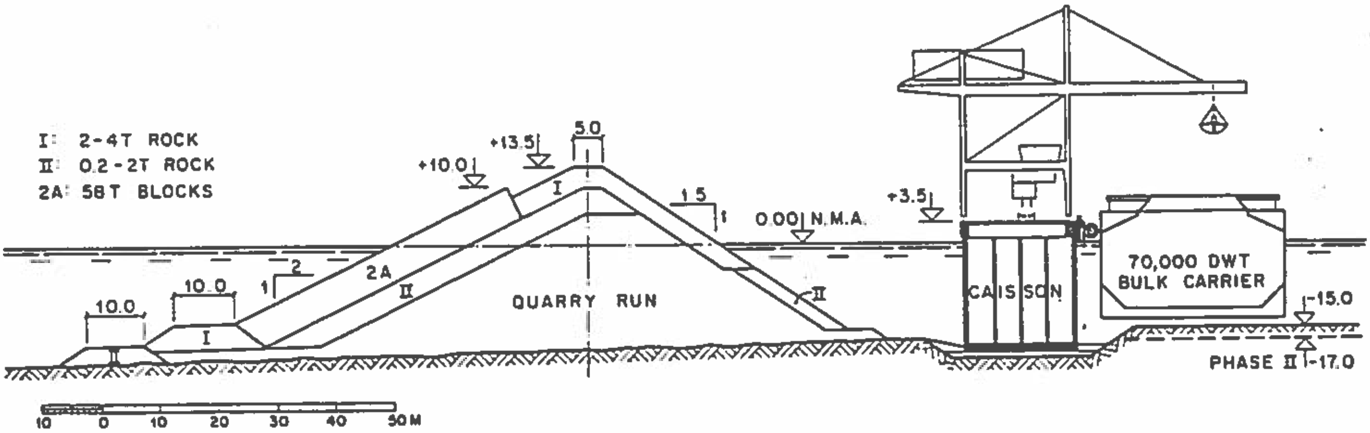
FIG. 13

WIND SPEED m/s:	12	WIND DIR deg.:	45
NOM. CURRENT:	.25	CURRENT DIR:	S



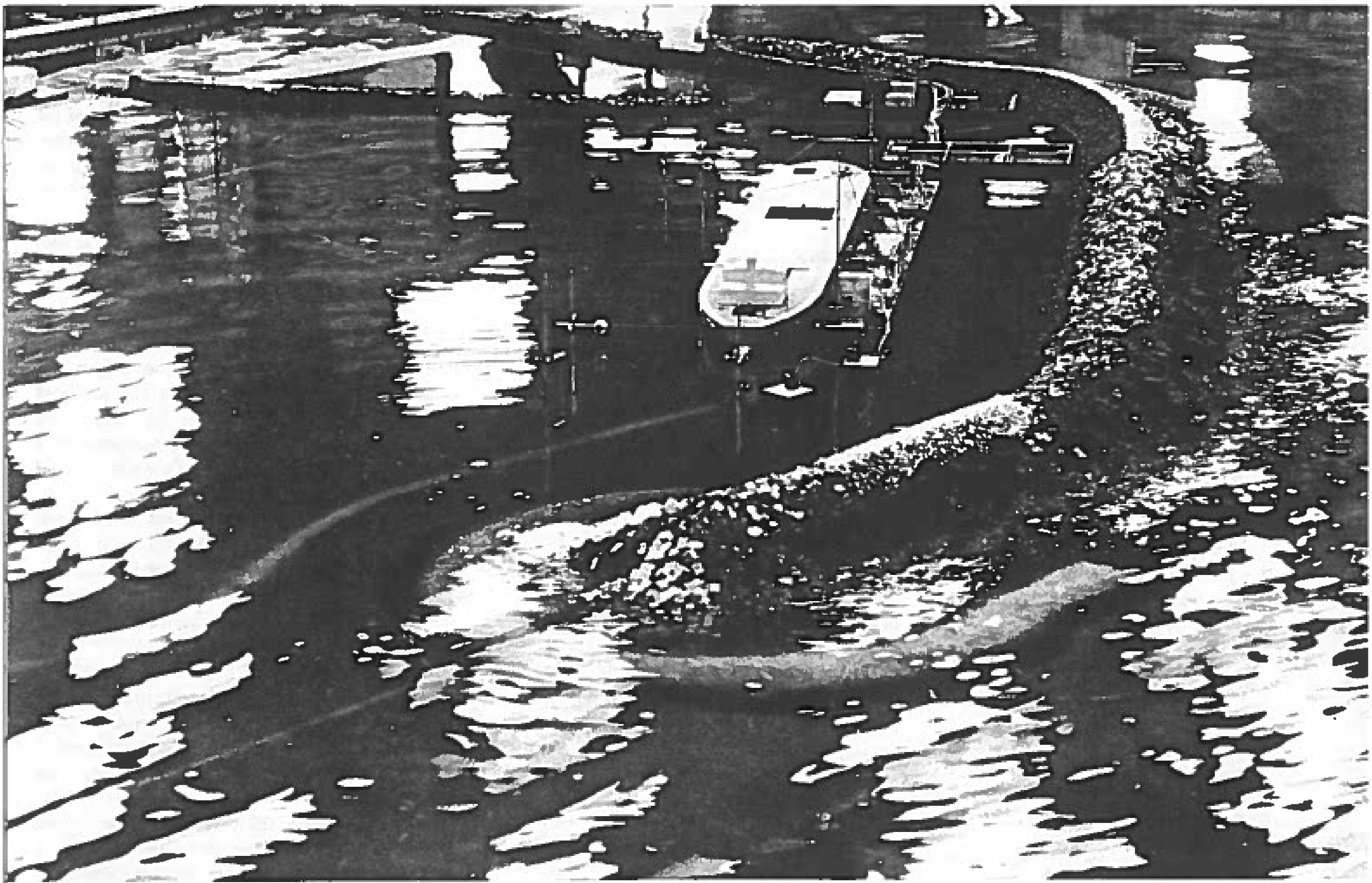
MANØVREFORSØG, SKIBSTEKNISK LABORATORIUM

FIG. 14



SCHEME DESIGN, TYPICAL SECTION IN BREAKWATER AND QUAY

FIG. 15



Model showing breakwater and moored 70,000 DWT coal carrier

Puerto de Carboneras

By Aksel Smith-Petersen and Ole Alenkær Madsen

Ole Alenkær Madsen graduated in 1967 as a civil engineer from the Technical University of Denmark.

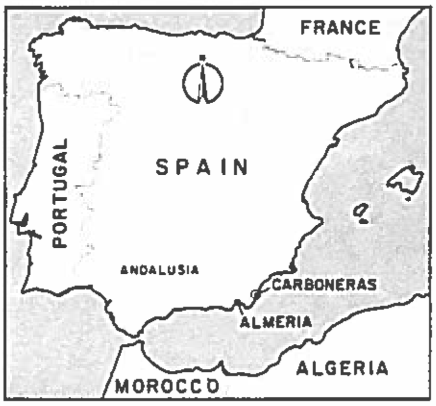
After employment with the Danish State Harbour Administration he joined C-N, Brazil in 1971 where he was engaged as site engineer on the Sepetiba Bay railroad bridge and the Ishikawajima 400,000 DWT drydock.

After 3 years with a firm of consulting engineers in Holland he rejoined C&N, Copenhagen in 1977 as senior design engineer.

At the Head Office he has been engaged in the design of ports and other marine structures and from 1979 to 1981 he was seconded to Algeria leading a team of advisers to the National Port Consultants, Laboratoire d'Etudes Maritimes.

The world-wide trend towards conversion from oil to coal in the energy production has also prevailed in Spain. Existing power plants are being changed to coal firing and new plants are designed for coal firing from the beginning. This is also the case with the new thermo-electric power plant which presently is being constructed near Carboneras, a small town in the Almería province on the Mediterranean coast of Spain.

The construction of the power plant is one of several efforts being made to promote the economic development of eastern Andalusia which has so far been one of the least developed regions of Spain. The region has few natural resources except for the highest amount of sunny hours in Spain, an asset which is the basis for cultivating vegetables in plastic covered greenhouses but which has not yet attracted the tourists in any great number.



The Carboneras power station is being built next to a cement factory which exports cement and clinker through a harbour which accommodates ships up to 40,000 DWT. However, coal to the power station is foreseen to be imported on much bigger ships from overseas sources and, therefore, a new port will have to be constructed.

Responsible for the construction of the new port is Puerto de Carboneras S.A. »PUCARSA«, a limited company within the organization of Empresa Nacional de Electri-

Tendering

Four joint ventures of major Spanish contractors presented proposals which in addition to tenders on the exhibited design also contained many alternatives, some of them elaborated to a high degree of detail and with a considerable amount of work involved.

C&N and INITEC participated in the evaluation of the tenders and also prepared an economic-technical appraisal of some of the alternatives.

A proposal presented by AGREDA, a joint venture of the Spanish firms:

Entrecanales y Távora S.A.,
Dragados y Construcciones S.A. and
Empresa Auxiliar de la Indústria S.A.

was accepted by PUCARSA. It consisted of a combination of the breakwater type and layout proposed by C&N and of an alternative design for the quay consisting of caissons placed on the sea-bottom. According to the tender documents the elaboration of the detail design of the structures was included in the tender price.

Revised Design

After the contractor had been selected it was decided that the power station should have only one unit of 550 MW and that consequently a revised scheme design should be made for a throughput of only 1.8 m t/year, including 0.3 m t/year to be supplied to the cement factory. The size of bulk carriers to be accommodated would be 70,000 DWT, however, with possibility of later accommodating carriers up to 120,000 DWT.

In February 1982, C&N and DHI were awarded the contract for the elaboration of a new scheme design in accordance with the revised conditions. The scope of work was to include investigation in the laboratory of the breakwater stability, the wave agitation in the basin and the effects of the waves on the moored ships.

Later it was accepted to make manoeuvre simulation tests in order to verify or revise the dimensions of the layout.

The scheme design was extended to cover the design of a cooling water intake which was to be placed inside the port.

In general, it is the objective of the new scheme design to establish all principal dimensions of the layout and of the main structural elements such as breakwater profiles, quays, dolphins, bridges etc. Based upon the scheme design the contractor will then work out the detail design.

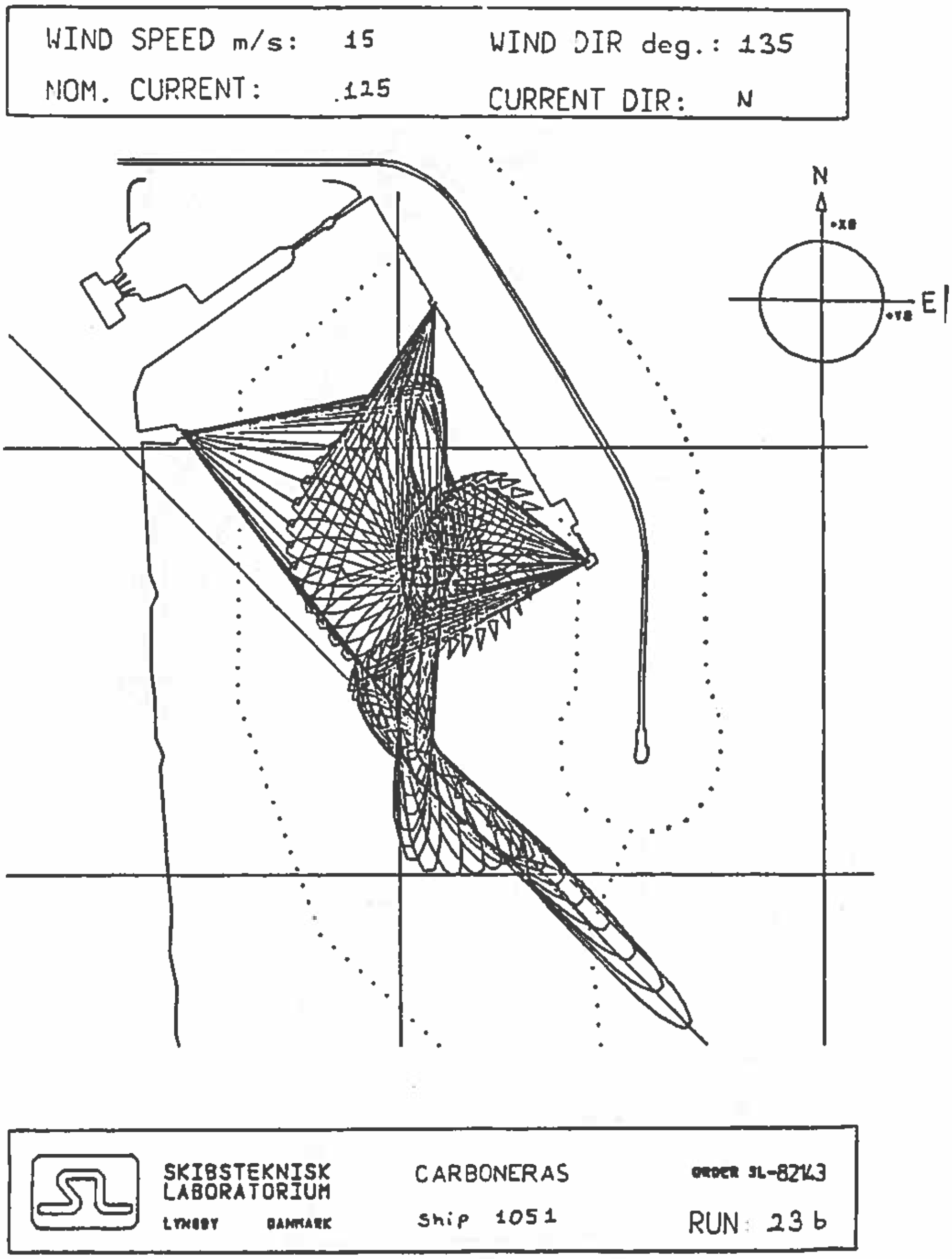


Figure 3. Plot of manoeuvre simulation

Hydraulic Laboratory Tests

The stability tests of the breakwater cross section were made in a wave flume, in the scales 1:40 and 1:60. In connection with these

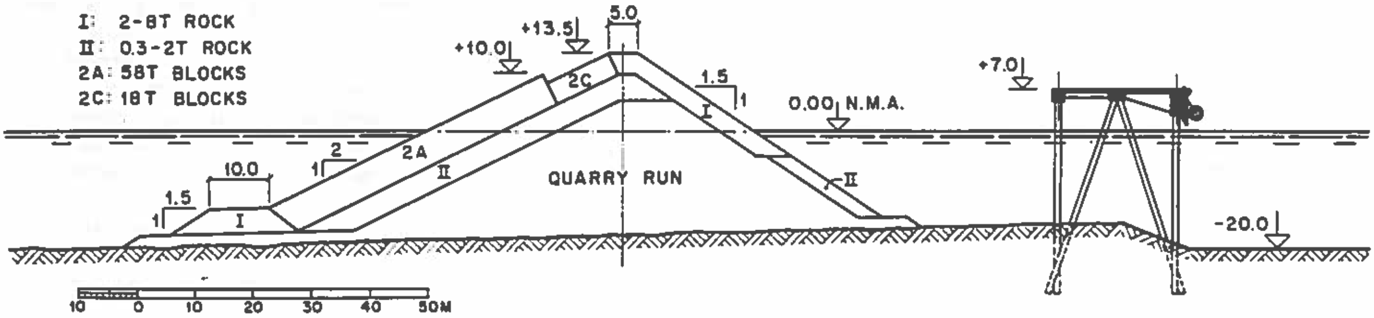


Figure 2. Preliminary design, typical section in breakwater and quay

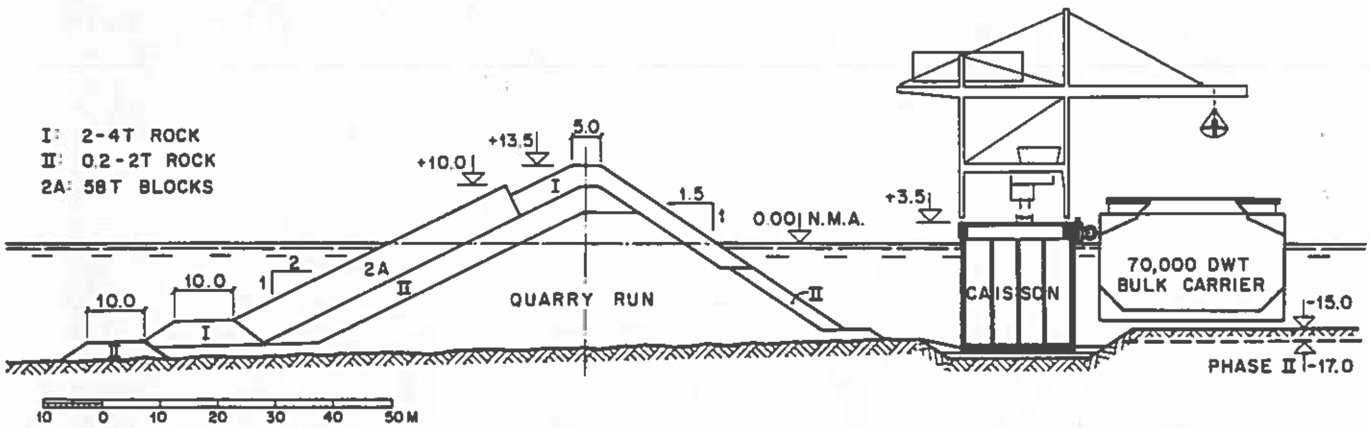


Figure 5. Scheme design, typical section in breakwater and quay

ment of the toe section is necessary in water depths less than 20 m.

On the breakwater head, the main armour is increased to 80 t concrete blocks.

Harbour Basin

The dimensions of the turning basin were determined on the basis of the manoeuvre simulation tests. These comprised manoeuvres with ships assisted by tugs and also manoeuvres in which the ship during all operations inside the port was »assisted« by land based winches. This system is still rather new and only tested in full scale at a few places. However, the tests indicated that it would work just as well as the tugs. At Carboneras the winch system is assumed to be advantageous because it can be provided at a lower cost than a fleet of port-owned tugs which would be required due to the rather large distance to ports where large tugs are available.

With the form of the basin now recommended there will be sufficient space for either type of manoeuvre.

The agitation tests have shown that in extreme weather situations there will be created a seiche in the basin which produces so large motions of the moored ship that it may have to leave quay. Since in such situations the storm will make it impossible to turn the ship inside the port it is recommended that the ship is always moored with the bow pointing outwards from which position departure can be made also during adverse conditions. This means that the turning manoeuvre must be made on arrival, with the laden ship, and consequently the turning area will have to be dredged to full depth.

The total area of the basin is 47 hectares out of which 25 hectares are dredged to −15 m in the first phase when 70,000 DWT ships are received. In phase 2, for 120,000 DWT ships, the dredging will be increased to −17 m.

Unloading Quay

The quay is 240 m long by 21 m wide and consists of 8 No. 30 m long by 19.7 m wide caissons. The foundation level is −19 m whereby dredging to −18 m can be made without making any modifications to the quay structure. The subsoil is a rather dense sand that is well suited as foundation for the caisson structures.

The fendering consists of 9 large cylindrical rubber fenders hanging on the front of the caissons.

The agitation tests have shown that the surge motion of the moored ship decreases with increased friction coefficient of the fender surface, therefore, fenders without pads have been adopted in order to take advantage of the high friction between rubber and ship's side.

Mooring bollards and quick release hooks are placed along the quay front and on 4 dolphins, two at each end of the quay. The outer dolphins (No. 1 and 4) also carry the manoeuvring winches mentioned above. A third manoeuvring winch is placed on the dolphin No. 10 near the shore. All these dolphins are made as rectangular caissons with a structure similar to those of the quay.

The quay will carry two unloading gantries which grab the coal from the ships' holds and deliver it to the rubber belt conveyor running to the storage yard ashore. Between the quay and the transfer station the combined conveyor and road bridge is supported on caissons (two of which are the abovementioned mooring dolphins). At the transfer station the conveyor branches off and runs direct to the storage yard while the road runs to the shore on a berm on the inner side of the breakwater. Near the shore the conveyor runs on top of the secondary breakwater which protects the cooling water intake.

Cooling Water Intake

With the outfall situated on the beach to the north of the port and the intake inside the harbour basin, the 1.1 km long breakwater will prevent the hot water of the outfall from being mixed with the water at the intake.

Analyses made by DHI confirmed that there will be no risk of suspended sand entering the intake through the harbour entrance. However, the intake must be protected against sand and silt raised by the waves and by the propellers of berthing and departing ships. Several configurations of intake and protective structures were studied theoretically and in connection with the agitation tests before choosing the final layout.

It was eventually concluded that a 100 m long channel of still water in front of the intake structure is required to ensure that suspended material would settle before reaching the intake. This channel is created by constructing a secondary breakwater, see Figure 4, which at the same time serves as support for the coal conveyor.

Other Structures

The contractor's working harbour is placed in the south-west corner of the basin. After completion of the port it may be converted into a shelter for fishing boats.

Construction

The execution of the work started mid 1981 and is scheduled to be finished by May 1984.

EN KULHAVN VED GLATVED

ved

overingeniør Jan Løgstrup,
Rambøll & Hannemann

EN KULHAVN VED GLATVED

Jan Løgstrup

Baggrund

Som følge af usikkerheden med hensyn til kernekraft søger Elsam alternativer i kulkraft til dækning af fremtidige effektbehov som følge af nedslidning af eksisterende værker og den generelle stigning i effektefterspørgslen.

En konsekvens heraf er et behov for nye kulkraftværker og nye kulhavne. Her skal alene omtales de overvejelser, der ligger forud for, at en lokalitet udvælges og detailprojekt besluttes.

Overvejelserne omfatter naturligvis momenter i forbindelse med selve kraftværket såvel som momenter i forbindelse med havneanlægget, hvorfor det endelige valg vil afhænge af en samlet vurdering.

Nærværende indlæg skal imidlertid stort set alene beskæftige mig med de havnemæssige vurderinger, hvortil Elsam-Kraft, Dansk Hydraulisk Institut, Dansk Geoteknisk Institut og Rambøll & Hannemann har ydet bidrag.

Hovedforudsætninger

De vil være naturligt først at se på de planlægningsforudsætninger, som Elsams Kraftværksafdeling og Brændselsafdeling opstillede (jfr. fig. 2).

Man ser bl.a. heraf, at der skal planlægges for en kulimport på ca. 8 Mt/år i den endelige udbygning, hvoraf ca. 4 Mt/år skal forudsættes reeksporteret dels søværts dels landværts.

Man bemærker også, at største skivbsstørrelse i 1. fase er forudsat til 150.000 t dw og 250.000 t dw i 2. fase.

Overvejelser hos Elsam førte til, at man udpegede en række potentielle lokaliseringsmuligheder, som bl.a. gerne skulle opfylde flere af følgende betingelser:

- lokaliteten skal have god plads til et kulkraftværk, gerne på en lokalitet som i forvejen er reserveret til et kernekraftværk.
- nærhed til forbrugstyngdepunkt.
- mulighed for fjernvarmeafsætning.
- gode havnemuligheder med dybt vand.

Dette førte til, at de lokaliteter, der er vist på kortet (fig. 1), blev udpeget, idet dog lokaliteten ved Frederikshavn blev foreslået af Frederikshavn Kommune.

Forbrugstyngdepunkt

Med hensyn til placeringen i forhold til tyngdepunktet for elforbruget, kan det konstateres, at Frederikshavn ligger dårligt, mens Rugård og Kobbergård er godt placeret og Risinge mindre godt.

Besejlingsforhold

Besejlingsforholdene (jfr. figur 3 og detailkort) for Frederikshavn må betegnes som gode ligesom for Rugårds vedkommende. Kobbergårdplaceringen anses for meget god på grund af den lille afstand til rute T og den korte, lokale sejlrende, mens Risinge anses for mindre god i denne henseende på grund af de mere komplicerede tilsejlingsforhold og den lange, lokale sejlrende.

Øvrige karakteristika er ligeledes sammenstillet i figur 3.

Havnearrangementer for forskellige lokaliteter

Frederikshavnforslagene er (fig. 4 og 5) placeret uden for kysten som en ø, der er forbundet med land med en dæmning. Dæmningens buede form er betinget af hensynet til nogle udvidelsestanker, man har for Frederikshavn havn.

Orienteringen af hovedkajen er gjort strømret. Placeringen af havneøen er resultatet af en økonomisk optimering, idet sejlrendens retning og bredde indgår som en funktion af forholdet mellem tværstrømmens hastighed og sejlhastigheden. Sejlrenden bliver ca. 3,5 km lang. Det ene forslag (fig. 4) viser en beskyttet kajplacering, og det andet forslag (fig. 5) en ubeskyttet hovedkaj. (Det gælder for Frederikshavn som for efterfølgende lokaliteter, at der kun er vist typiske eksempler på de undersøgte muligheder).

Rugård forslagene (figur 6, 7 og 8) forudsætter et kystnært anlæg. Det ene anlæg (fig. 6) er arrangeret med en lukket havn, mens det andet anlæg (fig. 7 og 8) er arrangeret med en fritliggende dækmole, som tillader, at lettede kulskibe kan forlade havnen fremover. I begge anlæg er kajerne placeret strømrette. Også her er sejlrendens retning og bredde optimeret ud fra tilsvarende parametre som nævnt for Frederikshavn.

Kobbergård anlægget (fig. 9) svarer i princip til Rugård anlægget; men da det dybe vand findes tæt på kysten, bliver arrangementet her smalt, da også landarealerne er snævre. Til gengæld kan der opnås en sejlrende på kun 0,5 km.

Da kysten ved Risinge (fig. 10 og 11) er lavvandet i forhold til Rugård og Kobbergård, har man her set sin fordel i et arrangement som medfører, at kajen kan placeres ret langt fra kysten. Men alligevel bliver sejlrenden lang, ca. 12 km.

Anlægsudgifter

En sammenligning mellem anlægsudgifter er foretaget i fig. 12.

Det fremgår, at Kobbergård 1 er billigst såvel i fase 1 som i fase 2. Mens Rugård og Risinge er nogenlunde jævnbyrdige i fase 1, skiller Rugård sig ud i fase 2 som det billigste af de to forslag. Udgifterne til uddybning og dækmoler ses at være udslagsgivende. Kan dækmolerne undværes ved Rugård og Kobbergård, forbedres disses økonomi væsentligt i forhold til Risinge. Undlades adgangsdæmningen i Frederikshavn vil økonomien forbedres væsentlig for disse forslag ikke alene med udgiften til adgangsdæmningen, men også derved, at anlægget med fordel kan flyttes længere ud.

Men en væsentlig ulempe ved Frederikshavn er de manglende muligheder for placering af et kraftværk.

Som tidligere nævnt skal valget af lokalitet afgøres af en samlet vurdering af havnemæssige og af kraftværksmæssige forhold, og denne vurdering har peget på Rugård, som herefter benævnes Glatved.

Dernæst er en usikkerhed med hensyn til beliggenheden af kalkens overflade blevet undersøgt nærmere, idet er høj beliggenhed ville have væsentlig indflydelse på anlægsomkostningerne især uddybningsprisen og på kajkonstruktionenes udformning. (fig. 14).

Glatved

I dette trin af undersøgelsen har vurderingerne derfor samlet sig om Glatved, idet planlægningsforudsætningerne for havneanlægget i mellemtiden er blevet revurderet, således at der nu dels er defineret en fase 1 og fase 2 med tilsvarende kapaciteter dels er foretaget nogle justeringer (fig. 13).

Det fremgår, at fase 1 alene forudsætter en kulhavn anlagt uden samtidigt anlæg af kraftværk, således at al kul skal eksporteres til andre kraftværker.

De første informationer antydede, at der ville være en risiko for at møde kalk under havbunden, således at såvel uddybninger som kajkonstruktioner risikerede at blive fordyret heraf. En nærmere undersøgelse af kalkens beliggenhed blev derfor iværksat, og resultatet heraf fremgår af figur 14. Kalkens placering var en medvirkende årsag til, at positionen Glatved blev foretrukket.

Som det fremgår at overslagene (fig. 12), udgør anlægget af en dækmole for importkajen en væsentlig udgift i de samlede anlægsomkostninger, hvorfor det er nærliggende at overveje, om anlægget af en dækmole helt kan undlades, de store skibe taget i betragtning. På grundlag af en vurdering af, hvor megen ekstra skibstid, (fig. 15, DHI), der tabes som følge af, at skibene er for urolige til losning eller ikke kan ligge sikkert fortøjet, kan det beregnes, at nutidsværdien heraf er betydningsløs i forhold til anlægsudgiften til en dækmole. (Man kan her tage i regning, at kulkranerne alligevel ikke kan arbejde, når vindhastigheden kommer over en vis størrelse. For skibe over 100.000 t dw vil kranernes stilstand p.g.a. vind helt dække de perioder, hvor skibene er for urolige til losning). Der er siden iværksat nogle overvejelser (ved prof. Per Brun) med hensyn til fortøjnings- og fenderarrangementer, med den hensigt at reducere den tabte skibstid og antallet af udsejlinger i tilfælde af dårligt vejr.

Efter disse overvejelser er der udarbejdet et nyt arrangement for Glatved, som tillader den nævnte faseopdeling i udbygningen (fig. 16 og 17). Da anlægsudgifterne til kajerne vejer tungt i det samlede omkostningsbillede, er udgifterne hertil kalkuleret på grundlag en af række skitseprojekter, der undersøger økonomien ved forskellige konstruktionsprincipper.

Undersøgelsen vedrørte dybvandskajerne, og omfattede følgende konstruktionstyper og gav de angivne relative priser

- Ballaseret flercellecaissoner	:	90%
- Spunsvæg af rørpæle	:	100%
- Platformskaj med spunsvæg af rørpæle	:	95%
- Åben brokaj	:	130%

En endelig løsning skal af prismæssige grunde søges blandt en af de tre først nævnte muligheder. Blandt disse synes caissonløsningen at skille sig ud, også fordi den kan have nogle tekniske fordele; men en mulighed for dybereliggende dyndlag under kajen kan dog udelukke denne løsning. Derimod har den åbne brokaj nogle tekniske ulemper (sårbarhed over for is, mere risikabel udførelse, ringere anlægsforhold), som udover prisen gør den uegnet.

Figur 16 og 17 viser det reviderede forslag i hhv. fase 1A og fase 2. Overslaget for fase 1A er anført i figur 12 under Glatved.

Samtidigt er kulpladsen placeret i større afstand fra kystlinien, således at eksportpramme og -skibe (op til 20.000 t dw) kan placeres i læ af kulpladsen. På kulpladsens sydøstside anlægges mod syd en anlægsplads for importskibe i en længde på 285 m og forudset til at kunne uddybes i fase 1B til -22,0 m fra -19,0 m. I en fase 2 er kulpladsen forudset udvidet mod nord (fig. 17) og importkajen forøget med 330 m på 19 m vand.

Ved den valgte placering fjernere fra kysten opnås endvidere, at fremtidige kølevandskanaler kan placeres mellem kysten og kulpladsen samt en bedre jordbalance mellem uddybning og opfyldning.

Som det fremgår at figur 12, er der herved sket følgende forskydninger i overslaget i forhold til Ruggård 1.

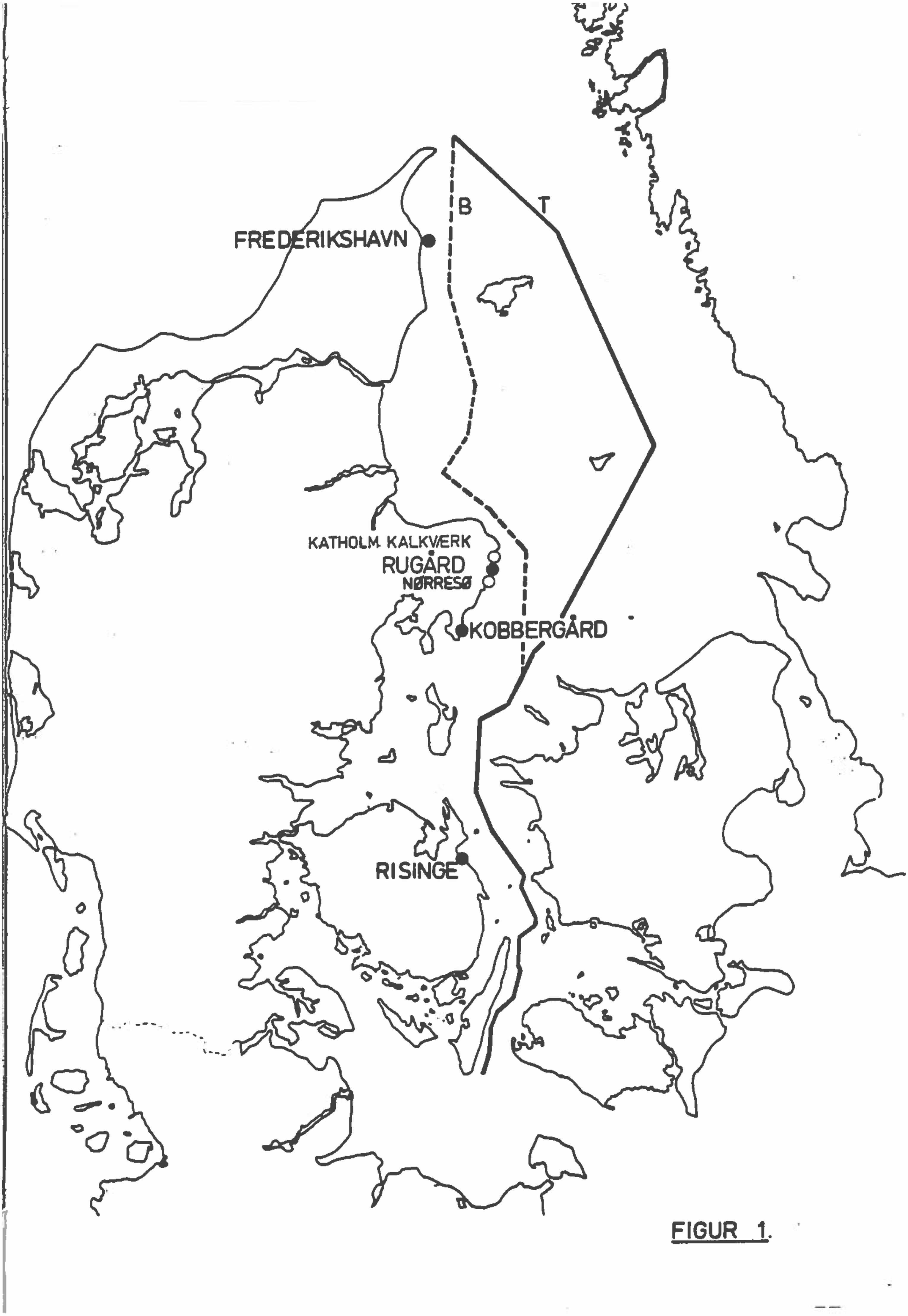
- anlæg eksportbåde (før 8.000 tdw, nu 20.000 tdw)		+ 5 mio. kr.
- anlæg for 160.000 tdw		
uddybning	- 27 mio. kr.	
dækmole	- 25 - -	
kaj (nu forberedt for 22 m vand)	+ 33 - -	- 19 - -
- areal uden for kyst		+ 23 - -
- total ændring		+ 9 mio. kr.

men her skal det bemærkes, at der nu er investeret i en kajkonstruktion, som senere kan uddybes til - 22,0 m. Af fig. 12 ses, at værdien heraf en 77-39 = 38 mio. kr.

Slutning

En forudsætning for at det beskrevne anlæg realiseres vil bl.a. være, at følgende udviklingslinier mødes:

- behovet for el-effekt skal stige tilstrækkeligt
- kernekraft kommer ikke i betragtning
- udviklingen i skibsstørrelser og havnekapaciteter gør de forudsatte skibe realistiske.


B
T
FREDERIKSHAVN
KATHOLM KALKVÆRK
RUGÅRD
NØRRESØ
KOBBERGÅRD
RISINGE

FIGUR 1.

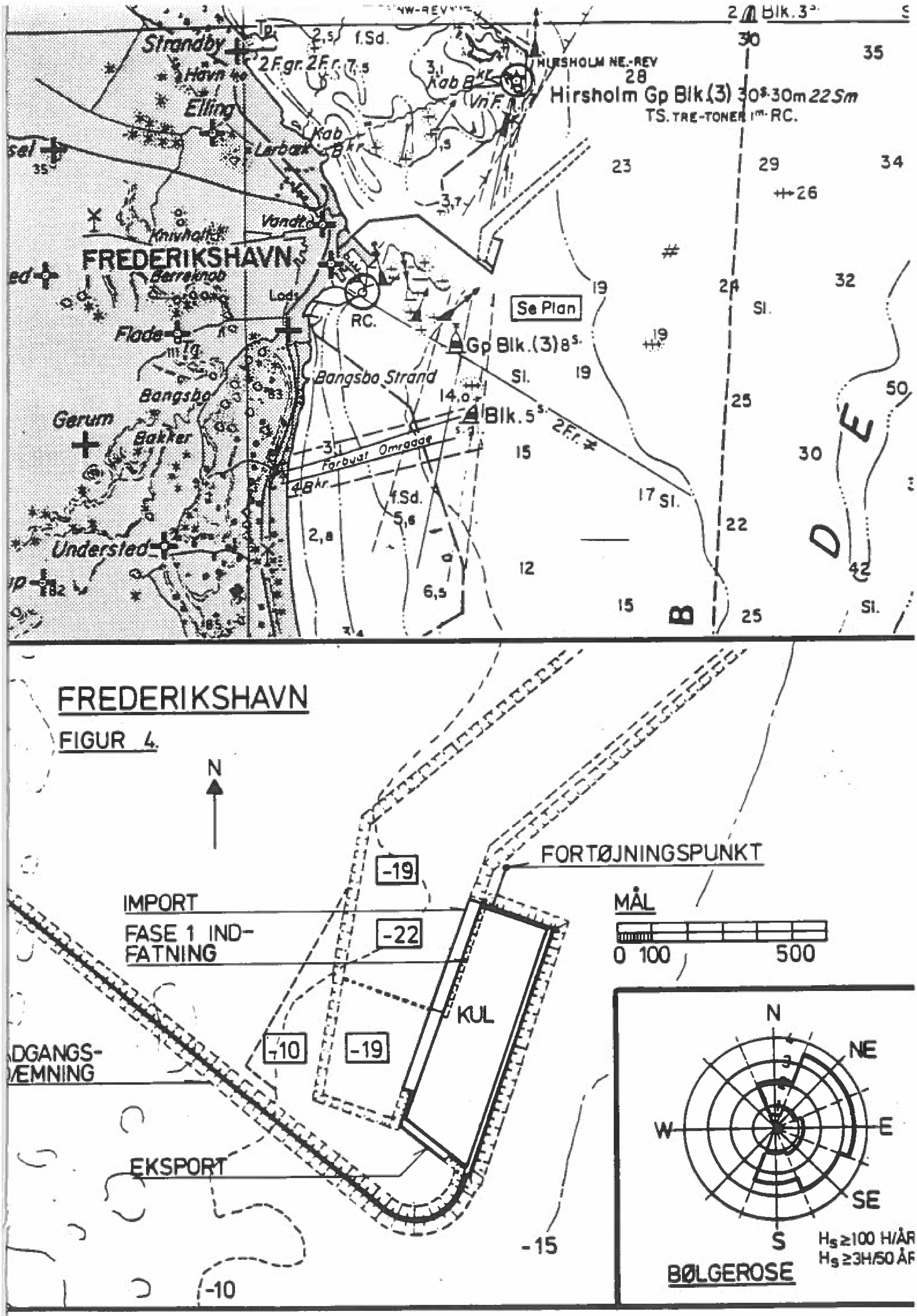
FIG. 2: PLANLÆGNINGSFORUDSÆTNINGER

- KULIMPORT		CA. 8,0 MT/ÅR
HVORAF		
- KULEKSPORT		
. I PRAMME	CA. 3,0 MT/ÅR	
. AD JERNBANE OG LANDEVEJ	CA. 1,0 MT/ÅR	CA. 4,0 MT/ÅR
- KULLAGERPLADS		
. MÆNGDE		CA. 4,5 MT
. AREAL		CA. 25 HA
- OLIELAGERPLADS		CA. 15 HA
- SKIBSSTØRRELSER		
. IMPORT 1. FASE		MAX. 150.000 DWT.
. IMPORT 2. FASE		MAX. 250.000 DWT.
. EKSPORT - PRAMME		CA. 7.000 DWT.
. FLYVEASKE		CA. 6.000 DWT.
. MASKINTRANSPORTER		CA. 3.000 DWT.

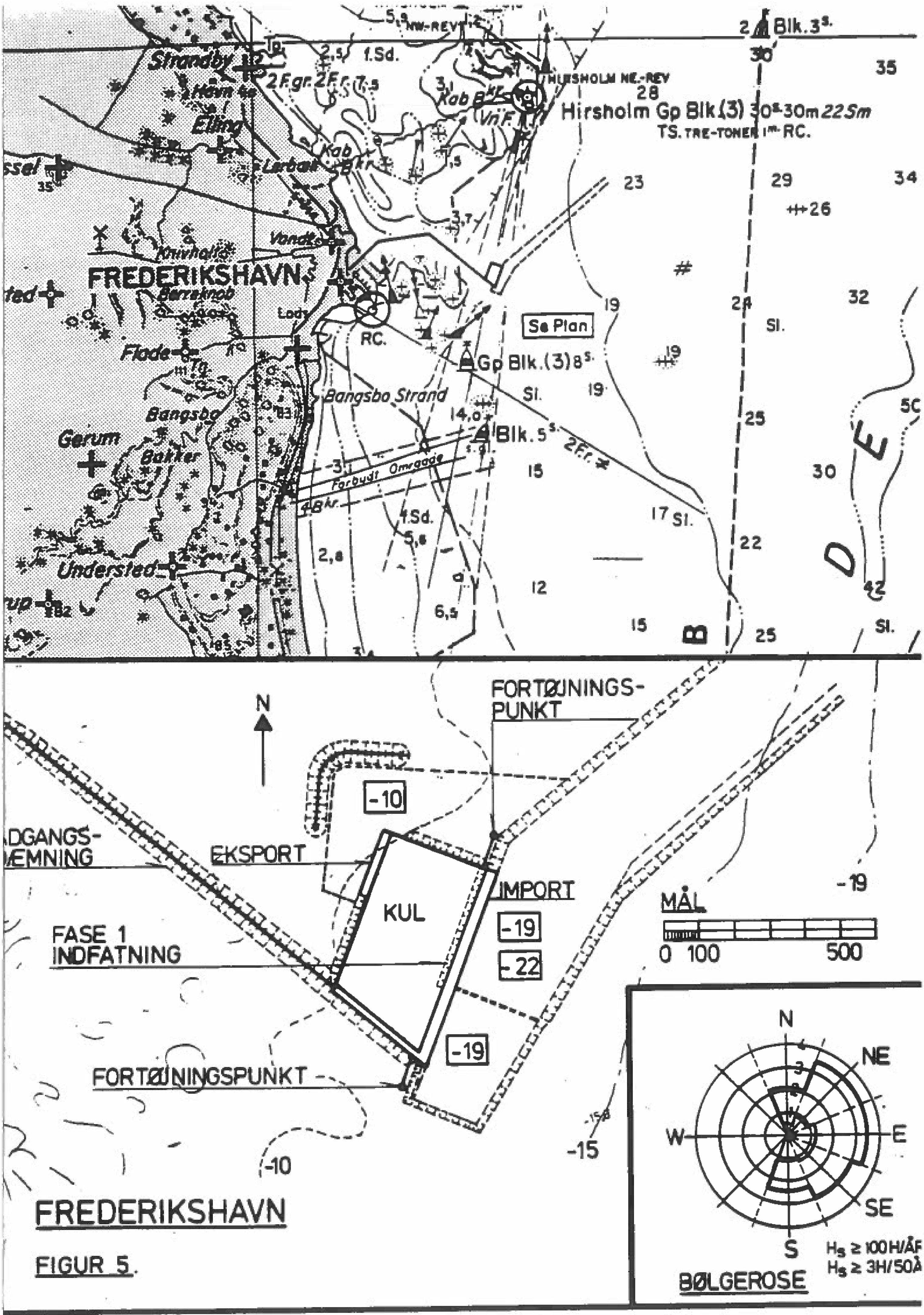
- KAJLÆNGDER OG VANDDYBDER
  . IMPORTKAJ
  1. FASE LÆNGDE: 590 M DYBDE: 19 M
  2. FASE LÆNGDE: 350 M HERAF UDDYBES TIL: 22 M
  . EKSPORTKAJ (KUL OG ASKE)
  LÆNGDE: 150 M DYBDE: 8 M

- ANDRE FORUDSÆTNINGER.
  IMPORTKAJEN SKAL HAVE ET SIDEAREAL, HVOR KULLASTEN KAN PLACERES I TILFÆLDE AF DELVIS FUNKTIONSSVIGT AF KULTRANSPORTANLÆG.

FIG. 3: KARAKTERISTIKA FOR LOKALITETERNE	FREDERIKSHAVN	RUGARD	KOBBERGARD	RISINGE
PLACERING I FORHOLD TIL TYNGDEPUNKT	DARLIG	GOD	GOD	MINDRE GOD
MULIGHED FOR PLACERING AF KRAFTVÆRK	INGEN	GOD	GOD	GOD
BESEJLING	(MEGET) GOD	GODE	MEGET GOD	MINDRE GOD
NØDVENDIG SEJLRENDE LÆNGDE KM	3,5	2,5	0,5	12
JORDBUND	POSTGLAC. LAG SAND?	SAND MORÆNELER KALK	MORÆNELER SAND DYBERE KALK	MORÆNELER SAND TØRVEDYND
BØLGER, H_S 100 H/AR M	2,8 (E+NE)	1,7 (E)	1,7 (E)	1,0 (E)
3 H/50/AR M	3,6 (E+NE)	3,6 (E)	3,6 (E)	2,5 (NE)
STRØMHASTIGHED MAX. KNOB	2,5	1,5	1,5	2,6
HYPPIGHED > HASTIGHED	6% > 1,5 KN	1% > 0,8 KN	1% > 0,8 KN	1% > 1,9 KN
RETNING: HYPPIGHED	N: 70%	N: 80%	N: 75%	N: 60%
IS VARIGHED AF SVÆR IS SOM OVERSKRIDES 1 GANG/10 AR	10-15 DAGE	10-15 DAGE	CA. 20 DAGE	CA. 30 DAGE
ISTYKKELSER SOM OVERSKRIDES 1 GANG/10 AR	30 CM	40 CM	40 CM	43 CM
SEDIMENTATIONSTRANSPORT STØRRELSESORDEN M^3/AR	(0/80.000)?	5.000	10.000	1.000

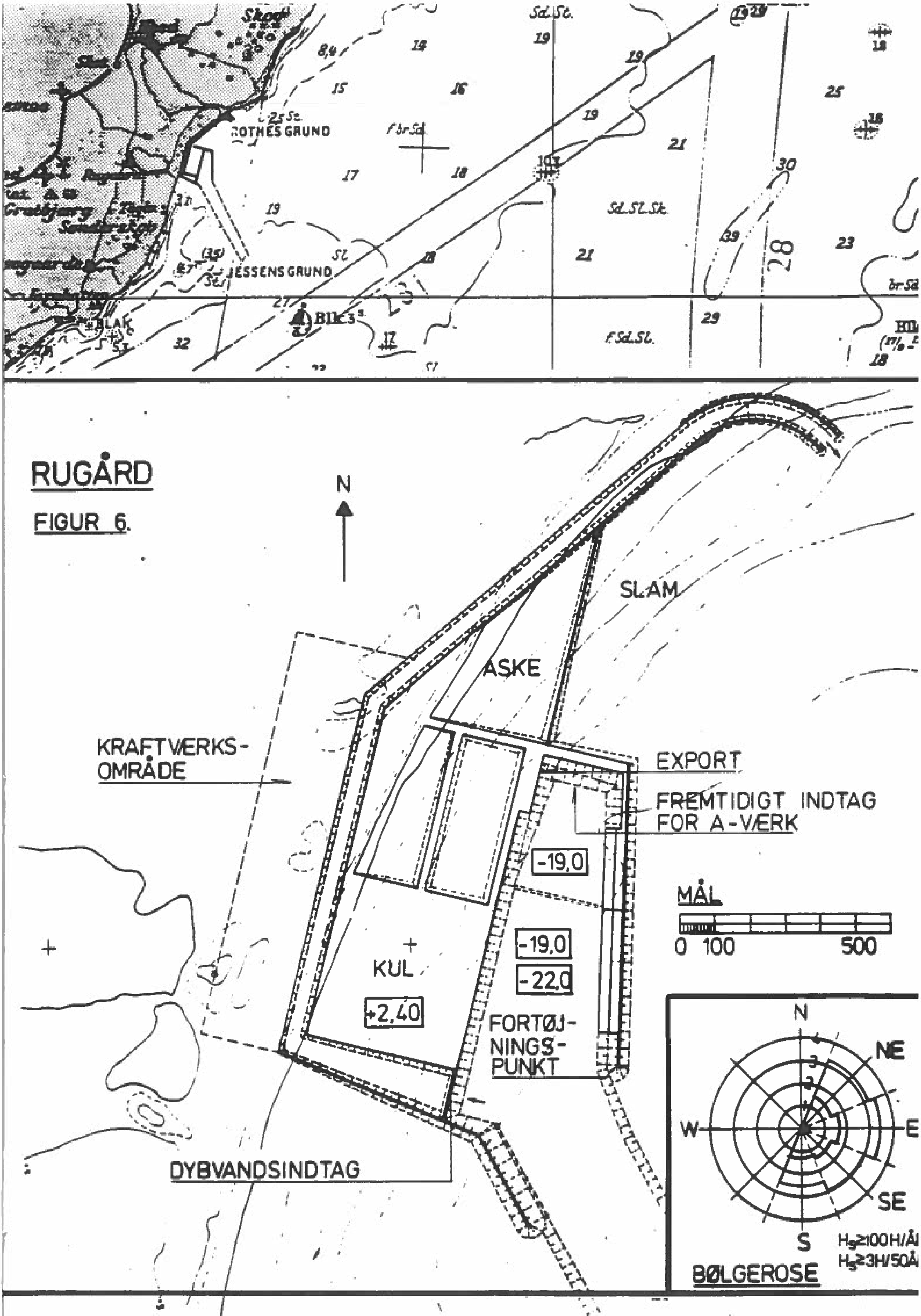


FREDERIKSHAVN
FIGUR 4.
Strandby
Elling
Lerbæk
Vandt.
Knivholt
FREDERIKSHAVN
Lods
RC.
Flade
Bangsbo
Gerum
Bakker
Understed
Bangsbo Strand
Forbudt Omraade
HIRSHOLM NE.-REV
Hirsholm Gp Blk.(3) 30s.30m 22Sm
TS. TRE-TONER 1m. RC.
Se Plan
Gp Blk.(3) 8s.
Blk. 5s.
N
FORTØJNINGSPUNKT
IMPORT
FASE 1 IND-
FATNING
KUL
-19
-22
-10
-19
ADGANGS-
DÆMNING
EKSPORT
-15
-10
MÅL
0 100 500
N
NE
E
SE
S
W
Hs ≥100 H/ÅR
Hs ≥3H/50 ÅR
BØLGEROSE



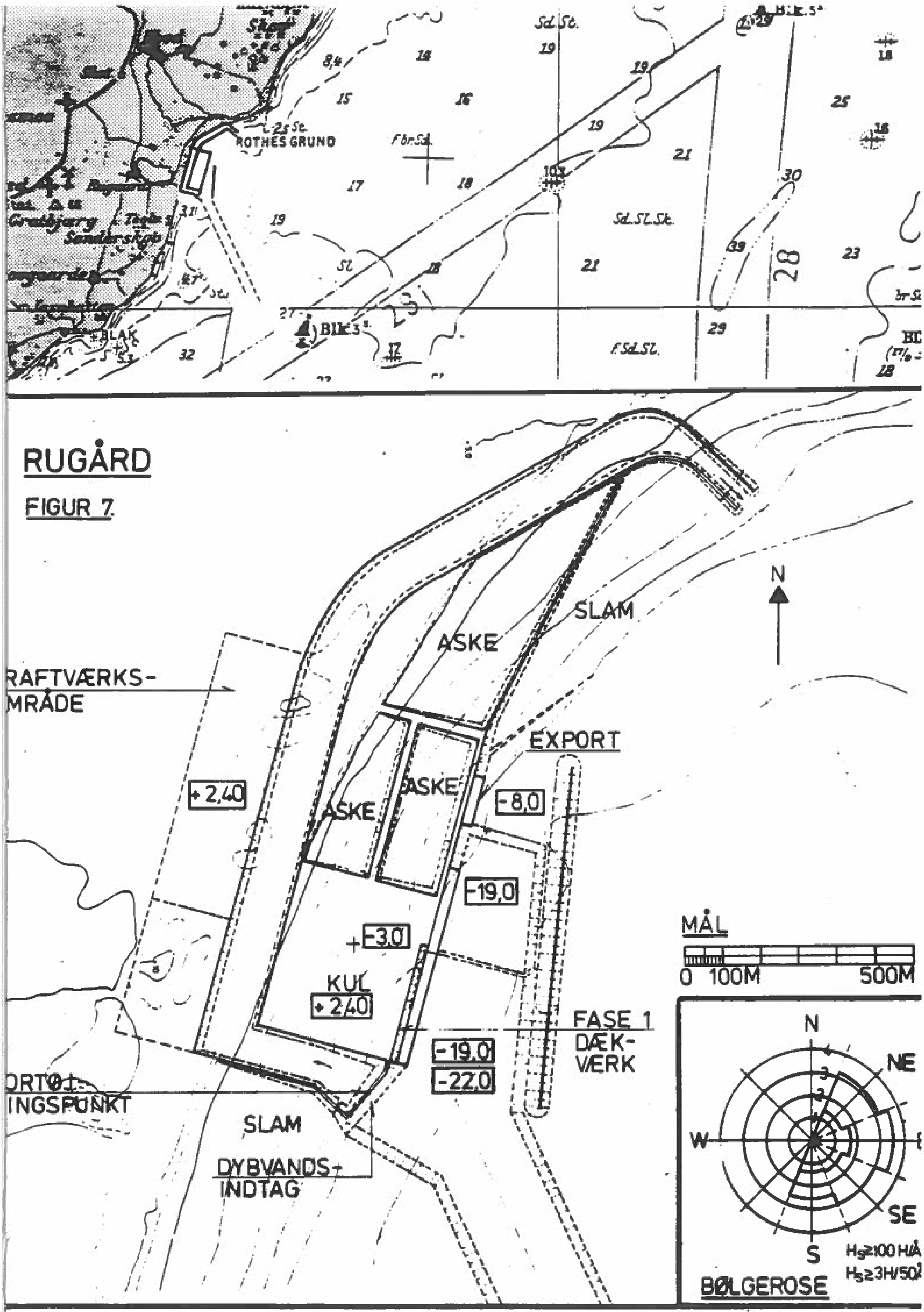
FREDERIKSHAVN

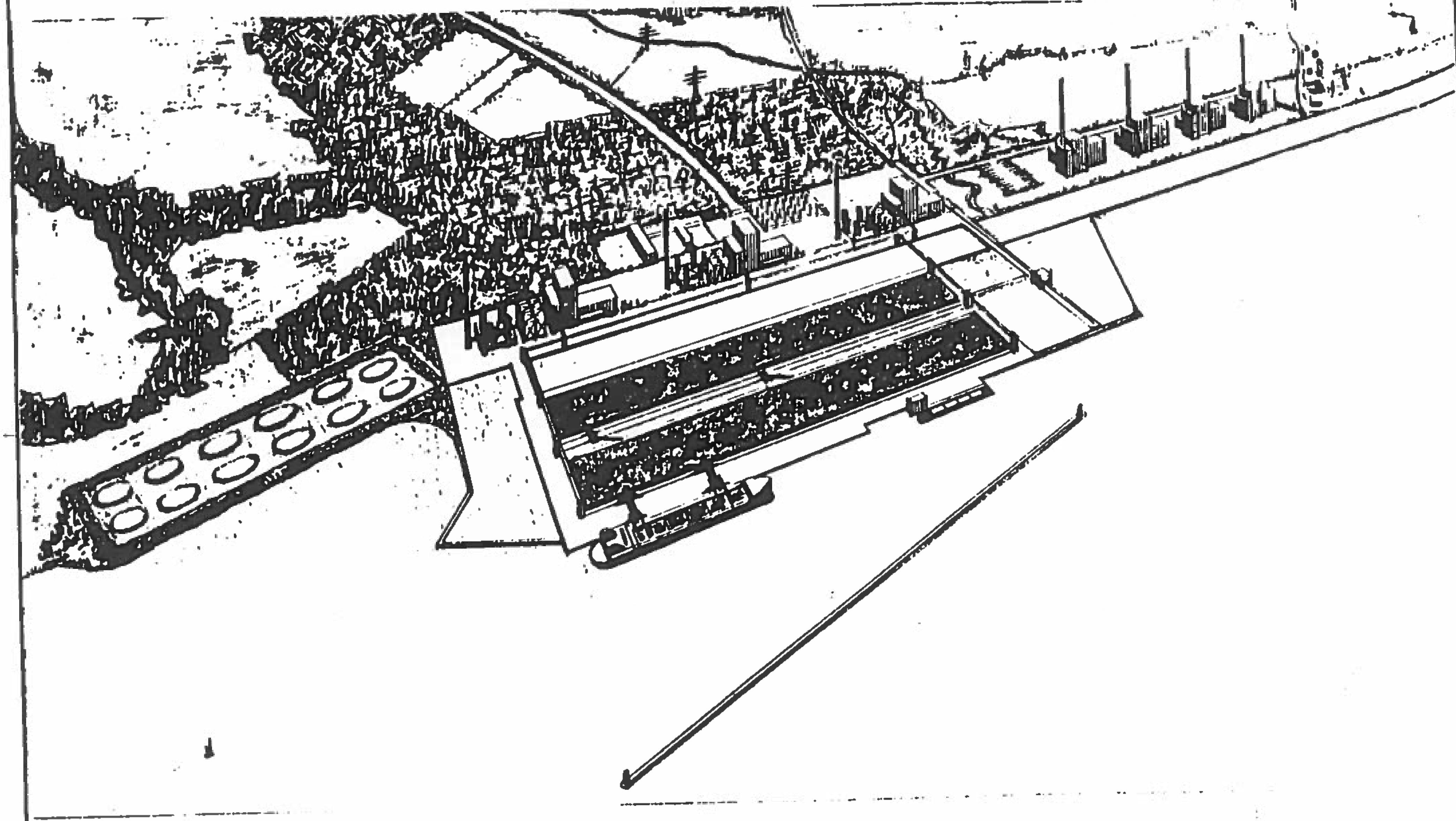
FIGUR 5.



RUGÅRD

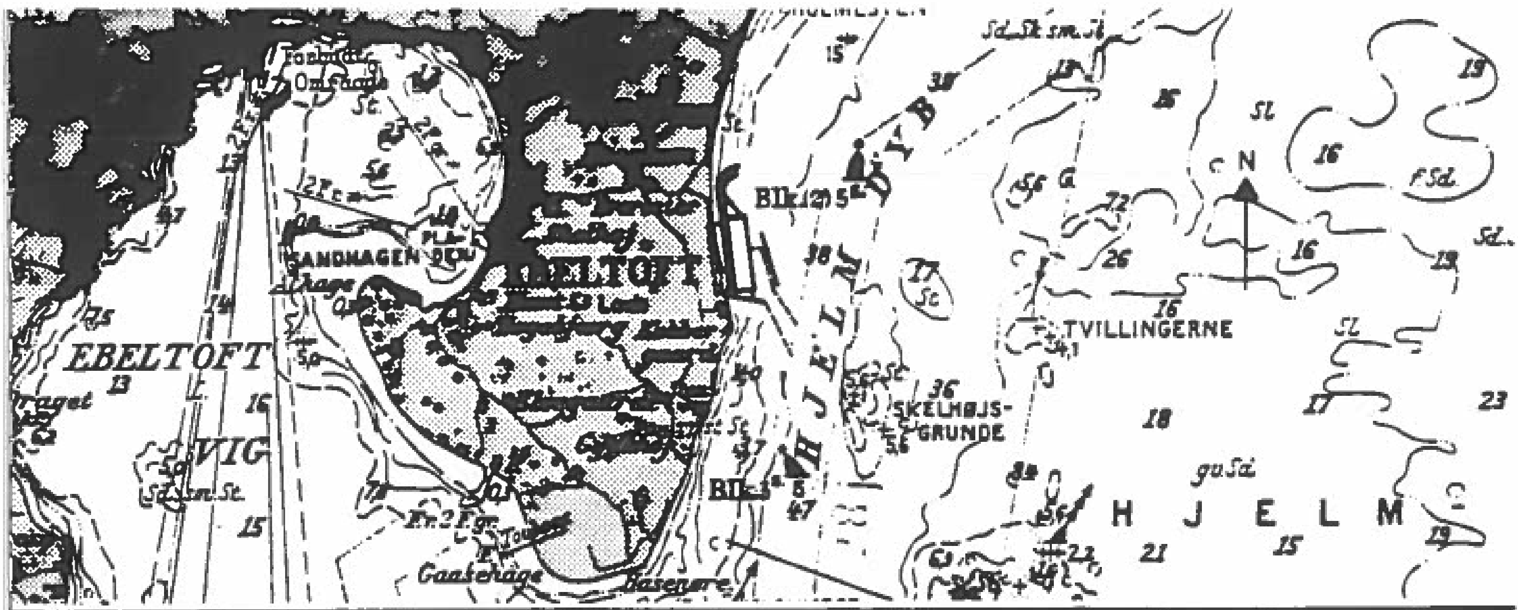
FIGUR 6.

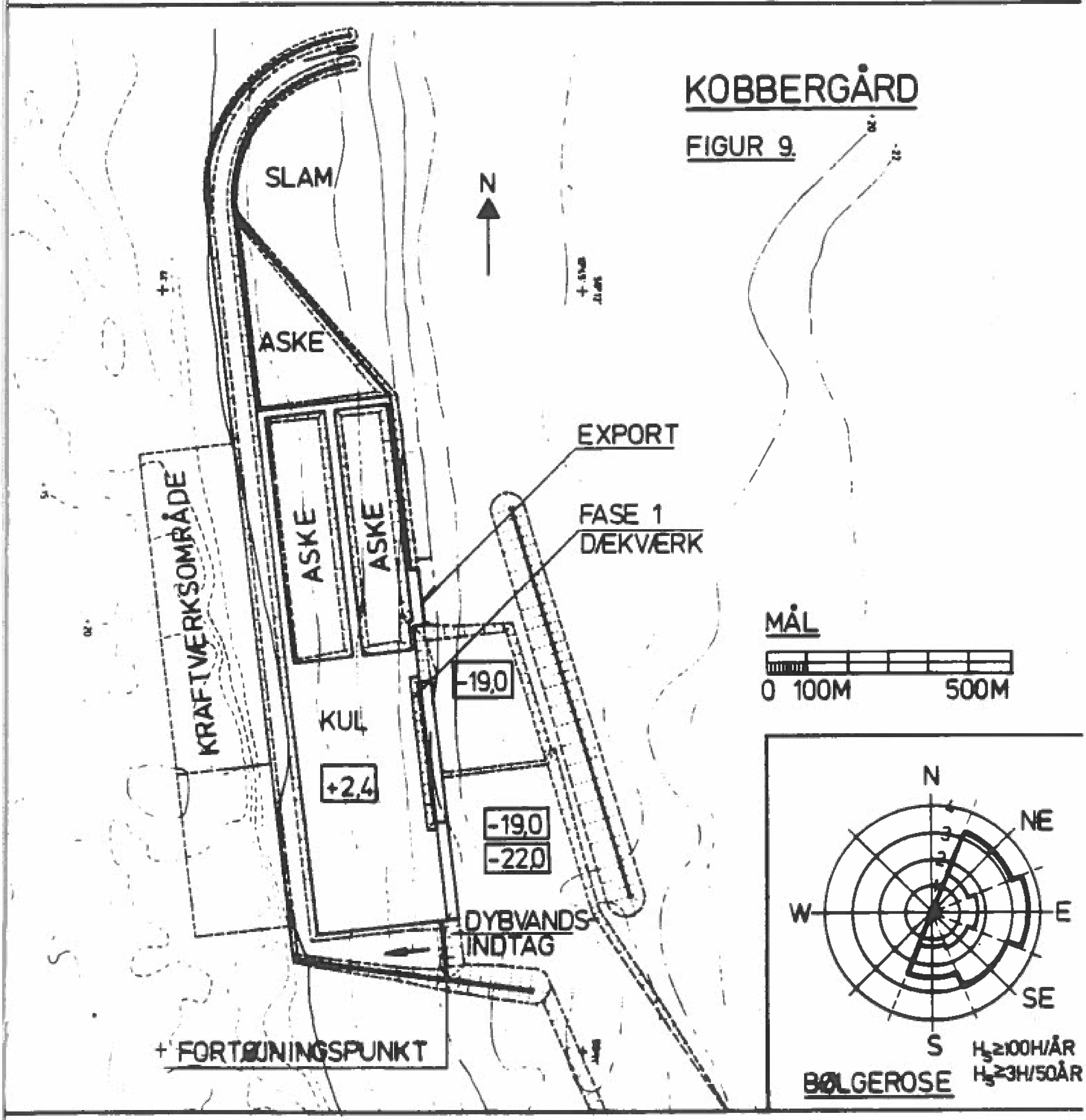

RUGÅRD
FIGUR 7
RAFTVÆRKS-
MRÅDE
ASKE
SLAM
ASKE
ASKE
EXPORT
+2,40
-8,0
-19,0
-3,0
KUL
+2,40
-19,0
-22,0
FASE 1
DÆK-
VÆRK
ORTØJ-
INGSPUNKT
SLAM
DYBVANDS-
INDTAG
N
MÅL
0 100M 500M
N
NE
W
SE
S
BØLGEROSE
ROTHES GRUND

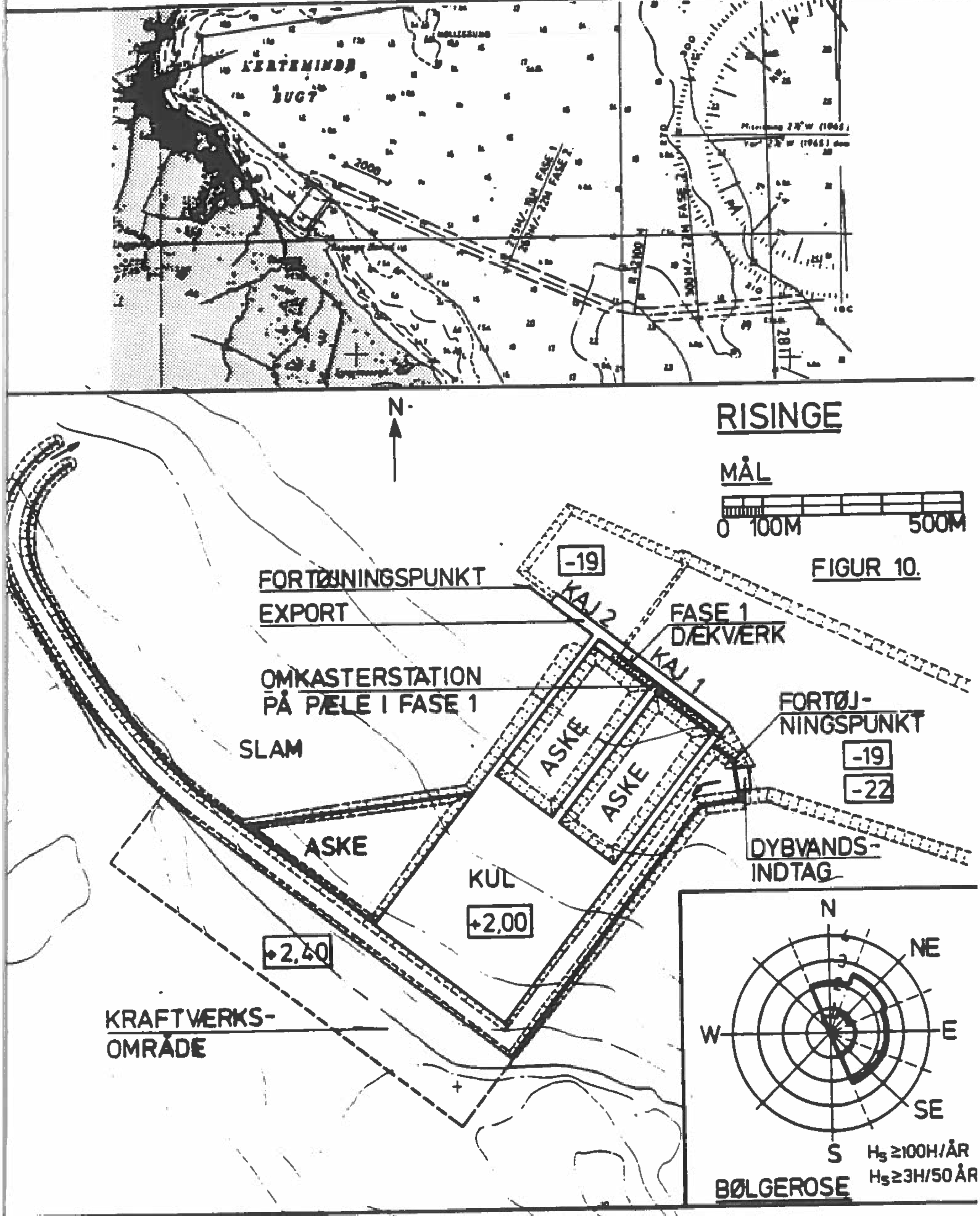


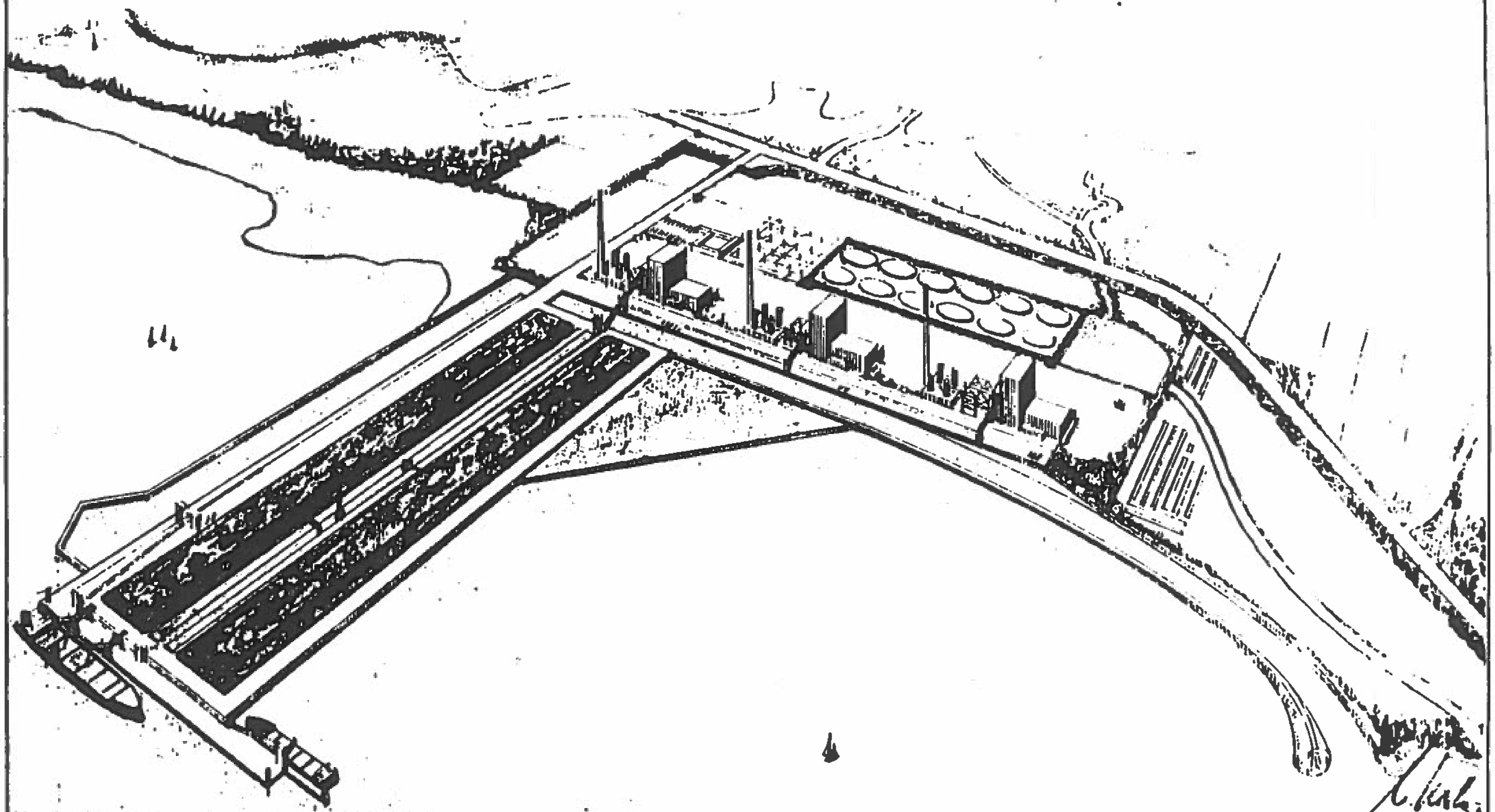
ELSAM KRAFTVÆRKSGRUPPEN	EK63. PLADSUNDERSØGELSE VY OVER ANLÆGGET KRAFTVÆRKSPLADS STRANDPLANTAGE	EK63-020.059

FIGUR 8.


EBELTOFT
VIG
SANDHAGEN
TVILLINGERNE
SKELHØJS-
GRUNDE
H J E L M
N
Gaasehage


KOBBERGÅRD
FIGUR 9.
SLAM
ASKE
ASKE
ASKE
KUL
+2,4
KRAFTVÆRKSOMRÅDE
EXPORT
FASE 1
DÆKVÆRK
-19,0
-19,0
-22,0
DYBVANDS-
INDTAG
FORTØJNINGSPUNKT
N
MÅL
0 100M 500M
N
NE
E
SE
S
W
BØLGEROSE
Hs≥100H/ÅR
Hs≥3H/50ÅR


KERTEMINDE
BUGT
N
RISINGE
MÅL
0 100M 500M
FIGUR 10.
-19
FORTØJNINGSPUNKT
EXPORT
KAJ 2
FASE 1
DÆKVÆRK
KAJ 1
OMKASTERSTATION
PÅ PÆLE I FASE 1
FORTØJ-
NINGSPUNKT
-19
-22
SLAM
ASKE
ASKE
ASKE
KUL
+2,00
DYBVANDS-
INDTAG
+2,40
KRAFTVÆRKS-
OMRÅDE
N
NE
W
E
SE
S
$H_s \geq 100H/ÅR$
$H_s \geq 3H/50ÅR$
BØLGEROSE



FIGUR 11.

ELSAM
KRAFTVÆRKSGRUPPEN

tegn	jan 79	
godk		
sat		
målestok	—	

EK63 PLADSUNDERSØGELSE
VY OVER ANLÆGGET
KRAFTVÆRKSPLADS RISINGE

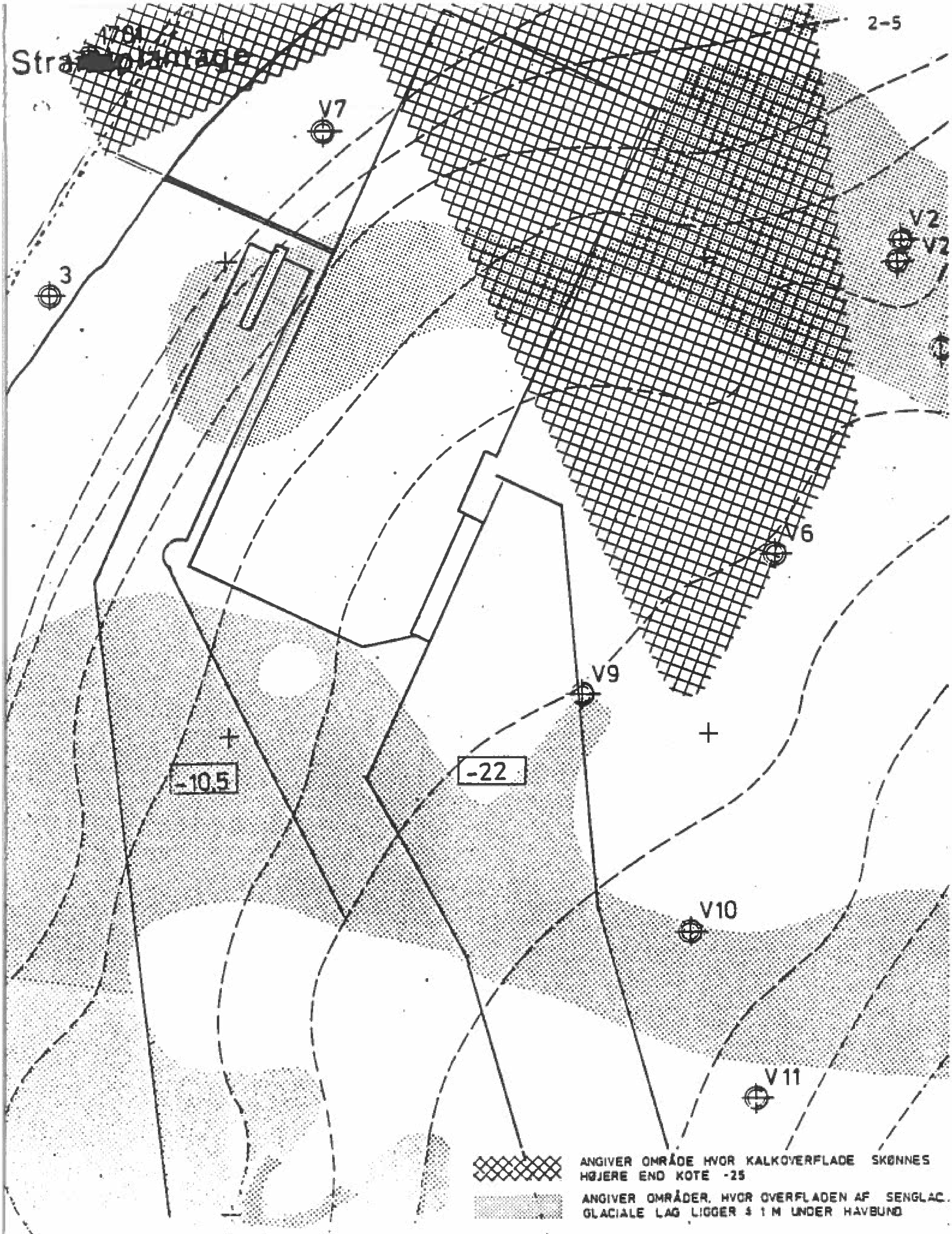
EK63-020.058

FIG. 12 ANLÆGSUDGIFTER (TAL I PARANTES IKKE SUMMERET)	FREDERIKSHAVN MKR		RUGGARD MKR		KOBBERGARD MKR		RISINGE MKR	GLATVED FORPROJ. MKR.
PRISNIVEAU, JANUAR 1979	1	3	1	2	1	2	1	1A
1. KØLEVANDSANLÆG	(0)	(0)	(85)	(56)	(49)	(49)	(62)	-
2. HAVNEANLÆG								
2.1 ANLÆG FOR 8.000 DWT. (FRH. 20.000 DWT.)	- 8	- 23	- 3	- 3	- 3	- 3	- 3	8
KAJ	8	10	3	3	4	3	3	8
DÆKMOLE	0	13	0	0	0	0	0	-
2.2 ANLÆG FOR 160.000 DWT.	- 118	- 141	- 154	- 169	- 128	- 159	- 148	135
UDDYBNING TIL -19 M/-10 M	80	53	90	92	29	57	111	63
DÆKMOLE	0	(45)	25	43	46	68	0	0
KAJ	38	43	39	34	53	34	37	72
3. AREAL UDEN FOR KYST	- 161	- 126	- 25	- 43	- 26	- 34	- 51	48
ADGANGSDÆMNING	107	82	0	0	0	0	0	
KULPLADS	54	44	18	36	20	28	40	48
ASKEPLADS	0	0	7	7	6	6	11	
KRAFTVÆRKSOMRADE	(0)	(0)	(13)	(4)	(15)	(16)	(12)	
TOTAL FASE 1	287	290	182	215	157	196	202	191
4. ANLÆG FOR 250.000 DWT.	- 131	- 145	- 135	- 134	- 99	- 103	- 217	
4.1 UDDYBNING FRA -19 M TIL -22 M	53	47	44	43	36	26	136	
4.2 DÆKMOLE	0	13	14	21	0	7		
4.3 KAJ	78	85	77	70	63	70	81	
5. AREALER UDEN FOR KYST	0	0	10	0	22	22	0	
TOTAL FASE 2	131	145	145	134	121	125	217	
TOTAL FASE 1+2	418	435	327	349	278	321	419	
INDEX FASE 1	1,82	1,85	1,16	1,11	1,00	1,25	1,29	
INDEX FASE 1+2	1,50	1,56	1,18	1,26	1,00	1,15	1,50	

FIG. 13 REVURDEREDE PLANLÆGNINGSFORUDSÆTNINGER

			FASE 1	FASE 2
- KULTRANSPORT	CA.	MT/AR	5,0	8,0
HVORAF				
- KULTRANSPORT				
· I PRAMME	CA.	MT/AR	5,0	4,0
· LANDVÆRTS	CA.	MT/AR	1,0	1,0
- KULLAGERPLADS				
· MÆNGDE	CA:	MT/AR		4,0
· AREAL	CA.	HA	35-60	60-80
- OLIELAGERPLADS	CA.	HA		15
- SKIBSSTØRRELSER				
· IMPORT	MAX.	DWT	150.000	250.000
· EKSPORT	MAX.	DWT	20.000	20.000
· FLYVEASKE	CA.	DWT		6.000
· MASKINTRANSPORTER	CA.	DWT	(3.000)	3.000
- KAJLÆNGDER OG VANDDYBDER				
· IMPORTKAJ	LÆNGDE	M	285	+ 330
	DYBDE 1A	M	19	19
	DYBDE 1B	M	22	
· EKSPORTKAJ	LÆNGDE	M	163	
	DYBDE	M	10,5	

- ANDRE FORUDSÆTNINGER
  IMPORTKAJEN SKAL HAVE ET SIDEAREAL, HVOR KULLASTEN KAN PLACERES I TILFÆLDE AF DELVIS FUNKTIONSSVIGT AF KULTRANSPORTANLÆG.



Jordbundsforhold

FIGUR 14.

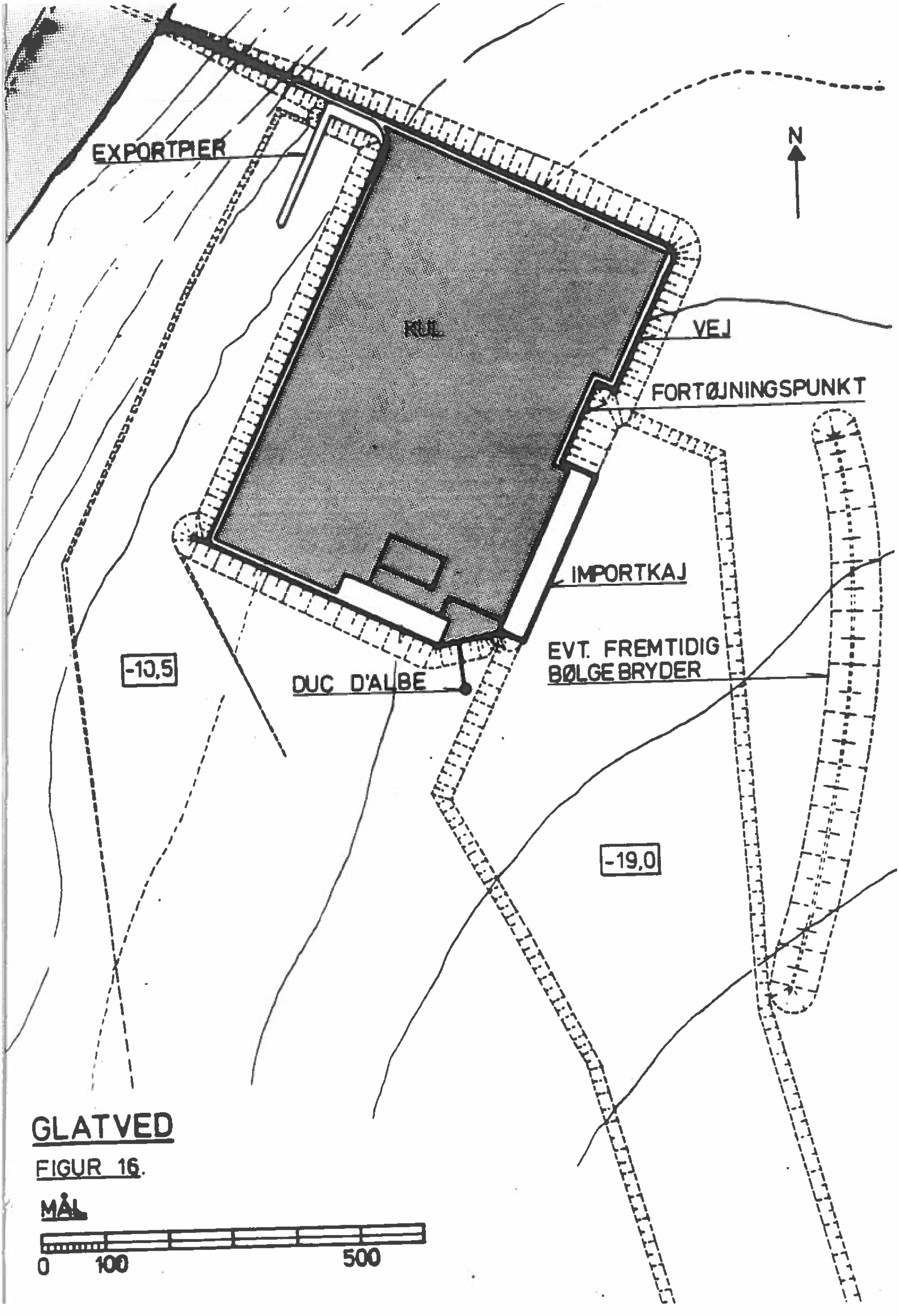
GEOTEKNISK INSTITUT

DYBDEKORT 1:1000

Å 80106 GLATVED.

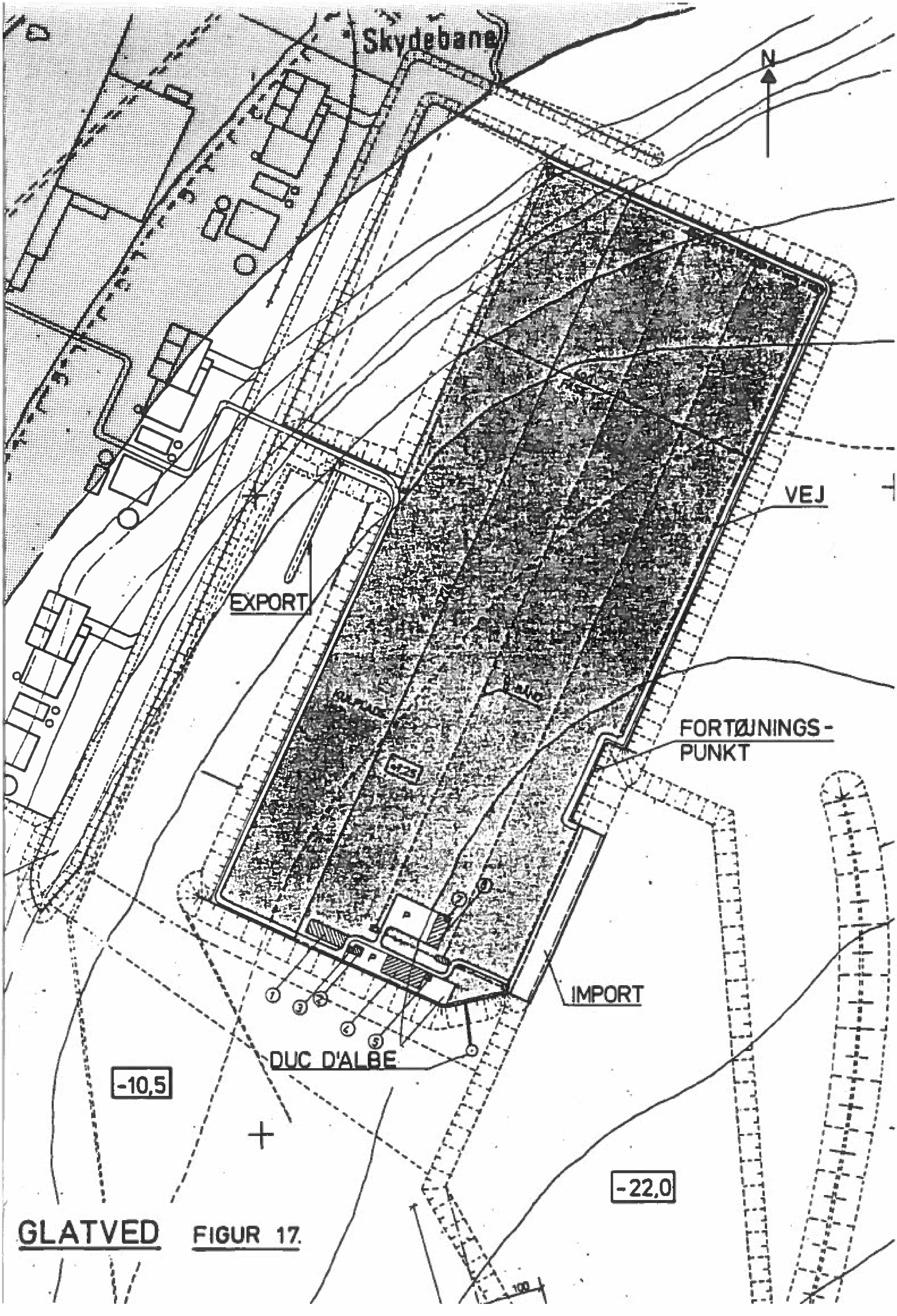
Skibsstørrelse	Antal timer pr. år, hvor losning er umulig	Antal timer pr. år, hvor skib ikke kan blive ved kaj
10.000 DWT	150	40
25.000 DWT	50	40
50.000 DWT	50	30
100.000 DWT	20	20
150.000 DWT	20	20
250.000 DWT	20	20

Fig. 15: Antal timer hvor skib ikke kan losses eller ligge ved kaj på grund af for store skibsbevægelser. Efter DHI.



GLATVED

FIGUR 16.



GLATVED FIGUR 17.

HAVNEPROJEKTERING I ISLAND FOREGÅR
I DET VÆSENTLIGE VED MODELFORSØG

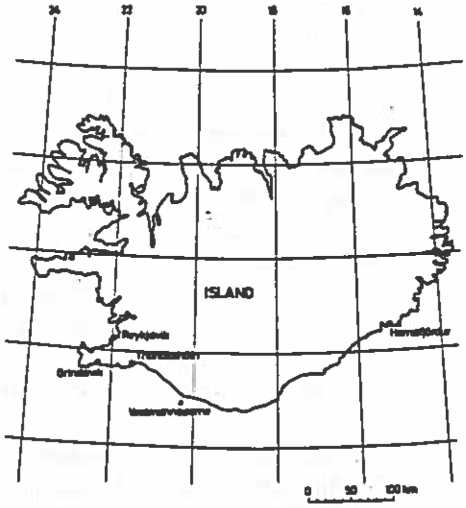
ved

civ.ing. Torben Ernst, Hostrup Schultz & Sørensen

Særtryk fra DK-Consult 4, 1974

Havneprojektering i Island foregår i det væsentlige ved modelforsøg

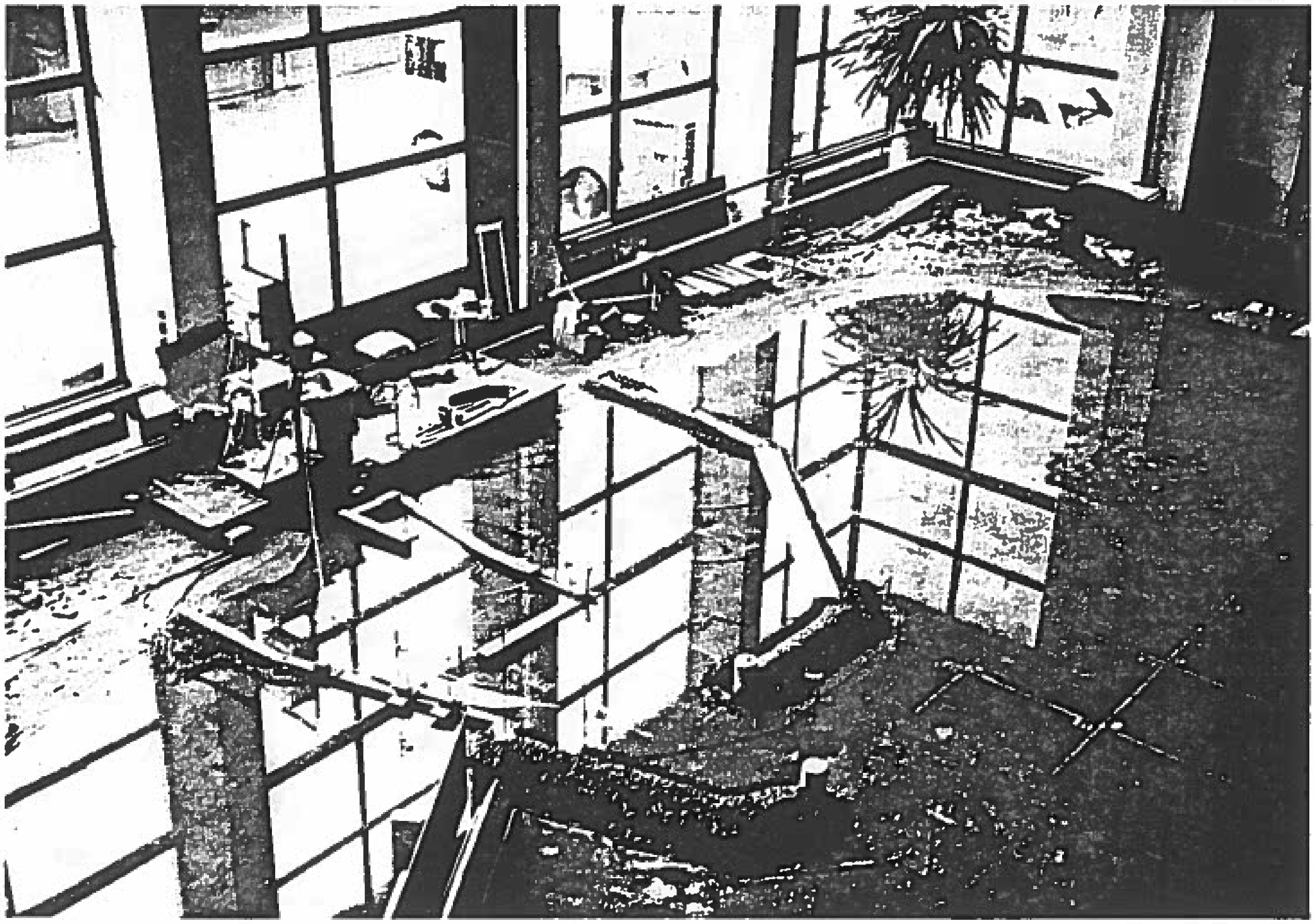
Af T. Ernst, civilingeniør, Hostrup-Schultz & Sørensen



Vulkanudbruddet på Vestmannaøerne i januar 1973 og de dermed følgende ødelæggelser var den direkte årsag til, at man i Island bestemte sig for at udbygge Thorlakshöfn og forbedre indsejlingen til Grindavik. Thorlakshöfn og Grindavik er eksisterende fiskerihavne på Islands sydkyst og beliggende i et område, der er udsat for hyppige og kraftige storme. Projekteringen er gennemført af danske og islandske ingeniører i forening. Modelforsøg og nært samarbejde med lokale myndigheder og skippere har haft afgørende indflydelse på projekterne. Til arbejdernes udførelse, der ventes igangsat sidst på sommeren, har Island opnået lån i Verdensbanken.

Fiskeri og vulkanudbrud

Fiskefartøjer, der fisker ud for Islands sydkyst, og som i sæsonen fanger en last på en dags tid, har kun få steder at lande fangsten. Helt mod øst ligger Hornafjördur, men havnen er lille og vanskelig — under urolige forhold umulig — at anløbe. Mod vest ligger Grindavik og Thorlakshöfn. Grindavik, der er den største, har en godt beskyttet havn, som imidlertid er vanskelig at anløbe fordi indsejlingen er smal og omgivet af undersøiske skær. Havnen i Thorlakshöfn er lille og yder ikke tilstrækkelig beskyttelse mod sø og dønninger. Ud for sydkysten ligger Vestmannaøerne, hvis havn anses for Islands vigtigste fiskerihavn dels på grund af den centrale beliggenhed i forhold til fiskebankerne, dels på grund af havnens kapacitet og anløbsforhold.



Belastningen på de enkelte havne varierer en del inden for sæsonen fra januar til maj. Således har det ofte været tilfældet, at fiskerbåde, der ville lande fisk f.eks. i Thorlakshöfn, har måttet vente i timevis for at komme til at losse fangsten eller ligefrem har måttet sejle til anden havn. Begge dele er lige uheldigt, fordi den medgåede tid er uproduktiv.

Der tegnede sig mørke udsigter for fiskeriet, der er rygraden i Islands økonomi, da vulkanudbruddet på Vestmannaøerne satte ind den 23. januar 1973 og varede ved til slutningen af juni. Så længe udbruddet stod på, og lavaen strømmede ned over store dele af byen og ned mod havnen og ødelagde flere af fiskeindustrierne, kunne det ikke afgøres, i hvilken grad fiskeriet og dermed økonomien ville blive ramt.

I Island blev det derfor besluttet, at en hurtig udbygning af Thorlakshöfn og en forbedring af de dårlige besejlingsforhold i Grindavik skulle gennemføres hurtigt og den islandske regering opnåede til disse formål et lån i Verdensbanken.

I slutningen af august måned 1973 engagerede regeringen Hostrup-Schultz & Sørensen, som havde etableret samarbejde med en række islandske og danske firmaer*), til at forestå projektering og tilsyn af de nævnte arbejder i nært samarbejde med de islandske havnemyndigheder. Projekterne blev udbudt i international licitation i maj dette år og licitationen er i skrivende stund endnu ikke afholdt.

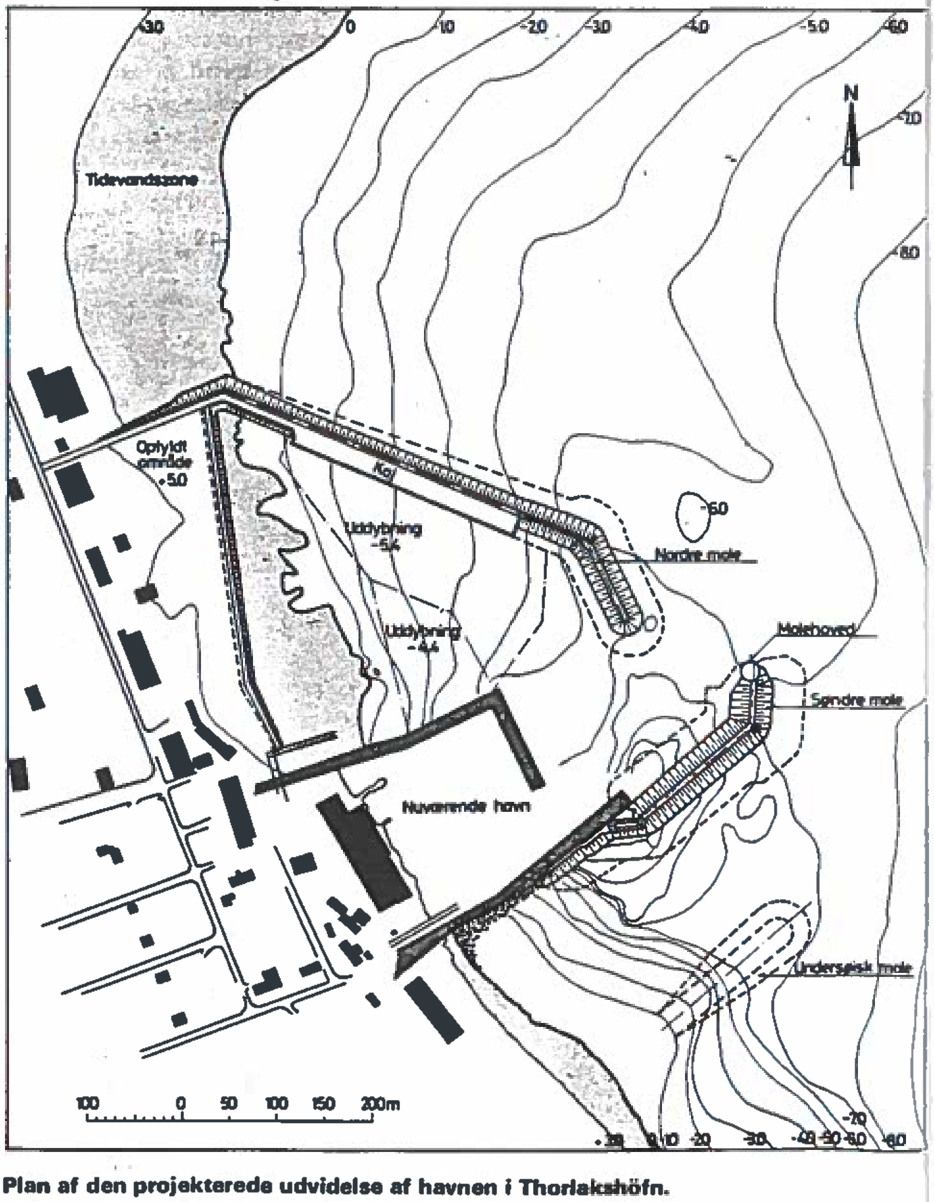
Nuværende forhold ved Thorlakshöfn

Atlanterhavet ved Island er et usædvanlig barskt farvand. Det skyldes den hyppige passage fra sydvest mod nordøst af kraftige lavtryk og dermed følgende storme kombineret med lange frie stræk.

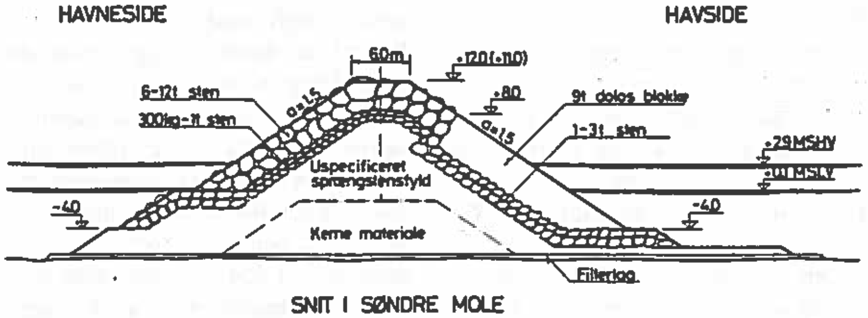
Thorlakshöfn, der ligger på vestsiden af en bred, åben bugt, ligger ubeskyttet i forhold til bølger fra dette farvand. Bugten har åbning mod syd, og syd for havnen strækker en lavapynt sig længere mod øst end selve havnen. Man skulle derfor umiddelbart forvente, at pynten ydede en vis beskyttelse mod sø og dønninger fra syd og

*) Ud over Hostrup-Schultz & Sørensen er de øvrige rådgivere Almenna verkfrædistofan hf., Verkfrædithj. Dr. Gunnars Sigurdssonar, Verkfrædistofan Fjarhitun hf. og Dansk hydraulisk Institut.

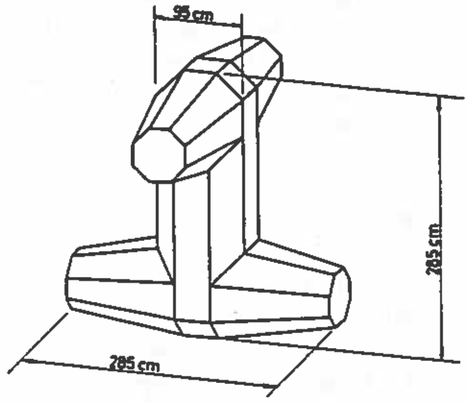
Bølgeoverskyl over nuværende søndre mole i Thorlakshöfn.



Plan af den projekterede udvidelse af havnen i Thorlakshöfn.



Snit i søndre mole. Næsten alle materialer under kote-1,0 vil formentlig blive sejlet ud med split- eller klappram. Alle stenmaterialer derover køres ud fra land og dels tippes, dels udlægges med kran.



Dolos blok af uarmeret beton. Der skal støbes ca. 2300 sådanne blokke. ▶

sydvest. Imidlertid viser både feltobservationer og teoretiske beregninger (refraktionsberegninger), at selv om bugten kun er geografisk åben fra sydsydøst til sydøst er den hydraulisk åben fra sydvest til sydøst. Det er for øvrigt karakteristisk for beliggenheden af Thorlakshöfn, at uanset hvor søen på dybt vand kommer fra mellem sydvest og sydøst vil bølger, der rammer havnen, have en forplantningsretning inden for et snævert vinkelrum af ca. 20° stort set vinkelret på den eksisterende søndre pier.

I området ud for såvel den nuværende som den kommende havn er vanddybden ca. 8 m målt fra middelspringtidslavvande og tidevandet er ca. 3 m. I dette område er maksimalbølgehøjden bestemt af vanddybden, idet bølger større end en vis størrelse bryder som følge af friktion mod bunden. Det kan således ikke undgås, at fiskefartøjer, der søger ind til den nuværende eller den nye havn under stærke storme, må passere et område, hvor der er risiko for grundbrydende bølger.

Den eksisterende havn består kun af ét bassin på ca. 170 × 120 m omgivet af pierer af betonsænkekasser. Under storm er der betydelig bølgeuro i havnen til stor gene for de fortøjede fiskefartøjer, der ofte af pladsnød ligger flere uden på hinanden. Det er ikke ukendt, at de fartøjer, der ligger på de mest urolige pladser ved optræk til dårligt vejr foretrækker at stikke til søs frem for at blive liggende og risikere at blive beskadiget. Har man imidlertid set forholdene under storm, er man imponeret over, at en så lille havn alligevel kan yde den grad af beskyttelse, den gør.

Opgaven var at udarbejde en dispositionsplan for den fremtidige udbygning af havnen i Thorlakshöfn og at projektere første etape af denne udbygning bestående af moler og kajer, alt under hensyn til de beskrevne forhold.

Projektering i et vandbassin

Selve projekteringen af en havn under nævnte forhold sammenlignet med anden projektering foregår i langt højere grad i et vandbassin iført gummistøvler end ved et tegnebord. Modelforsøg, modelforsøg og atter modelforsøg har været grundlaget for dette projekt, og det er helt givet, at projektet ikke kunne gennemføres alene på grundlag af lærebøger og formler og slet ikke uden at indbygge sådanne grader af sikkerhed, at projektet var blevet urimelig dyrt.

I modellerne — der er udført flere forskellige med henblik på undersøgelse af forskellige forhold — reproduceres de faktiske forhold, såvel de topografiske som de bølgemæssige, med størst mulig nøjagtighed.

En tredimensional model i målestok 1:100 blev opbygget af den eksisterende havn og den omkringliggende del af bugten. I denne model blev en lang række udformninger af den nye havn visuelt vurderet med hensyn til bølgeuro i bassinerne, og på grundlag heraf valgtes de få principielle udformninger, der skulle nærmere undersøges. Disse blev bedømt på grundlag af målinger af bølgehøjderne forskellige steder i havnen. I modellen af den til slut valgte principielle udformning af havnen undersøgtes endelig betydningen af ændringer af forskellige parametre, så som havnemundingens bredde, molernes længde, mindre vinkeldrejninger af moler og kajers placering og retning. Virkningen af parameterændringer fandtes dels ved at måle bølgehøjder i havnen, dels ved at måle fortøjningskræfter på korrekt ballasterede modeller af fiskefartøjer og fragtskibe som vides at ville benytte havnen.

Den heldige gennemførelse af den beskrevne forsøgsrække skyldes i høj grad hyppige besøg af lokale skippere, der kender de lokale forhold efter mange års fiskeri fra havnen. Disse folks hjælp er uvurderlig, når modellen køres ind, og det kontrolleres, at det er de faktiske forhold, der reproduceres. Dernæst kan skipperne bedre end andre gøre rede for de besejlingsmæssige konsekvenser af de forskellige udformninger, der prøves i modellen. Endelig er det af betydning for projektets modtagelse i lokale kredse at velrenommerede repræsentanter har set »disse her modeller« og kan berette at »det der laves i modellen virkelig ligner, det der foregår i havnen og på havet udenfor.«

Af de forskellige udformninger, der blev undersøgt, samlede interessen sig naturligt om de, der forbedrede forholdene i og omkring den nuværende havn, således at de allerede eksisterende konstruktioner i højere grad kan udnyttes, end det er tilfældet idag. Det viste sig, at en udformning svarende til en forstørrelse af den eksisterende havn vil være den mest hensigtsmæssige inden for det beløb, der er til rådighed.

På et tidligt tidspunkt stod det klart, at sænkekassemoler i de store dimensioner, der her er tale om ikke ville være hverken praktisk eller økonomisk gennemførlige, og man valgte derfor at gennemføre projektet med stenkastningsmoler.

Tværsnittet i den søndre mole, der er langt den hårdest udsatte, blev dimensioneret i en 60 cm bred bølgerende både under hensyntagen til stabilitet og til bølgeoverskyl. Som den mest økonomiske valgte man blandt mange udformninger en mole med kronen 12,0 m over middelspringtidslavvande og kastninger af henholdsvis 9 t dolos blokke på havsiden og af 6-12 t sten på havnesiden.

I en ca. 4 m bred bølgerende blev den søndre moles tilslutning til eksisterende konstruktioner og til molehovedet undersøgt, da der begge steder gør sig særlige forhold gældende. Herunder blev der konstateret en risiko for ud-skæring i bunden foran molehovedet, såfremt en bundsikring ikke udføres.

Også den nordre mole blev omhyggeligt undersøgt i 4 m renden. Dette med henblik på at »klemme« tværsnittet så meget, som det var forsvarligt muligt. Det var nemlig fra begyndelsen klart, at bølgeangrebenes styrke ville variere stærkt langs molen, fordi bølgerne kun angriber fra en bestemt retning.

Det skal nævnes, at der ved alle modelforsøg er anvendt uregelmæssige bølger. Bølgeregistreringer fra naturen er overført til hulbånd. Når disse afspilles, overføres de afgivne impulser ved hjælp af elektronisk udstyr til en hydraulisk bølgemaskine, som i modellen reproducerer i korrekt målestoksforhold de i naturen registrerede bølger. Denne teknik, der er ganske ny, er blevet udviklet af Dansk hydraulisk Institut.

For at få mere end blot relative resultater ud af de mange modelforsøg, der blev udført, er det nødvendigt at vide, hvor hyppigt den eller de situationer af vandstand og bølger, der reproduceres i modellen, overskrides i naturen. I den henseende var man ved projektets start heldigt stillet, idet der hos de islandske havnemyndigheder fandtes dels nogle amerikanske bølgemålinger foretaget på dybt vand uden for Thorlakshöfn, dels nogle målinger af vandstande inde i havnen. Disse observationer blev suppleret med målinger af kort- og langperiodiske bølger og tidevand inde i havnen udført af de islandske havnemyndigheder i efteråret 1973 og vinteren 1974. På grundlag af de forskellige registreringer blev det muligt blandt andet via teoretisk beregning af bølgeforhold under tidligere, kendte storme korreleret med eksisterende bølgestatistik fra vejrskibe i Atlanten nogenlunde at bestemme, hvor hyppigt farlige kombinationer af bølger og vandstand statistisk vil optræde.

Som kriterium for acceptable forhold for fortøjede skibe valgtes, at den maksimale fortøjningskraft ikke må overstige ca. 50% af fortøjningens brudstyrke. Det er en situation, der beregningsmæssigt vil overskrides hvert 5. år. Under de nuværende forhold overskrides den samme situation ca. 10 dage pr. år. For molernes vedkommende valgtes som kriterium for, at de kan regnes stabile, at skaderne ikke vil overstige 1,5-2% for bølge- og vandstandsforhold, der beregningsmæssigt vil overskrides hvert 50 år.

Sten

Et meget vigtigt led i hele projekteringen var en undersøgelse af, om de nødvendige mængder af sten til brug for molebyggeriet kunne skaffes i de rigtige størrelser og til rimelige priser. Ialt skal der bruges godt 600.000 t stenmaterialer.

I den sydvestlige del af Island består alt fjeld af lava. Det meste er dannet under eller efter istiden. Det er altså ungt fjeld, og det viser sig at være meget sprækket. Geologer, der undersøgte omegnen omkring Thorlakshöfn, påpegede to lokaliteter som mulige ressourcer for stenmaterialer.

Den ene var en fjeldknold, der forekom lidt mere modstandsdygtig end det omkringliggende fjeld, og som ligger 6,5 km fra havnen. Som et led i projekteringen åbnedes her et stenbrud, hvor 1.400 m^3 blev brudt. For hver af de ialt 4 salver, der blev skudt, blev stenmaterialet sorteret, og vægten af alle sten over 1,5 t skønnet. Ved vejning af ca. 8% af disse sten blev det konstateret, at der var god overensstemmelse mellem skønnet og målt vægt. Resultatet af undersøgelsen viste, at mens 13% af det brudte stenmateriale gav sten, der vejede mere end 3 t, gav kun 1,5% sten, der vejede mere end 9 t. Et magert resultat, da man gerne havde set sprængsten, der vejede både 20 og 30 t.

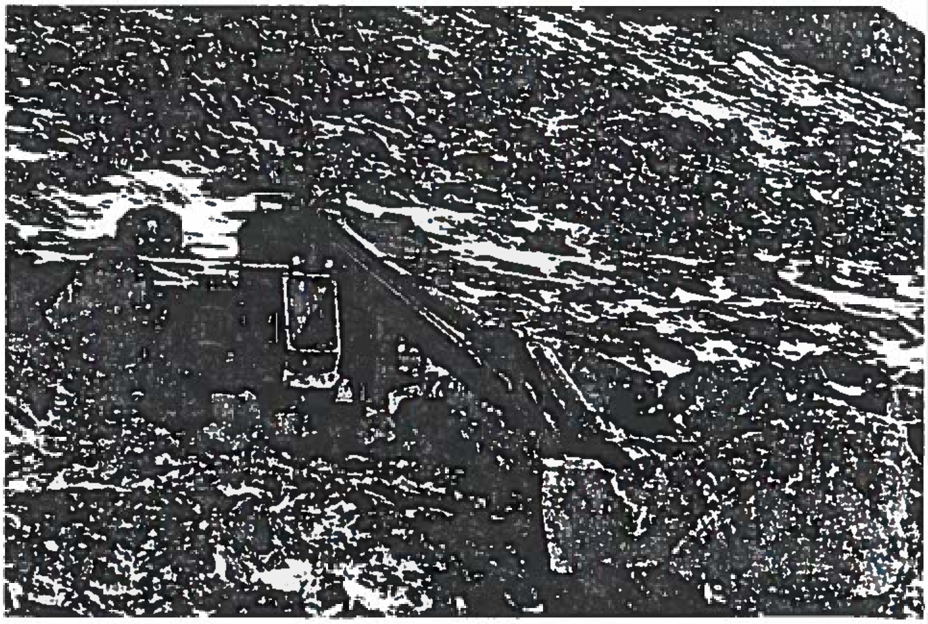
Med hensyn til maksimalstenstørrelsen var det ikke mere opmuntrende på den anden lokalitet, der blev undersøgt. Her findes, ca. 13 km fra havnen, et stejlt lavafjeld, hvis fod er dækket af ur hovedsagelig bestående af sten. Uren er 40 m høj og dens overflade danner ca. 30° med vandret. En gennemgravning af uren til belysning af de stenstørrelser, der her fandtes, kunne ikke gennemføres, da der manglede både tid og penge. Derfor nøjedes man med på et 1.500 m^2 stort areal af overfladen at registrere sten, der skønnedes at veje mere end 4 t. Kun 15 af disse vejede over 10 t.

Ved prøvegravninger på det flade land foran uren fandtes forbavsende mange vandskurede sten, der gennemgående vejede 2-10 t. Disse sten stammer fra en periode, da havet stod højere og bearbejdede og nedbrød det nævnte lavafjeld.

På baggrund af de udførte undersøgelser skønnedes det, at de nødvendige sten under 10 t kan skaffes ved kombineret udnyttelse af de to undersøgt ressourcer.

Dolos blokke

Da det viste sig, at sten af størrelsesordenen 20-30 t ikke kunne skaffes, måtte kunstigt fremstillede sten tages i brug. Man kunne sådan set have fremstillet 20-30 t tunge kubiske betonblokke og have lagt dem ud som sten. Det er imidlertid sære uøkonomisk. Derfo benyttede man sig af udenlandske erfaringer og bestemte sig for at udlægge dolos blokke af uarmeret beton. Disse blokke har facon som et vredet H og lægges — ikke i mønster — men tilfæl-



Prøvegravning efter sten til molebyggeri i foden af ur ca. 13 km fra Thorlakshöfn.

digt på molens skrå side. I modsætning til sten eller kubiske betonblokke, der i det væsentlige opnår deres stabilitet i molen som følge af deres vægt, opnås stabiliteten i en dolos kastning i det væsentlige på grund af den naturlige sammenlåsning, der finder sted, når disse specielt formede elementer udlægges. Dette er årsagen til, at vægten af de enkelte blokke kan reduceres til de 9 t. Selv om hulrumsprocenten i dolos-kastningen er nær ved 60% skal der udstøbes ca. 9.000 m³ beton i dolosblokke.

Da det stod klart, at det var nødvendigt med dette store betonarbejde iværksattes hos det islandske institut for undersøgelse af bygningshåndværk prøvning og analyse af fire mulige forekomster af betongrus. En af disse forekomster var sømaterialer. Ud over bjælke- og trykstyrkeprøver koncentrerede undersøgelserne sig om at fastslå risikoen for alkalireaktioner og om at fastslå graden af holdbarhed over for vekslende påvirkninger af frost og tø. Kun en af de fire forekomster fandtes tilfredsstillende, og den ligger ca. 120 km's kørsel fra den kommende byggeplads.

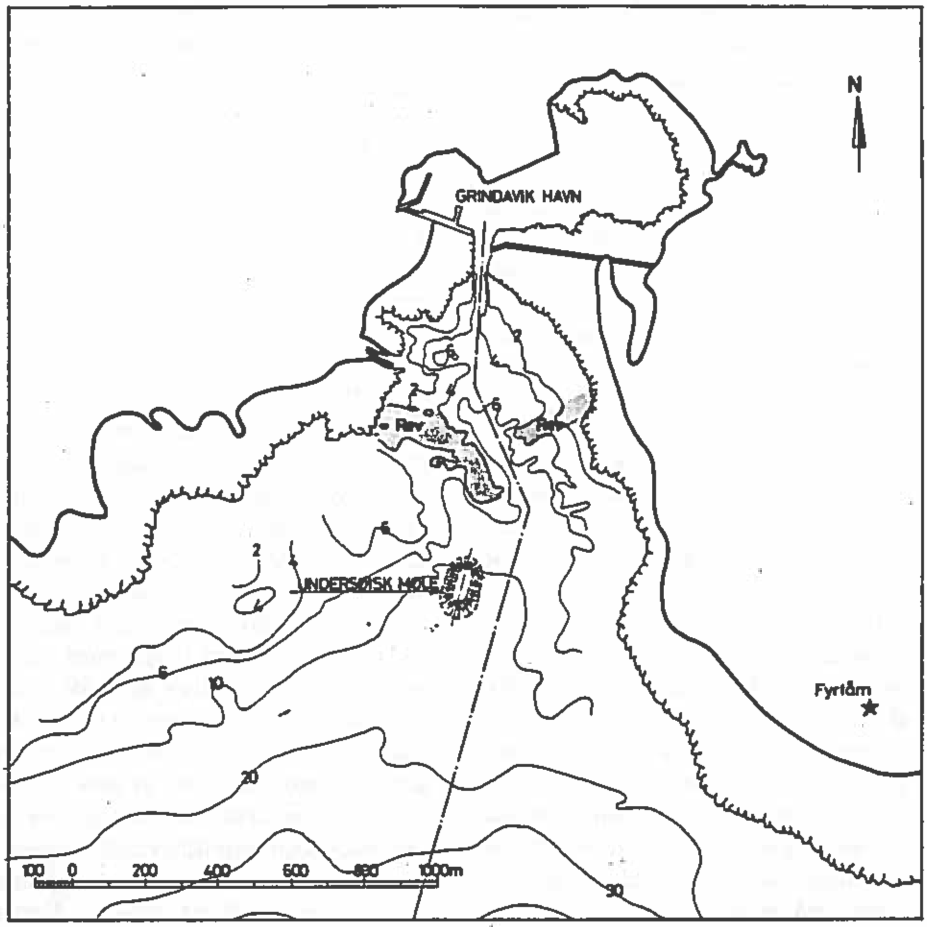
Alle prøvestøbninger udførtes med nedknust lava fra prøvestenbruddet som grove tilslagsmaterialer.

Øvrige undersøgelser

Ud over en lang række hammersonderinger, der allerede var udført på vandområdet nord for den nuværende havn af de islandske havnemyndigheder, blev geotekniske og geologiske undersøgelser udført dels af et islandsk, dels af et amerikansk firma.

Det islandske firma, der er specialiseret i forundersøgelser for vandkraftanlæg, har uden tvivl bedre baggrund end andre firmaer for at kunne undersøge og redegøre for de specielle geologiske forhold i Island. Så at sige al jordbund består af lava eller andre vulkanprodukter og af jordarter dannet af nedbrudte og bearbejdede vulkanprodukter. Særligt i randene af gamle lavaflomme er forholdene ganske regelløse, og medmindre man har erfaring på området, kan man blive overrasket. Således er det f.eks. ikke ukendt, at hvad man anser for at være kompakt og bæredygtig lava viser sig at være et relativt tyndt lavalag underlejret af løse sedimenter.

Det amerikanske firma er geofysikere og råder blandt andet over et ekkolod med så stor gennemslagsstyrke, at signalet kan trænge igennem sedimenter og reflekteres af eventuelle klippeformationer.



Plan over indsejling til Grindavik.

Resultatet af de to firmaers undersøgelser var, at der findes lava inden for en linie, der stort set falder sammen med kystlinien ved middelspringtidslavvande. Uden for denne linie findes sand underlejret af et lag fastere sedimenter, der i det væsentlige formentlig består af sten. Dette lags overflade falder bort fra kysten, og hyppigheden af sten i dette lag forventes at aftage med afstanden fra kysten.

En del af en projekteret spunsvæg, der danner indfatning for kajen, skal rammes ned i det nævnte stenlag, ligesom en lille del af den projekterede uddybning nødvendigvis må foretages i samme lag. Egentlige geotekniske boringer herfor er henlagt til denne sommer fordi på det tidspunkt, hvor placeringen af spunsvæg og uddybning var fastlagt, var det af vejrmæssige grunde utænkeligt at udføre boringer eller sonderinger på vandet. I udbudsmaterialet er indbygget visse muligheder for efter arbejdets påbegyndelse at forskyde den projekterede kaj, hvis det viser sig hensigtsmæssigt.

Projekteringen omfattede også en undersøgelse og en vurdering af, hvor mange folk entreprenøren vil anvende på pladsen, og hvordan disse folk kan indkvarteres i nærheden af Thorlakshöfn, der kun har 600 indbyggere. De projekterende anbefalede i den forbindelse, at bygherren opfører en arbejdslejr til ca. 30 personer, som kan stå klar samtidig med, at entreprenøren starter. Hermed vil entreprenøren få mulighed for at komme tidligere igang med det egentlige havnebyggeri.

Molehoved

Molehovedet for enden af den søndre mole var under projekteringen genstand for usædvanlig mange overvejelser på grund af de risikofyldte forhold, det skal bygges under. Et molehoved på dette sted er nødvendigt alene af hensyn til molens stabilitet, medmindre molen var blevet afsluttet med en kegle bygget op af dolos blokke væsentlig tungere end de, der skal anvendes på molens regulære strækninger. Molehovedet tjener desuden til markering af hvor tæt til molen, der kan sejles, samt som et muligt punkt, skibene kan falde af på i kritiske situationer.

Molehovedets udstrækning i planen er bestemt af molens højde, idet molens skråninger skal kunne løbe af langs molehovedets omkreds uden at nå frem på forsiden og derved reducere vanddybden. Med et cirkulært molehoved fandtes en diameter på 20 m at være absolut minimum. Molehovedet står på 7 m vand og det er 18 m højt for at nå op til overkant mole, der på dette sted er lidt lavere, end hvad der ellers er gældende for søndre mole.

De traditionelle konstruktioner, nemlig en sænkekasse af jernbeton eller et stenfyldt molehoved af spredte, rammede pæle blev forkastet, dels fordi der ikke i nærheden fandtes egnede byggepladser for en sænkekasse, dels fordi der under de givne barske forhold er så megen risiko forbundet med anvendelse af de traditionelle metoder, at prisen skønnedes at ville blive uacceptabel høj.

Ifølge det endelige projekt skal molehovedet udføres som en cirkulær, stenfyldt tømmerkiste med lodrette stokke af tropisk træ og vandrette udvendige trækbånd af pladestål. Tømmerkisten udføres i to dele, en underpart og en overpart. Begge parter tænkes bygget på land på en mindre bedding. Under transport fra bedding til endelig placering må begge parter forsynes med afstivning og passende opdrift. Når underparten først er placeret på et stenlag med overfladen ca. 1,0 m under den omkringliggende bund, når overkanten ikke højere op, end at en klap- eller splitpram kan sejle hen over underparten og tømme sit indhold af sten ned i tømmerkisten. Herved sikres en billig og hurtig fyldning af den nederste del, hvorefter den underste del af molen kan bygges frem til molehovedet. Overparten skal placeres direkte oven på underparten, hvilket stiller store krav til løfte- og/eller opdriftarrangement. Stenfyldningen af den øvre part tænkes at ske direkte fra enden af den næsten færdigbyggede søndre mole, hvorved en nogenlunde hurtig stenfyldning kan påregnes, idet stenmaterialerne kan transporteres frem landværts.

Selve placeringen af den øvre part anses for den vanskeligste del af arbejdet med molehovedet og den periode, stenfyldningen af den øvre del varer, anses for den mest risikofyldte.

Grindavik

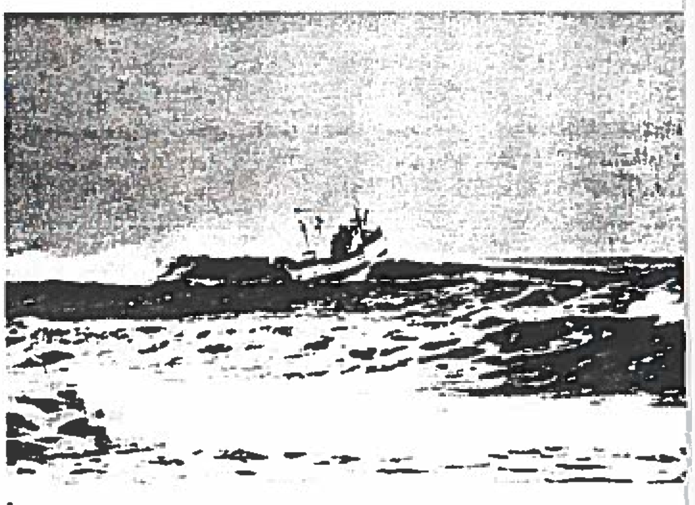
For Grindaviks vedkommende var den stillede opgave at undersøge mulighederne for forbedring af havnens besejlingsforhold ved hjælp af uddybning, afmærkning og/eller bygningen af eventuelle molekonstruktioner.

Grindavik havn ligger godt beskyttet i bunden af en tragtformet bugt, der er knap 2 km dyb og godt 2 km bred ved åbningen, der vender mod syd. Fra hver af bugtens to bredder udgår et undersøisk rev af lava. Vanddybden over disse rev er kun 3-4 m ved middelspringtidslavvande. Revene er således beliggende, at besejlingslinien har to knæk på 30-40°. Under storm fra sydvest eller syd bryder bølgerne både over det yderste rev og ud for spidsen af dette, i området ved den første drejning fiskerbådene skal igennem på vej ind. Refraktionsforholdene i området er således, at under storm fra sydvest til sydøst falder bølgernes forplantningsretning stort set sammen med indsejlingslinien uden for det yderste knæk. Bølgerne kommer i grupper, og mens næsten alle bølger bryder på det yderste rev, bryder kun de største bølger i området ved det yderste knæk. I sådanne situationer holder fiskerskipperne nøje øje med bølgegrupperne og forsøger at passere den første drejning mellem to grupper for at undgå de brydende bølger.

Fra starten stod det klart, at resultater kun ville kunne opnås gennem intensive studier i en tredimensional model. En sådan byggedes i København og en lang række forsøg er udført under daglig ledelse af en islandsk ingeniør udlånt af de islandske havnemyndigheder. Denne kombination af grundigt kendskab til de aktuelle og lokale forhold og udnyttelsen af avanceret modelforsøgsteknik har givetvis haft afgørende betydning for opgavens løsning.

Sammenlignet med modellen for Thorlakshöfn var de navigationsmæssige problemer i denne model langt vigtigere. Derfor har det været af meget stor betydning, at flere af fiskeskipperne fra Grindavik gentagende gange har besøgt modellen og studeret den i timevis. Under disse besøg har man diskuteret hvilke forhold, der påvirker besejlingen. Er det indsejlingsretningen i forhold til bølgeretningen, er det antallet, størrelsen eller frekvensen af brydende bølger, er indsejlingen for smal og ikke dyb nok eller er det drejningerne i indsejlingen, der skaber vanskeligheder, o.s.v. Man nåede på den måde frem til, at det væsentligste var at få nedsat antallet af brydende bølger i området ved det yderste knæk, og dette blev så valgt som parameter ved vurderingen af forskellige forslags effektivitet.

Uddybning i enderne af de to rev ved bortsprængning af lava viste sig at være både kostbart og lidet effektivt, og bygning af traditionelle moler til dækning af det udsatte område viste sig at ligge langt uden for de økonomiske rammer, der var stukket af.



To situationer af fiskerbåds indsejling til Grindavik under hårdt vejr.

Forsøg med forskellige placeringer af undersøiske moler, eller kunstige rev om man vil, viste derimod gode resultater samtidig med at de økonomiske rammer kunne holdes. Som resultat har man bestemt sig for at bygge en ca. 100 m lang og ca. 40 m bred undersøisk mole med kronen 1 m under middelspringtidslavvande. Molen placeres vest for indsejlingen og virker primært ved at bølgerne drejes mod vest og derved svækkes i det område, hvor der skal sejles og sekundært ved at bølgerne brydes. I det yderste knækpunkt af indsejlingen reduceres antallet af brydende bølger til en trediedel.

Med indførelsen af et kunstigt rev introducerer man et risikomoment, idet der naturligvis ikke kan sejles hen over revet. Ingeniørerne kan her beskrive og vurdere det kunstige revs fordele, nemlig reduktionen af brydende bølger, men det er fiskeskipperne, der må vurdere den forøgede risiko i navigationsmæssig henseende. Og den vurdering faldt altså ud til forslagets gunst.

Projekteringen

Samarbejdet mellem islandske og danske ingeniører blev etableret med de aktuelle sager for øje, og vi føler fra vor side at dette samarbejde har været til ubetinget fordel for bygherren og projektet.

Der har undervejs været en lang række problemer, som i væsentligt omfang er løst på grundlag af de erfaringer, de islandske ingeniører sidder inde med. Det drejer sig specielt om alle spørgsmål, hvor lavaen har været inde i billedet, spørgsmål vedrørende betonfremstilling af de lokale materialer, tolkning af de islandske normer og bestemmelser, vurdering af arbejdsmarkedet, udregning af overslagspriser og opstilling af betingelsernes prisreguleringsmekanismer under hensyn til risikoen for devaluering og til en stedse kraftig inflation. Hvad det sidste angår har man som dansk ingeniør tilsyneladende lært noget, der kan være nyttigt under de hjemlige forhold.

Bortset fra modelforsøgene udførtes projekteringen indtil jul i Island. Tre danske ingeniører var permanent i Island i efteråret. Efter nytår fortsattes projekteringen i København og i en periode var tre islandske ingeniører i København for at medvirke ved færdiggørelsen.

TEKNISK FORLAGS TRYKKERI

TESTING OF MARINE STURCTURES IN HYDRAULIC MODELS

by

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Testing of Marine Structures in Hydraulic Models

Planning and design of marine structures involve a larger number of engineering disciplines than most other structures. In many cases, the processes are so complex that the consequences of simplifications introduced in order to treat the problem theoretically are not easily detected.
This is due to the three-dimensionality of the problems - both of structures and of force fields - and due to the dynamics.

Consequently the engineers traditionally make use of an analogue representation of the problem in the form of a physical model. With the advance of large computers and refined numerical methods mathematical models are now also available for description of complex design problems and will gain in importance in the years to come.

Hydraulic modelling aims at describing the response of structures to external forces through a description of the environmental conditions. DHI regards it as a corollary of this objective that the natural conditions must be reproduced as faithfully as possible, i.e. all relevant proporties must be represented in the model. Furthermore a measuring system which is capable of determining the parameters of interest for the subsequent design procedure must be applied. In this way, it is possible to use straightforward and easily understandable criteria instead of 'black-box' criteria and thereby explain the important physical properties instead of using rough empirical relationships.

A typical example of this is wave disturbance tests in which it has been found relevant to use very exact representations of wave fields, including both short and long periodic wave energy, and to describe conditions along the wharves in terms of simple practical parameters such as mooring forces and vessel movements. With the development of electronics and data processing it has become possible to measure, and analyse on-line, a considerable number of the parameters involved. With these aids a more reliable resolution of the complex problems is obtained.

We do not consider models as true representations of nature. Physical and mathematical models are more or less simplified representations of full-scale problems and therefore they do not, in all aspects, represent nature. The inherent scale and model effects may, in some cases, be of such significance that only comparisons with full-scale behaviour can give us a true picture of the importance of these effects.

How accurately do we have to reproduce the natural conditions in our models? This depends, of course, on the problems we have to describe. The hydraulic engineer has to find the balance between cost and accuracy, which provides a satisfactory reliability. This balance is not always the same and the gradual improvement of measuring technology, the increasing complexity of prototype structures, and the increasing consequence of damage tend to displace the balance towards more refined methods. We have to improve our modelling techniques in order to gain further knowledge. Improvement of our modelling capability is a prerequisite for a better understanding of the natural processes. In some cases we can compare with full-scale observations and thereby get an impression of the reliability of the methods applied.
In most cases, the possibility of verification does not occur until years after testing. Our only course is then to ensure that the modelling is soundly based on techniques which have been verified in the past and are properly related to the laws of physics.

Modelling is not just a question of representing known full-scale properties. Models are applied to study design aspects, and in this context it is necessary to deal with long-term design conditions which are extrapolated from known daily or yearly conditions. This means that the theories of statistics must be combined with knowledge of the prototype and of the model techniques. If any of these components fail, the model results may become invalid. The conclusion is therefore that the best possible results will appear when all elements of the analysis are compatible and planned as one operation with thorough understanding of all relevant natural processes involved.

The present issue of Danish Hydraulics aims at describing the methods currently in use at DHI within these fields.

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Hydraulic Port Design

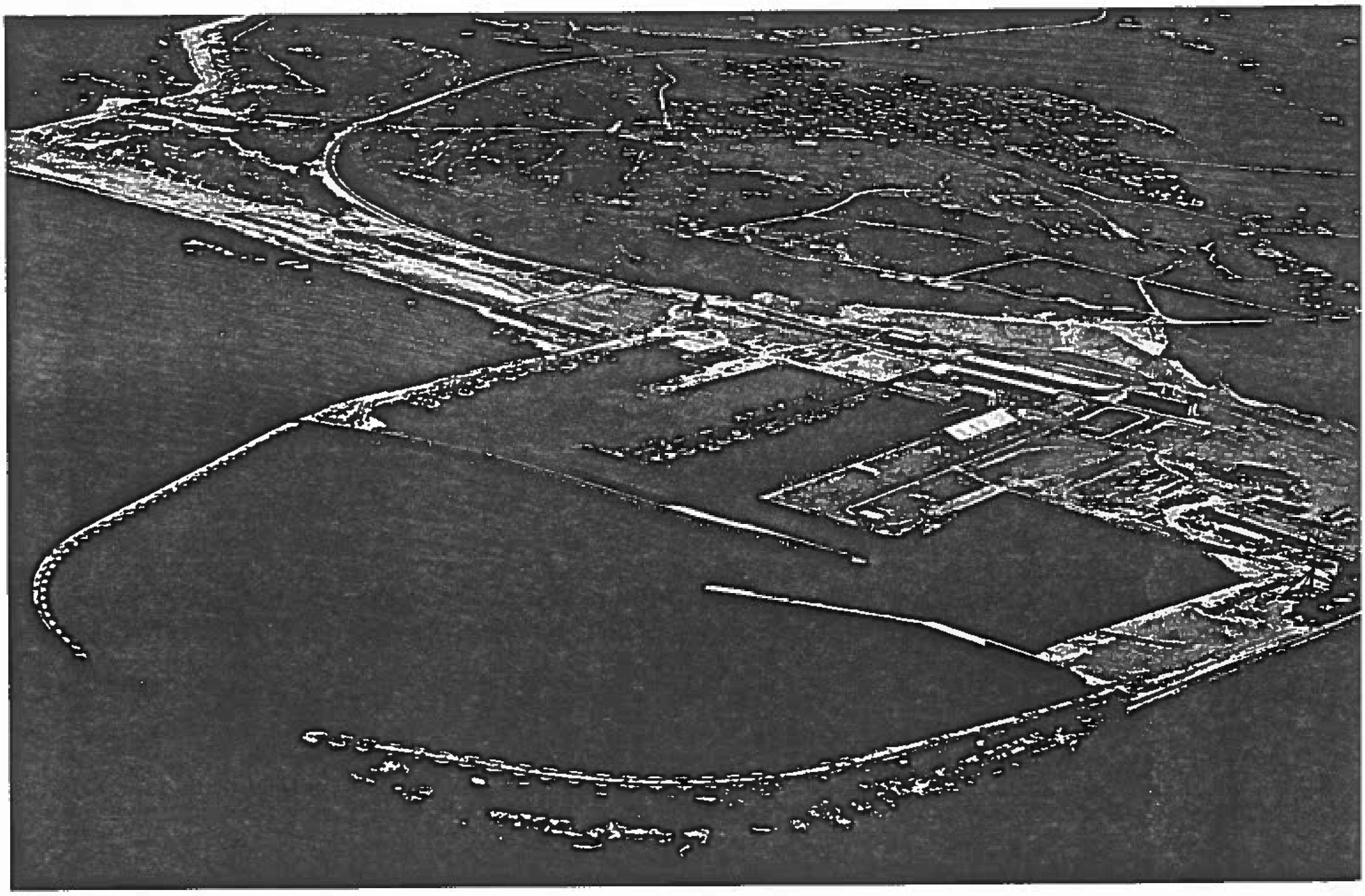


Fig. 1

The environmental conditions at the harbour site play an essential role in the planning and design of the proposed facility. The consideration of these factors in relation to the development of a new port or marine terminal are here contained in the term 'hydraulic port design'.

Hydraulic port design comprises the following basic subjects:

- marine environment, i.e. climate and hydraulic and sediment transport processes
- requirements for the loading systems and mooring arrangements for the proposed types of ships and cargo
- definition of alternative facilities capable of fulfilling the requirements in the particular environment
- optimization of the project considering the environmental conditions and operational requirements

These subjects deal only with the hydraulic part of the port design. The harbour designer must, however, consider a number of other factors, for instance, soil conditions, infrastructure and construction materials.

The above subjects are important for the location and layout of the harbour and thereby for the economy of the project. However, the hydraulic factors, and hence the hydraulic design, are essential both for the safety of vessels during navigation and at the quayside and for cargo handling. Safety as well as operational availability are key elements for the port users and therefore for the port owner.

The activitites included in the hydraulic design are conducted by hydraulic laboratories specialized in port problems. It must be emphasized, however, that close cooperation and

Fig. 1. Hanstholm Harbour.

Fig. 2 a, b and c. Wave measurements by surface buoy.

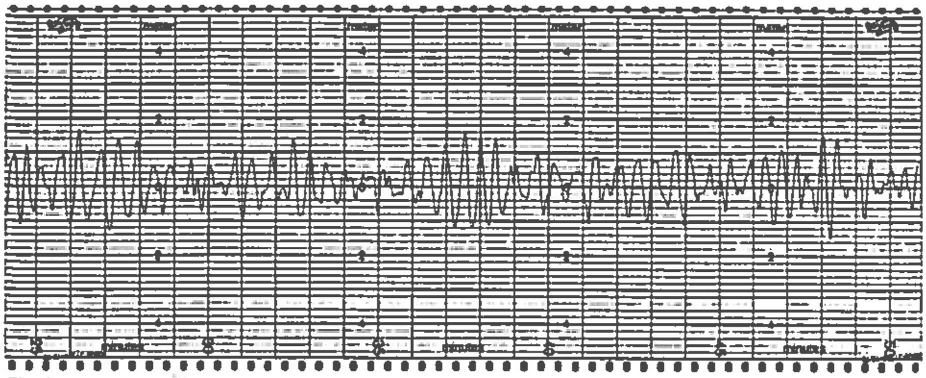


Fig. 2a

coordination with consultants on other design activities, in particular the prime engineer, are essential in order to conduct efficient and result-oriented hydraulic investigations.

The elements of hydraulic investigations for ports are outlined in the following.

Hydrographic Conditions

The first phase of the hydraulic investigation is to collect information on hydrographic conditions at the site of the proposed port facility. The approach naturally depends on the type of project, whether it is an extension of an existing harbour, a new harbour in a well-known area, or a new project in an area with little or no reliable information available. Existing harbours often have a hydrographic survey unit, which collects information for planning purposes, for control of maintenance of dredged areas, and for support to traffic control. This means that adequate data may already be available for expansion projects.

For new projects, hydrographic data collection and field investigations usually have to be initiated. Investigations concern waves, currents, wind, water levels, and sediments. For some of these parameters, long-term measurements are needed to establish reliable statistics as basis for preliminary and detailed design of the harbour. This means that analysis of existing data and evaluation of the need for further measurements must be carried out at a very early stage. Generally, it is recommended that measurements be made during at least one year in order to describe the seasonal variations. Sediment transport measurements should preferably be made under rough weather conditions in order to give reliable information as the processes of interest only occur under extreme wave and current conditions.

Waves

Wave measurements should cover both typical and extreme conditions in order to allow operational statistics to be developed. The measuring position should be as close to the proposed harbour as possible. But the optimal position is influenced by the uniformity of the wave field, zones of wave breaking, and possibilities for correlation with earlier measurements. It may, in some cases, be advantageous to use more than one wave recorder, at least during part of the measuring period, to establish correlations between wave conditions in various positions in the area.

Information on wave direction is often of decisive importance for the layout of a harbour. Unfortunately, operational field equipment for measuring wave directions is still not available at a reasonable cost. In a number of cases, DHI has obtained good direction results from a 2-directional current meter deployed in a bottom-mounted frame together with an accelerometer buoy on the surface or a pressure cell recorder. Alternatively, reliable mean direction statistics can be obtained by visual observations and it is strongly recommended to start an observation programme simultaneously with wave height measurements. Especially in locations exposed to swell it is not sufficient to rely on wind information only.

Mathematical hindcast models have been developed to a high degree of refinement during late years and are today in certain cases a valid alternative to wave measurements. In almost all wave measuring programmes hindcasts are essential to tie the measured data to long-term conditions through comparison with historical extreme events.

DHI's wave hindcast model (S20) has been used in a large number of studies and has shown remarkable precision.

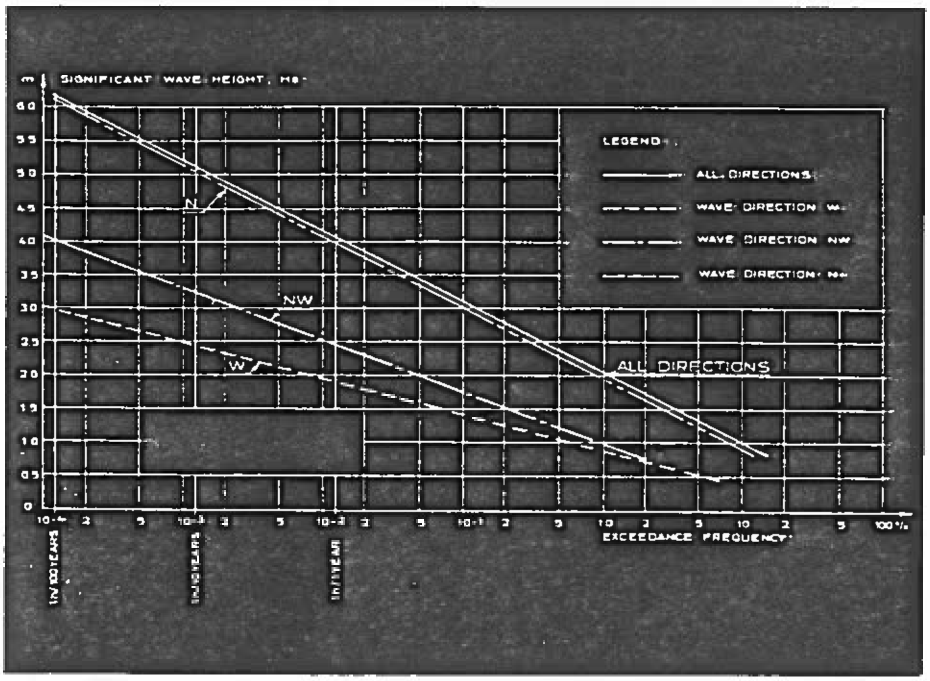


Fig. 2b

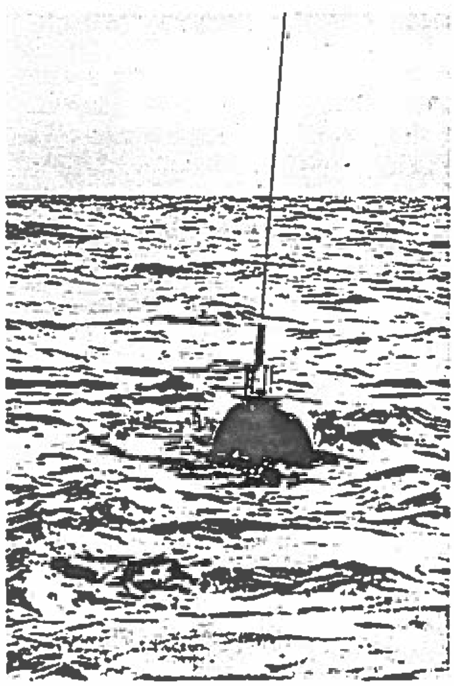


Fig. 2c

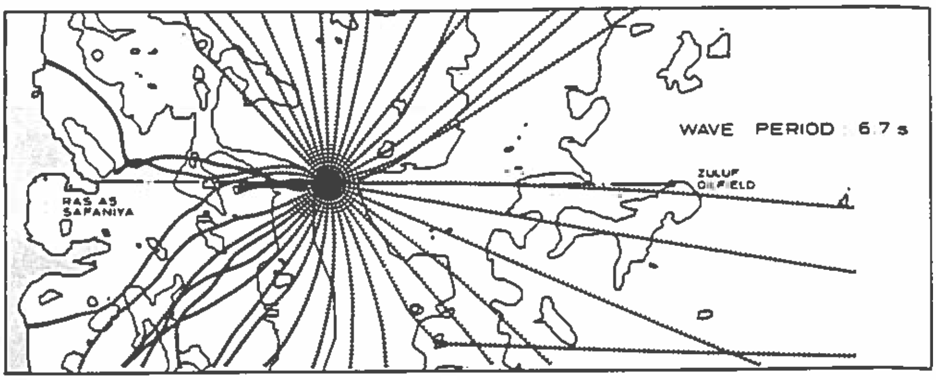


Fig. 3

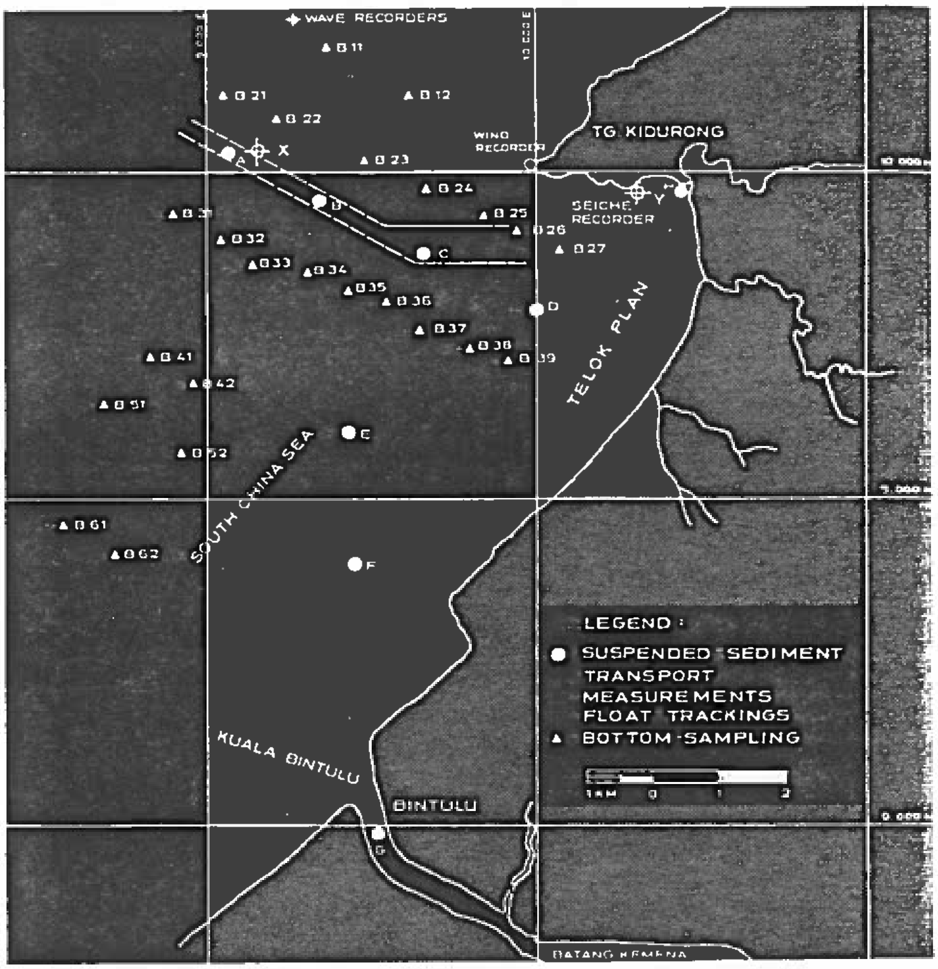


Fig. 4

Examples are shown in Danish Hydraulics No. 3. The main restriction on the use of these models is usually lack of realistic meteorological information. Therefore, in some areas, one is forced to use artificial storm descriptions based on ocean weather statistics. Hindcast models are less applicable when the wave climate is dominated by swell, created by distant storms.

The wave refraction models are traditionally used to transform deep-water waves to shallow-water waves or to correlate measured shallow-water data to deep-water data.

Long Waves
Long-periodic wave activity may generate long-periodic oscillations (seiche) in ports and movements of larger vessels in harbours. The oscillation periods in question are in the range of 1 to 3 minutes. In some cases, these long waves are caused by wave grouping, i.e. storm or swell waves travelling in groups, as shown in Fig. 6. Measurements of long waves are complicated because long-wave components have small heights compared to the shorter waves which we actually see. It requires a wave gauge with a fixed reference level, e.g. a pressure or a radar recorder or a wave staff, whereas a buoy (accelerometer) recorder cannot detect long waves.

When planning the field measuring programme it is useful initially to analyse available tide gauge records to see if long waves appear. In the affirmative, a measuring programme should be carried out. It is important to note that a standard for describing long waves has not yet been formulated. However, to reproduce the governing hydraulic processes it is necessary (and possible) to generate long-wave phenomena in laboratory models. Only continuous attention to this type of waves can help port designers to obtain an improved statistical description and hereby arrive at a realistic description of the operational performance of the ports to be constructed.

Currents
Description of the currents at the harbour site is usually obtained by the use of recording and profiling current meters, as well as float tracking. The description is required for sediment transport studies, for manoeuvring analyses and for studies of berth orientation at unprotected terminals. Simultaneous water level and wind recordings must be carried out for correlation with the current. Mathematical modelling is often a useful tool for more comprehensive descriptions of the current field during normal and extreme weather conditions.

Sedimentation Investigations
In addition to the description of waves and current, the essential factors in a sedimentation investigation are a study of the coastal morphology, sampling of the sea bottom sediments and determination of the sediment concentrations and settling velocities of sediments contained in water samples collected during relatively extreme weather conditions. In this way, useful information for identification of sediment transport patterns, areas of erosion and deposition becomes available, forming the basis for optimization with respect to capital and maintenance dredging.

Planning of Field Investigations
As appears from the description above, proper planning of field investigations requires a good understanding of the characteristics of the site. It is necessary to include all relevant seasons in a site investigation. An initial reconnaissance survey undertaken by an experienced coastal engineer will often provide the most valuable information for formulating optimal investigation strategy.

The elements of a field investigation programme for a port facility, with both sedimentation and wave disturbance

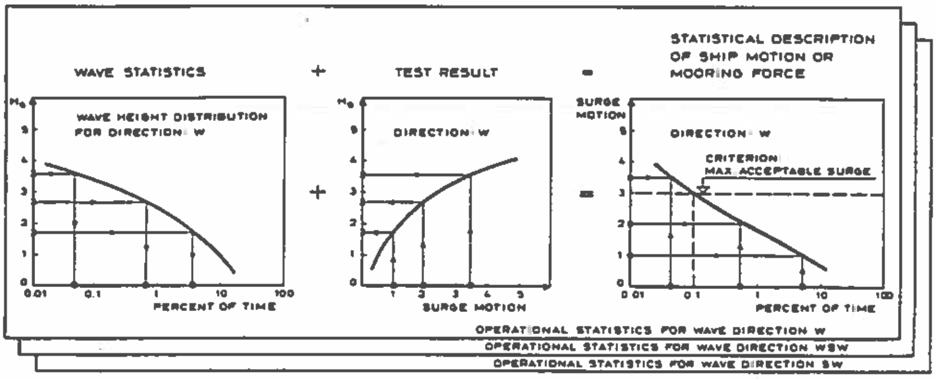


Fig. 5

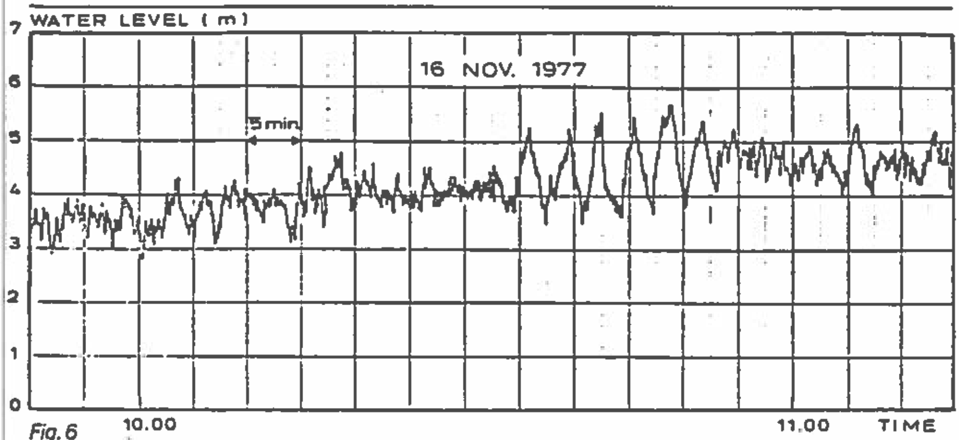


Fig. 6

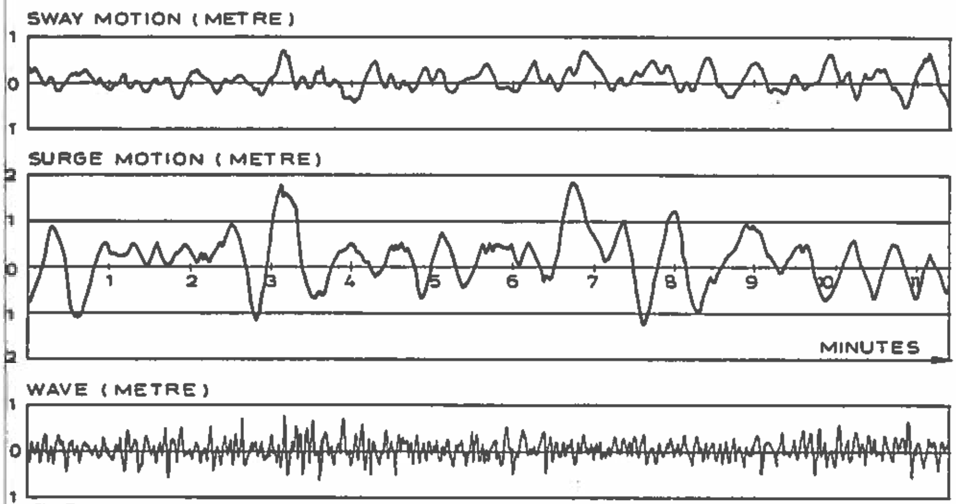


Fig. 7

Fig. 3. Zuluf-Safaniya wave study. Backward tracing of wave orthogonals is part of wave refraction studies with directional distribution. The graph shows the wave directions which reach the position of interest.

Fig. 4. Hydrographic field investigation, measuring positions.

Fig. 5. Analysis procedure for wave disturbance test.

Fig. 6. Long-periodic waves measured by water level recorder.

Fig. 7. Time series of horizontal motions of a moored tanker. Periods of motions are dominated by the natural frequencies of surge and sway, whereas the wave periods seldom appear in the records.

problems, are illustated by the chart of the Tg. Kidurong area in Sarawak, Malaysia, the site of the new Bintulu Deepwater Port (Fig. 4).

Port Design Criteria

The quality of a port design may be judged by its ability to allow safe navigation through the harbour entrance up to the wharves and its ability to ensure sufficiently calm conditions along the berths. These requirements are, to a certain degree, conflicting since the navigation needs the largest possible space, that is opening of the harbour, whereas the conditions along the quays generally improve when the entrance is narrowed. It is therefore important to establish realistic criteria for acceptable wave disturbance in harbours. In the past, these criteria were generally expressed in terms of maximum acceptable wave heights at the wharves. However, as moored vessels form oscillating systems with natural frequencies, such criteria are of little value. Wave disturbance criteria are typically expressed in the form of acceptable vessel motion, either as basic modes of oscillation (viz. heave, sway, roll, etc.) or more adequately as movements of important points of connection (e.g. tanker manifold or ferry ramp). The criteria are expressed as the percentage of time during which a prescribed maximum excursion is exceeded.

The ship movement criterion is often expressed as a maximum tolerable movement to be exceeded only during a few storms per year. For tankers, a motion of up to ±3 m along the wharf is usually acceptable if allowed for in the design of the loading installations. For container vessels, the acceptable motion is much smaller. The graph in Fig. 7 shows that it is not simple to apply such a criterion as the period of oscillation also enters the picture. A substantial large excursion is naturally more acceptable with a long period than with a short period, provided that the mooring system and connected cargo handling equipment can tolerate the full excursion.

Similarly, criteria are required for tolerable mooring and fender forces. A mooring system is considered safe when the breaking strength of the mooring lines is exceeded only during very rare storms.

Port Layout Studies

One or more preliminary concepts for the harbour layout may be proposed as a result of analysis of available hydrographic data, soil information, cargo volumes, etc. After selection of the most suitable layouts by a desk study, possibly including computer modelling,

Fig. 8. Wave disturbance test for Port Issers, Algeria.

Fig. 9. Short-wave model S21MK8 with 3-dimensional wave input.

Fig. 10. Manoeuvring simulation. Arrival of a 70,000 dwt bulk carrier using landbased winches.

a detailed wave disturbance study may be performed.

Physical model tests using irregular waves are generally considered the most reliable tool used by port engineers. In some cases, however, mathematical modelling is a valid alternative.

Physical Wave Disturbance Tests
The wave disturbance test describes conditions at the wharves for a number of test situations selected to represent the wave climate at the site. By wave climate is meant a statistical description of waves, comprising wind-generated waves and swell, as well as long-periodic waves. Fig. 5 illustrates the analyses based on wave disturbance tests. The results from each test run describe typical vessel movements, mooring and fender forces and wave heights in selected positions of the harbour. These results are related to the wave statistics, leading to cumulated distributions of the parameters.
By summation of results for all essential combinations of sea states, spectral types, and directions, a representative description of the essential parameters is developed for comparison with the selected criteria. The basic layout is tested with typical ships moored according to common practice.

The first results from the model will usually show that improvements can be obtained, for instance in relation to:

Change of port layout
- length of breakwater
- width of entrance
- arrangement of approach channel

Change of quay structures
- orientation of berth/pier
- arrangement of absorbing or reflecting quay structures

Improvement of mooring systems
- rearrangement of bollard and fender positions
- recommendations regarding number and type of moorings, i.e. mooring stiffness and force capacity
- pre-tension

A test programme of six to eight weeks, combining a number of the above alternatives, tests of various basic physical phenomena, (e.g. resonance problems and documentation of operational sta-

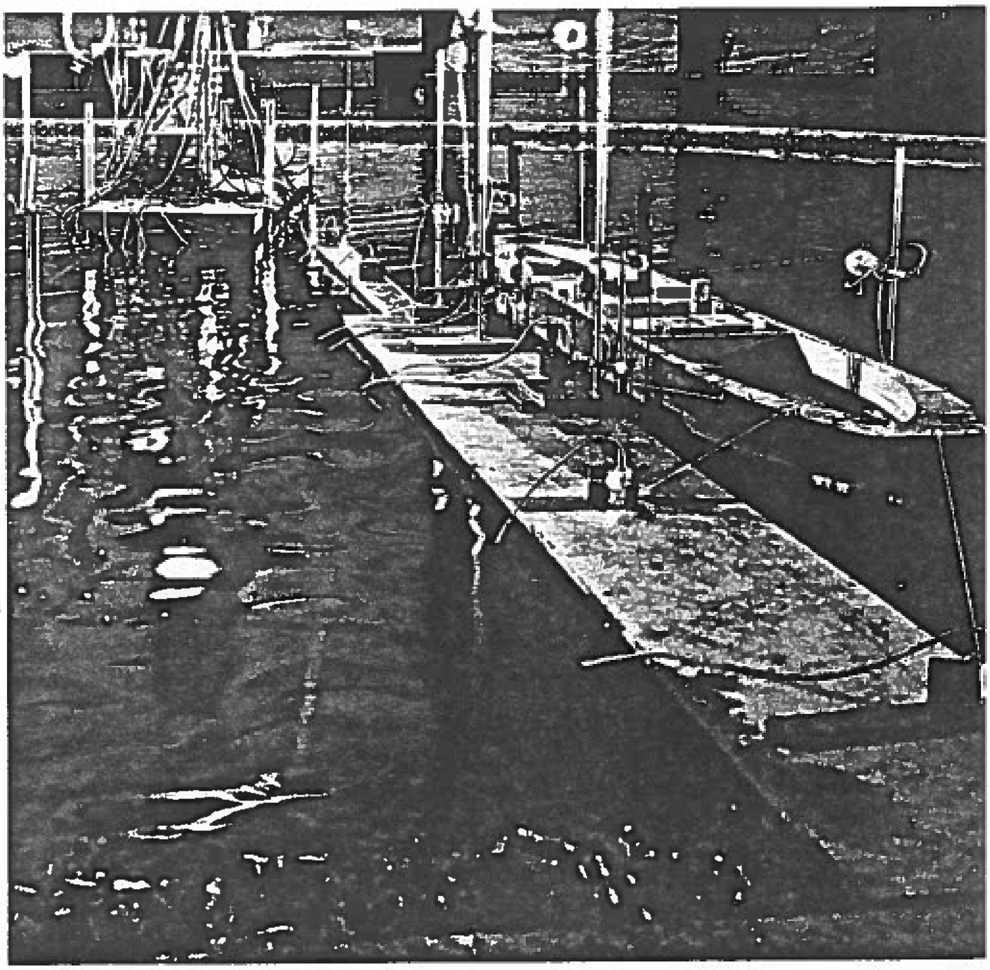


Fig. 8

tistics) usually contains more than 300 test runs, each reproducing a sea state of a one-hour prototype duration.

Selection of the mooring system is rather important. For large vessels it is usually found that a set of relatively soft moorings results in an optimal system in relation to mooring forces and vessel movements. This may, however, not be in accordance with the interest of the users and it is therefore always adviseable to involve the potential port users as early as possible, in particular when dealing with a terminal for specific vessels (e.g. ferry terminal). In ordinary multi-purpose harbours, it is generally not acceptable to lay down too strict a set of rules as they may be hard to enforce.

An advantage of describing the test results in terms of operational or safety conditions is that it allows the port designer to carry out cost/benefit analyses of the various alternatives.

Mathematical Models for Wave Disturbance Analyses
Computational models have been developed as alternatives to physical testing. DHI's System 21MK8, described in Danish Hydraulics No. 3, is such a model, capable of describing the wave pattern in an entire harbour as it varies in time. This type of model is today particularly useful for wave disturbance studies, where wave heights can be used as an adequate criterion (e.g. small craft harbours, extension projects of existing harbours). This type of model is useful, also as a supplement to physical models. Both the physical models and the mathematical models are now being developed to simulate natural 3-dimensional sea states. An example of an application of the mathematical model is shown in Fig. 9.

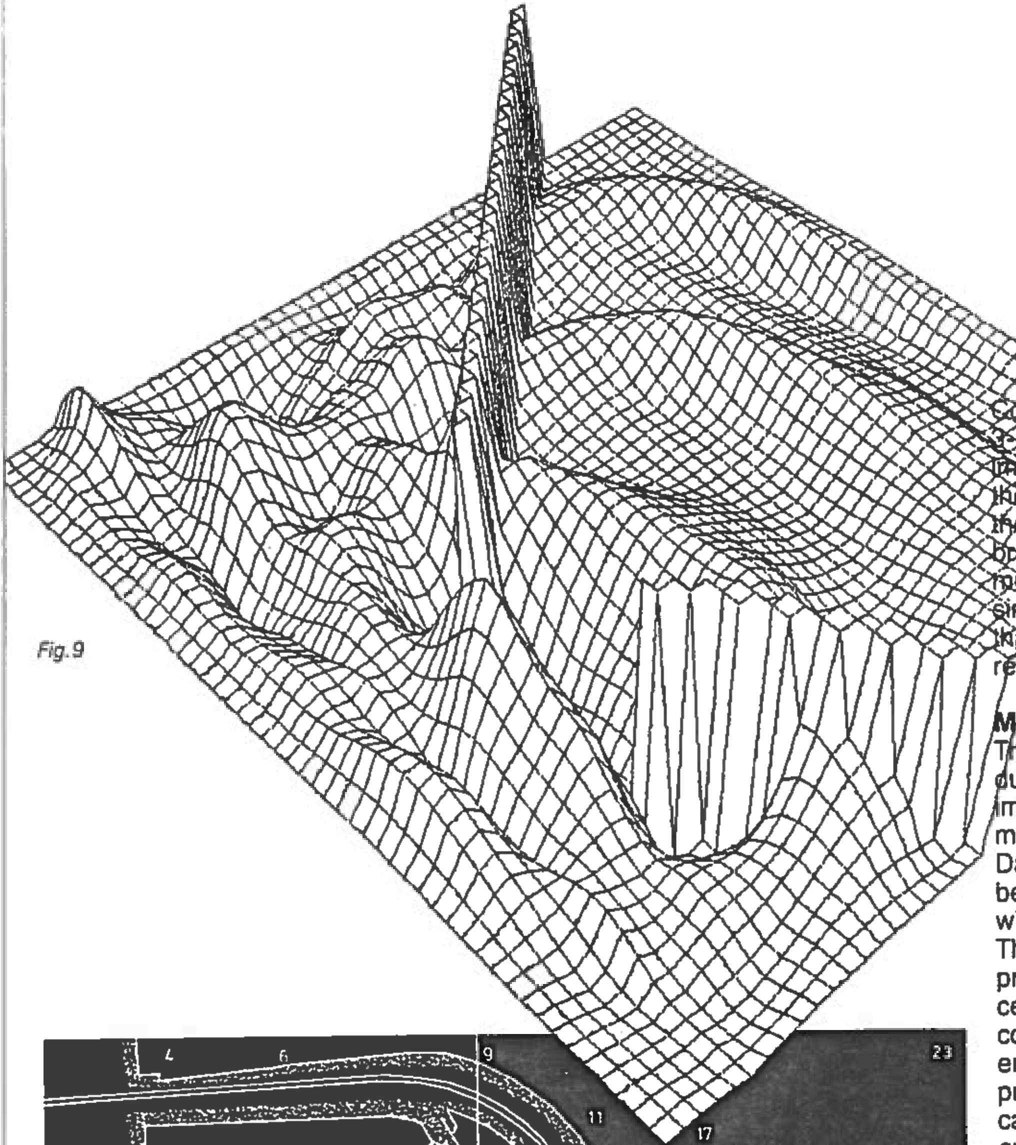


Fig. 9

Comparative tests between 2- and 3-dimensional models have shown important differences as indicated in theoretical descriptions, namely that the waves are more spread behind breakwaters in a 3-D wave field. One must remember that model tests are simplified descriptions of nature and that improvements, especially in wave reproduction, are still to be expected.

Manoeuvring Studies

The considerations of ship manoeuvres during arrival and departure have been improved by mathematical simulation models. The model developed by the Danish Maritime Institute (DMI) has been used several times in connection with port studies carried out by DHI. The simulation is used for defining proper dimensions of harbour entrances and approach channels taking into consideration both ordinary and emergency manoeuvres. Also new principles for approach manoeuvres can be studied. Fig. 10 shows an example of the studies recently carried out by DMI for the projected new harbour at Carboneras, Spain. Due to the distance from existing ports it was found to be economicly advantageous to replace tugs with landbased winches. It was concluded that a 70,000 dwt bulk carrier could be safely turned and docked, using three 50 t winches.

Study of Harbour Structures

When the harbour layout has been determined, the design of structures can commence. Actually, preliminary design concepts are evaluated simultaneously with the layout study in order to introduce the cost of the structures in the optimization of the layout.

The construction of breakwaters may have considerable consequences for the economy of the project.
To optimize structures it is therefore useful to conduct detailed physical tests. It is recommended, however, that preliminary analyses are made based upon experience from structures in similar wave climates, combined with knowledge of available construction materials.

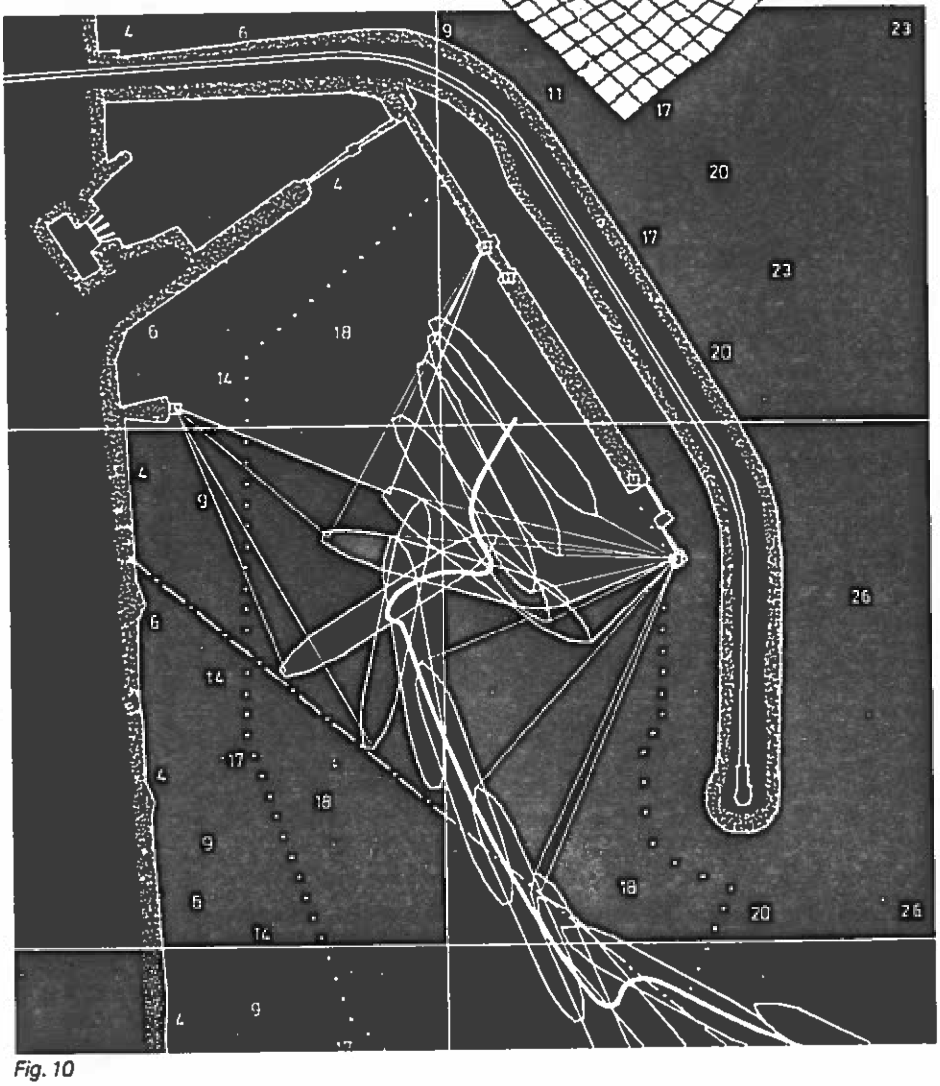


Fig. 10

Moored Vessels Exposed to Waves, Wind and Current

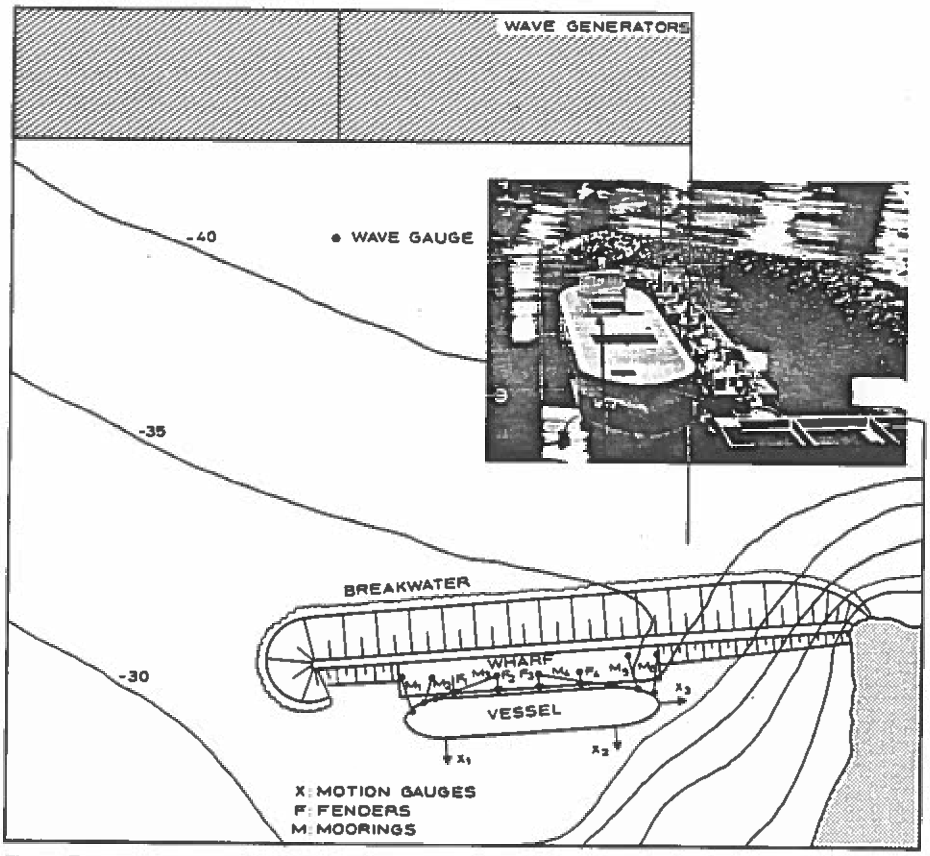


Fig. 1. Test set-up with moored vessel.

General

The movements and associated mooring forces for vessels in harbours and at unprotected terminals are essential parameters for the assessment of the operational quality of the berth and may therefore be of governing importance in the design process. The economic consequences are related both directly to the capital investment and indirectly to maintenance cost and operational reliability. Movements and forces are caused by environmental loads, such as short and long periodic waves, wind and current.

For analyses of these parameters, a model of the entire oscillatory system must be applied, i.e. vessel, moorings, fenders, and essential driving forces. The modelling of environmental forces, in particular, is extremely important to obtain reliable results of the study. This implies that too simplified modelling of wave conditions, for instance, may lead to both qualitatively and quantitatively wrong and thereby misleading results. An important example of this is the unsatisfactory representation of long waves and drift forces in many mathematical and physical models.

Waves

In harbours, at open terminals, and at offshore mooring installations, the wave climate is the main responsible for generation of unacceptable vessel motions and mooring line forces. The waves of interest may be local wind waves and/or swell in combination with the long periodic effects of wave grouping (slow drift forces).

In enclosed harbour basins both short periodic waves (<20 seconds) and long periodic waves (>20 seconds) may be of inconvenience for port operations. In harbours for small crafts or fishing vessels, for instance, the short periodic waves are normally determining berthing conditions.

For larger vessels moored at berths it is, in general, the longer waves which cause serious motions and mooring forces. As will be discussed later in detail, the long waves typically have periods of the same order of magnitude as the natural periods of large moored vessels. For this reason there is a risk of resonance.

Wind

The wind force on a vessel moored at a wharf or pier may push it away from or press it against the fenders. The first situation may be of decisive importance when combined with vessel motions due to wave action. This is because the vessel motions are generally larger when the vessel is not in contact with the fenders. In case of strong wind blowing towards the quay front, the wind force may hamper or make it impossible for a vessel to leave the berth. This is especially a problem for ferries and other vessels with a large surface exposed to wind and for unloaded vessels.

Current

For vessels moored at berths in current, either parallel or at an angle to the vessel alignment, the current may create large, long, periodic vessel motions. These motions depend on the current velocity, the inertia of the vessel around

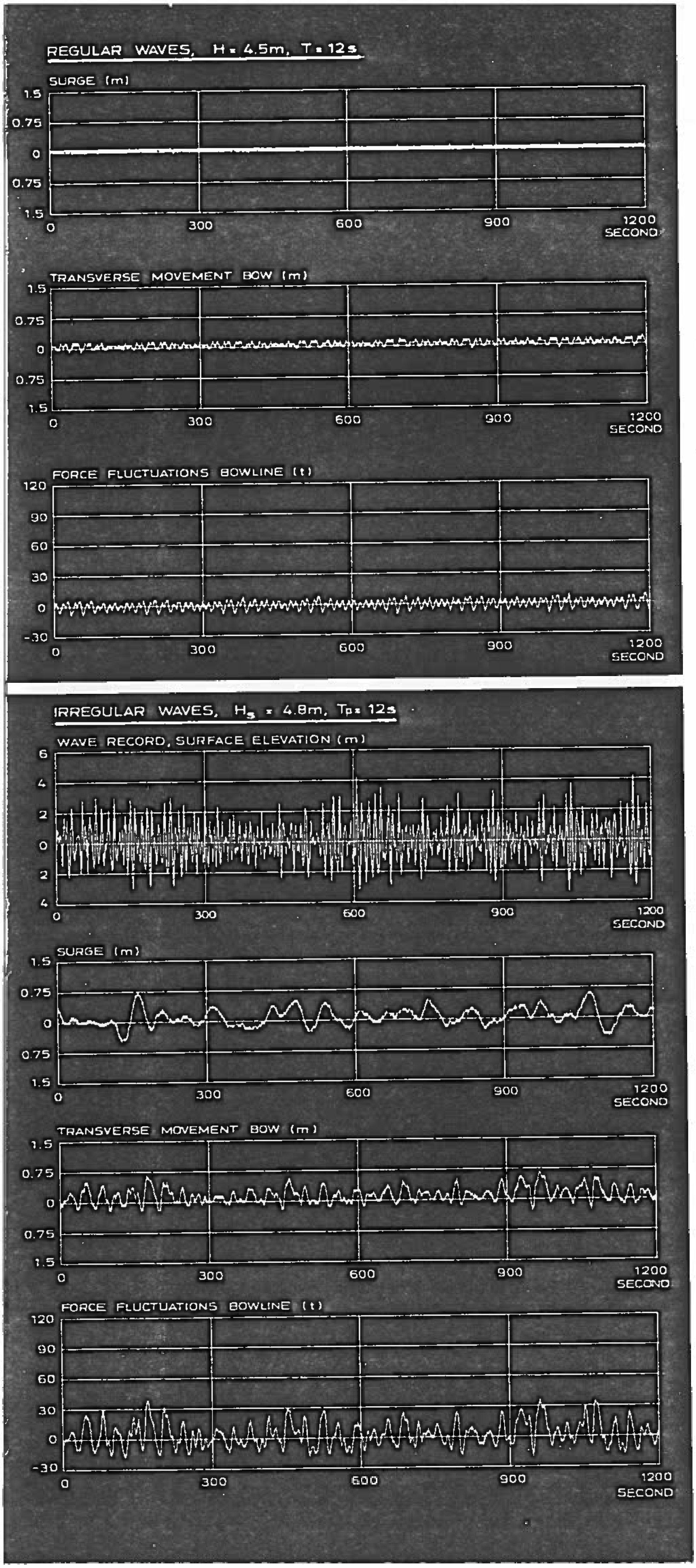


Fig. 2. Time series of waves, motions and mooring forces for moored vessel, as shown in Fig. 1.

the centre of gravity relative to the mass of the vessel including the hydrodynamic mass, and on the stiffness of the mooring/fender system as well as the moment of the respective forces around the centre of gravity.

In addition to the motions caused by current, the horizontal variation in the current at a pile supported bridge may give rise to a differential head, resulting in stand-off forces pushing the vessel away from a terminal.

Motions of Vessels Moored at a Berth in a Harbour

Large vessels moored at a berth in a harbour have two different types of oscillations. One type is the natural movements of a freefloating vessel, (heave, roll and pitch). These motions are almost unaffected by the moorings and fenders. The other type of oscillation comprises surge, sway, and yaw. These motions are all related to the mooring and fender forces that tend to counteract displacements of the vessel from its equilibrium position. It is the mass of the vessel, including its hydrodynamic mass, in relation to the mooring/fender arrangement (moments of inertia) that governs the natural periods of the moored vessel.

As previously explained, the vessel motions increase significantly when the periods of the driving forces are equal to or in the same range as the natural periods of the moored vessel. The vessel cannot, in general, be characterized by the simple theories for response of flexible mechanical systems exposed to external excitation.

The physics of a moored vessel is more complex since the characteristics are highly non-linear. This is because the stiffness characteristics are not symmetrical (fenders are usually stiffer than moorings). Furthermore, the load-deflection characteristics of fenders and mooring lines are not linear. In addition to these characteristics, related to the moored vessel itself, the acting wave forces are highly irregular and time-varying.

Fig. 2 illustrates the mooring motion characteristics for a large vessel

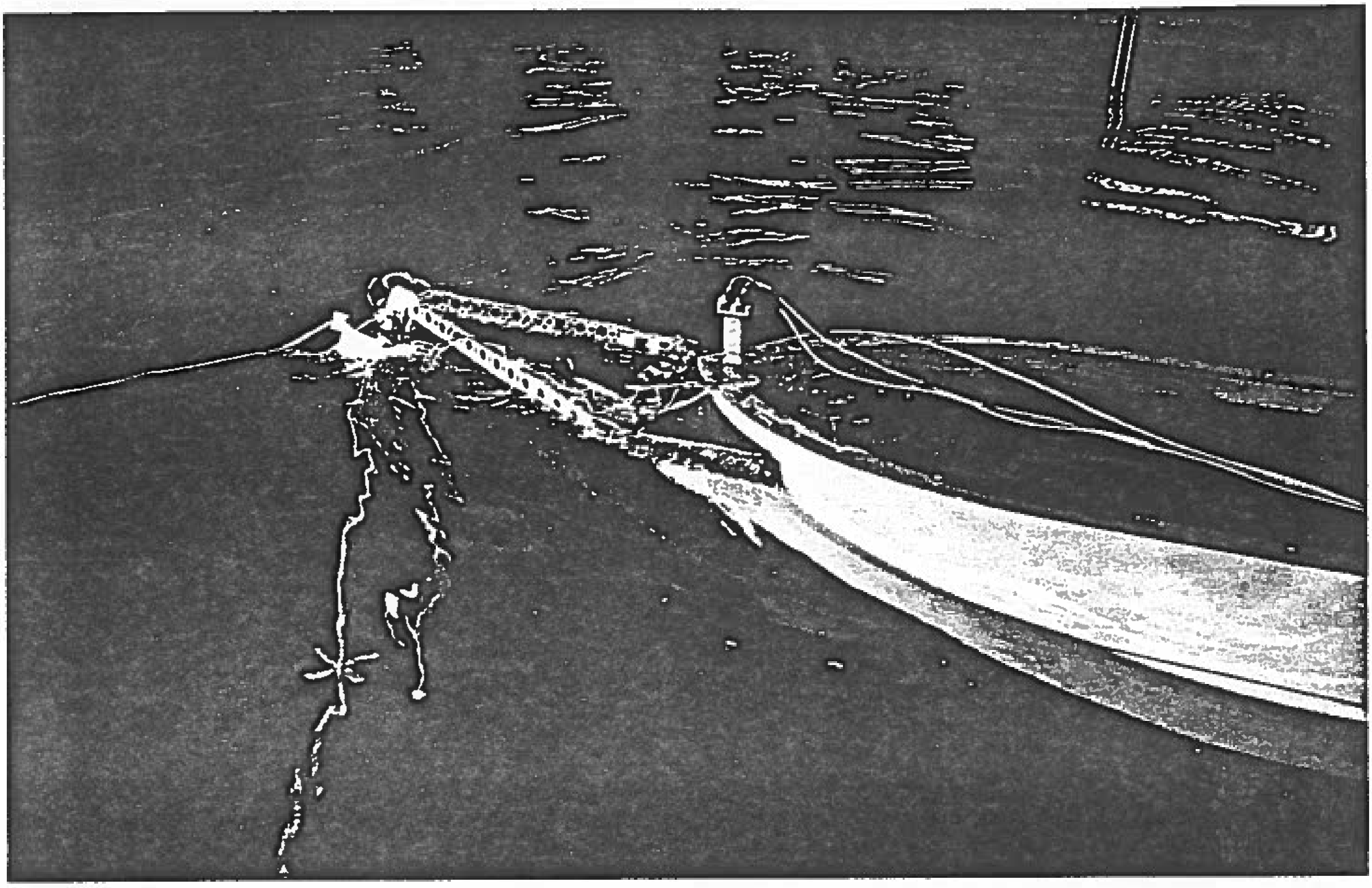


Fig. 3. *The wave drift results in varying tightening of the chain leg. When displaced from the equilibrium position, restoring forces increase significantly.*

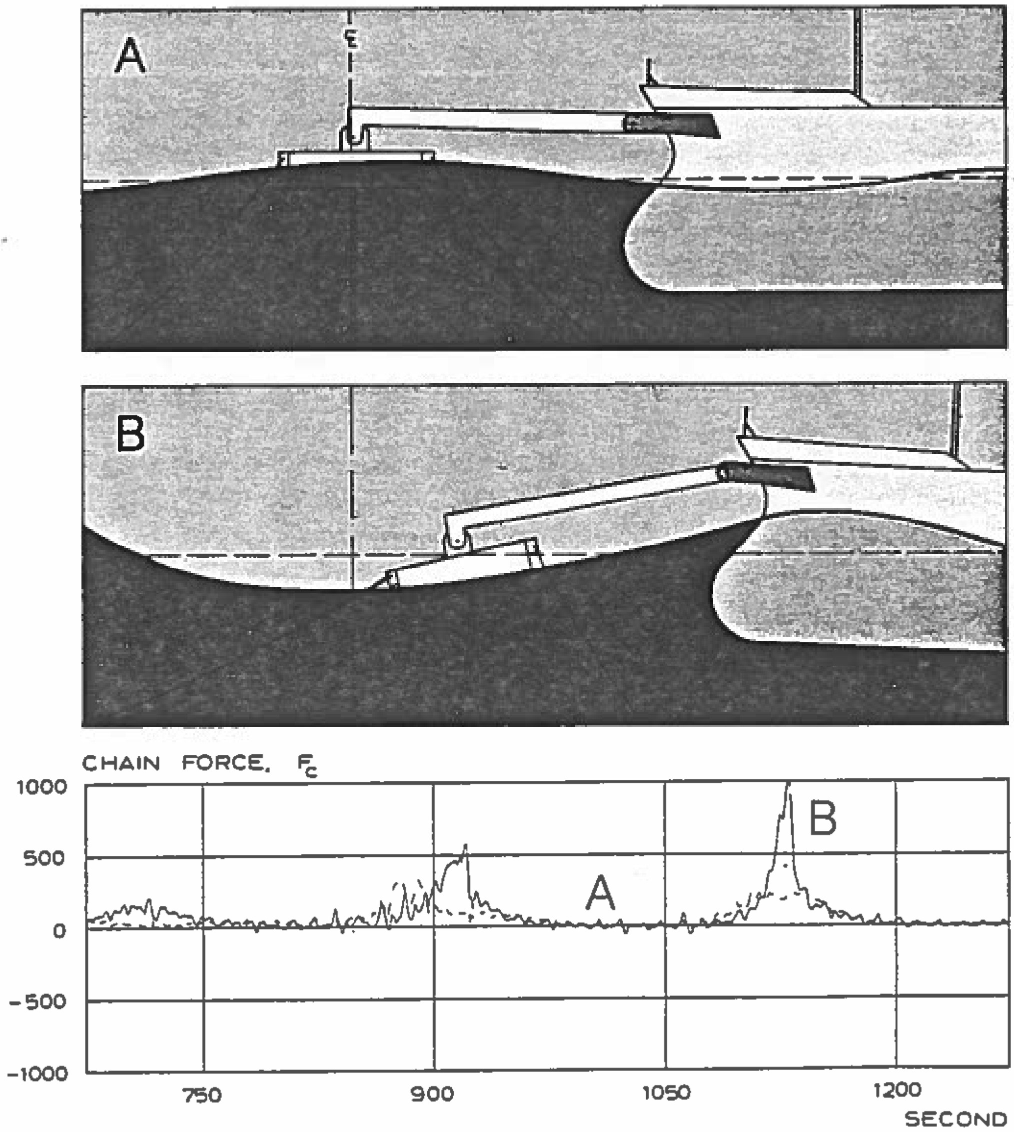
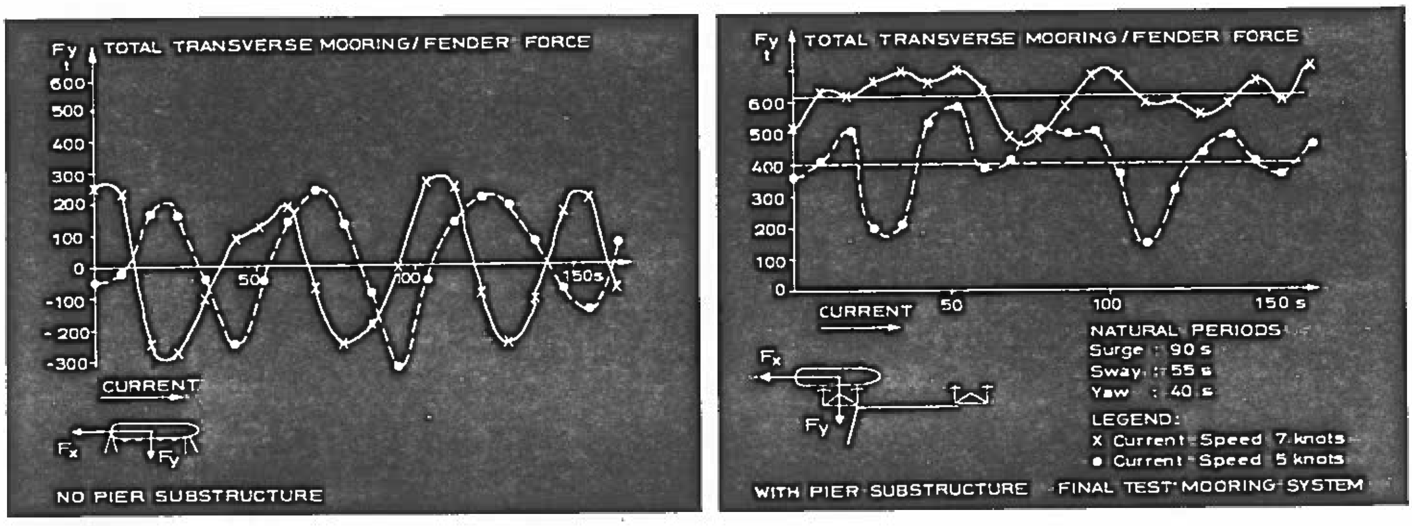


Fig. 4. Test results for vessel moored at a berth in strong current.



moored at a berth. It shows time series of wave motions and mooring line forces. Tests were conducted, both for regular waves and for irregular natural waves generated from a natural wave record. Such tests are normally conducted in scale 1:100 and Froude's Model Law can be used for conversion from model to reality.
As shown in Fig. 2, the wave heights (H and H_S) and wave periods (T and T_P) are almost identical for the two set of tests.

The time series for regular waves show almost no planar vessel motions. The reason why the planar motions (surge, sway, and yaw) are limited is that the wave periods in the regular waves are much smaller than the natural periods of the moored vessel. Consequently, the amplification is almost zero.

The situation with irregular waves is quite different. The vessel is now forced into large motions due to the long periodic seiches in the harbour caused by wave grouping. The seiches have periods in the same range as the natural periods of the moored vessel. However, the effect of the normal sea waves diffracted into the harbour is still apparent and very small, compared to the slow drift motions.

This clearly demonstrates the importance of conducting tests with irregular waves simulating, as closely as possible, the irregularities and grouping phenomena of natural waves.

Vessels Moored to Offshore Buoys

A vessel moored to an offshore buoy (a single point mooring, SPM) constitutes in principle the same hydraulic problem as a vessel moored in a harbour. When moored to a buoy or similar, the system (vessel, buoy, anchors, etc.) is complex, having much freedom of motion and several natural periods. This system may be forced into resonance due to the effect of wave (and wind and current) action. Due to large sea or swell waves, the vessel is also having relatively large vertical motions.

However, the largest forces in the system are normally associated with slow-drift motions due to wave grouping with periods in the same range as the natural period of the system.

Fig. 3 shows time series from testing of an SPM system simultaneously exposed to waves (H_S=10.7 m) and wind (65 knots). Records of chain forces are represented. It is seen that, due to vessel motions, the forces are oscillating with periods equal to the periods of the normal sea waves.

However, these forces are superimposed on very slowly varying forces with a period of 200 seconds. This period is equal to the natural period of the longitudinal motion of the system and the motion is occurring due to the effect of wave groups.

Vessels Moored at an Open Terminal

At an open terminal, motions introduced by sea and swell waves are more pronounced compared to the slow-drift motions than what may be experienced in harbours. This means that roll, heave, and pitch motions at such facilities often will be critical for operations.

Vessels Moored at a Berth in Strong Current

As described in the introduction, a vessel moored at a berth in strong current may also be forced into oscillation. The motions occur when a moored vessel is dynamically unstable to lateral displacements. It requires, however, that the current velocity exceeds a threshold value, usually around 1 m/s (2 knots). The motions of a large vessel are long-periodic. With traditional moorings, a typical natural period would be one minute.

Fig. 4 shows model results of total transverse mooring forces for current velocities of 5 and 7 knots and for situations with and without the pier substructure, respectively. Both the stand-off force for the situation with the pier substructure and the variations in the force, due to ship motions, are clearly seen.

Breakwater Investigations

Different Types of Breakwater Studies

The following different types of breakwater studies can be identified:

- **Caisson breakwaters (or composite structures).** Overall stability and design forces, stability of toe protection and overtopping are studied in hydraulic models
- **Rubble mound structures** Design of all components and evaluation of overtopping from flume tests
- **Breakwater heads and other 3-dimensional features** Evaluation of stability by use of hydraulic wave basin models.

Introduction

Hydraulic investigations of breakwaters include testing in wave flumes and 3-dimensional wave basin tests.
At DHI, such studies are made by using natural wave records as basis for direct reproduction of the irregular wave trains in the models.

General Considerations on Breakwater Design and Investigations

Hydraulic model investigations are at present the only reliable tool to be used in the design process of breakwaters. However, model tests have certain limitations due to scale effects, so the interpretation of test results has to be made by using different model laws and practical 'rules' derived from experience. It is of great importance for a breakwater project that the hydraulic laboratory is involved at an early stage. The laboratory may thus assist with the formulation of the requirements to field data and with the outline of different types of structures to be considered.

Study of Caisson and Composite Type Breakwaters

The development of the Hanstholm Harbour on the Danish North Sea coast led to the development of a new type of caisson breakwater with a sloping face above still water level.

The concept was developed through hydraulic model tests. The tests showed that a considerable reduction in the horizontal wave force and the overturning moment could be obtained by making the upper part of the caisson with a sloping face. At the same time, however, the wave disturbance in the harbour basin, generated by overtopping waves, requires more consideration. For Hanstholm Harbour the caissons were circular, which had certain structural advantages. Further, the wave forces on circular caissons are reduced compared to forces occurring on rectangular caissons.
The same concept has, with various modifications, been used for other breakwaters, such as: Brighton Marina, U.K. and Marsa El Brega, Libya. Fig.1 shows a flume test situation in a project for Sagunto, Spain.

It has been documented that the design of large caisson breakwaters requires careful model tests with the generation of natural wave trains, since only such waves can reproduce the important wave shocks. The wave shock forces are determinant for the stability of the caissons, both in overturning (rocking) and in sliding.
In models, representative shock forces only occur in irregular waves. Also for determination of overtopping and for the scour protection it is necessary to test in waves reproducing the irregular characteristics of natural waves.

Study of Rubble Mound Structures

Study of Breakwater Failures

A number of breakwater failures have occurred in recent years. Studies of the Bilbao Breakwater in Spain (1977) and of the provisional repair of the breakwater in Port d'Arzew El Djedid in Algeria (1981/82) have been carried out by DHI. There is no common reason for the failures of these structures and the other major failures experienced lately. It is, however, evident that the methods used for design of smaller breakwaters in shallow water have been extended much too far in relation to breakwaters in deep water with complicated profiles and artificial concrete armour units.
In deep water, very steep slopes are usually adopted to minimize the volume of construction material. Breakwaters in deep water with large artificial concrete armour units on a steep slope are subject to rapid break-down in case the main armour layer is seriously damaged. In deep water, the maximum incident waves are not limited by wave breaking, which is further increasing the probability of serious damage.
In shallow water, the situation is often more favourable since the highest waves, which are able to reach the structure, are limited due to wave breaking.

It is evident that there is a need for safer breakwater structures. It is therefore interesting to note that, for almost all major breakwater failures, the repairs are made using blocks that are more simple than the ones originally used.
Dolos and tetrapods were previously

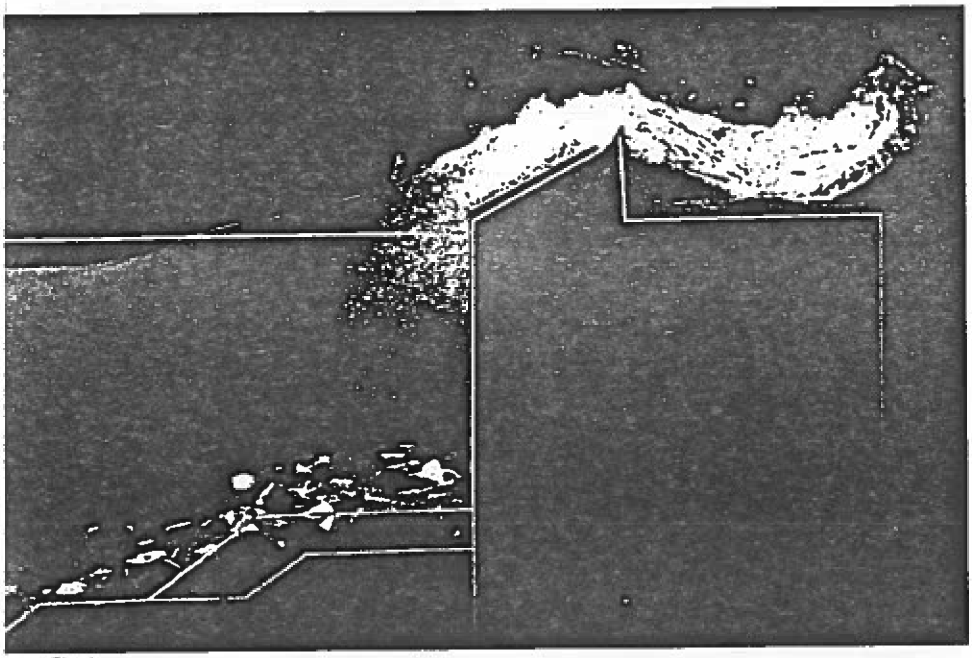


Fig. 1

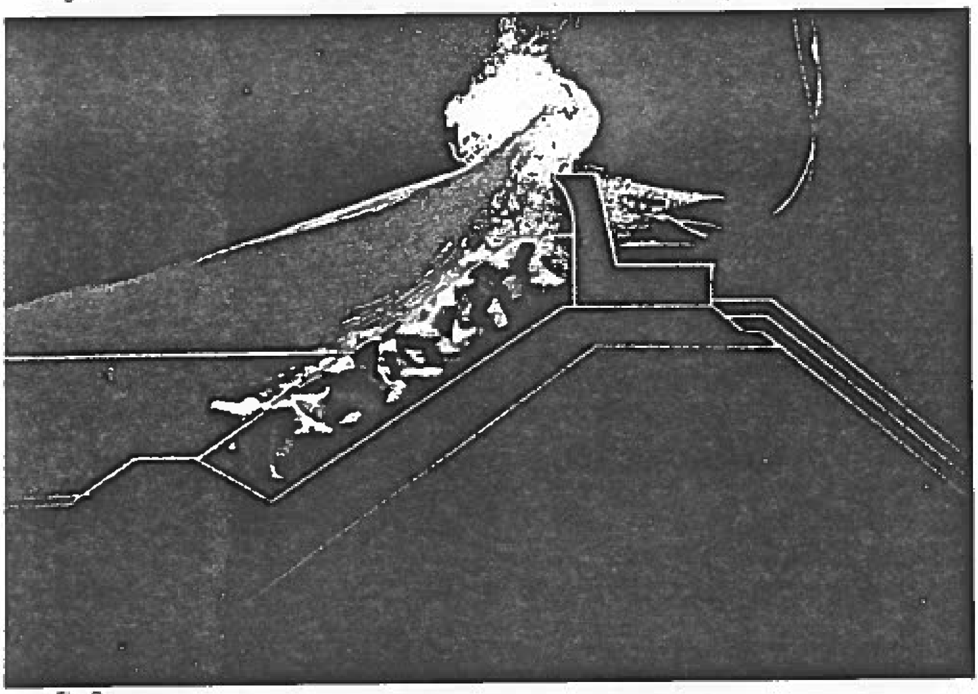


Fig. 2

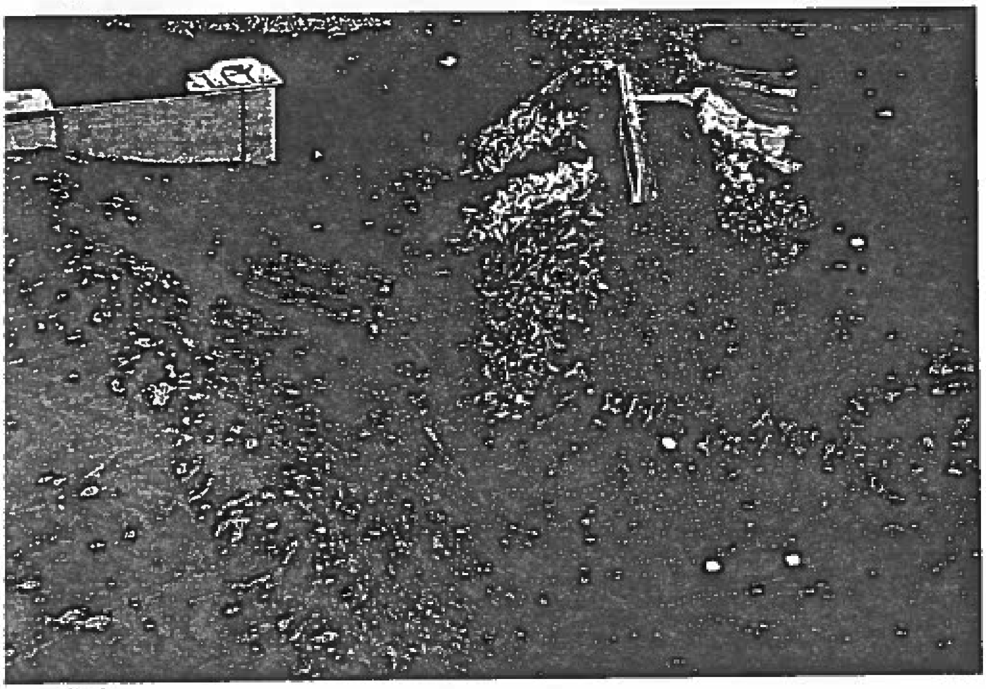


Fig. 3

Fig. 1. Flume tests with caisson breakwater for Sagunto, Spain.

Fig. 2. Breakwater flume model exposed to heavy overtopping.

Fig. 3. Model study of breakwater construction stages.

considered the most economic solution. For Bilbao, which originally had a slope of 1:1.5 and rectangular blocks of 65 tons, the repairs are being made by using 150 t rectangularly shaped concrete blocks on a slope of about 1:2.1, and in Arzew El Djedid the provisional repairs are made by means of 24 t Antifer blocks (grooved cubes).

Also for new breakwater designs in deep water, such as the one shown in Fig. 7, rectangular concrete blocks are used to achieve a safe breakwater structure.
The use of more simple concrete blocks than dolos and tetrapods does not imply that these are completely abandoned, but only that they should be used more carefully due to their fragility and be subject to more strict damage criteria than previously considered necessary.

Breakwater Flume Tests

Flume tests are used to investigate the trunk of breakwaters in cases where the breakwater is exposed to nearly perpendicular wave incidence. The scale usually ranges from 1:20 for small breakwaters to about 1:60 for larger structures in deep water. In order to obtain representative conditions, it is important that irregular natural waves, reproduced from natural wave records, are used in the model testing and that the sea bed profile is reproduced to sufficient depth or distance from the structure. In order to study overtopping, wave flumes should also be equipped with installation for the simulation of wind.

At the moment, it is not possible to model the concrete strength properly in small-scale hydraulic models. For this reason, the interpretation of small-scale tests implies a large degree of engineering judgement.

Flume tests are used to evaluate the stability of all parts of a breakwater design, such as:

- toe and berm
- armour layer
- filter layer
- crest and rear side
- crown wall
- overtopping

Overtopping

Besides the structural stability, the overtopping has proven to be of major importance for many breakwaters.

At an early stage in the design process, the effect of overtopping on structures, roadways, etc., placed closely behind a breakwater, should be considered to establish pertinent criteria. In extreme cases (i.e. very low structures) overtopping waves generate waves in the harbour which may cause damage to moored vessels and structures. Fig. 2 is an example of a breakwater flume model exposed to substantial wave overtopping.

3-Dimensional Wave Basin Testing

For sections of a breakwater exposed to very oblique wave attack, breakwater bends, and roundheads, etc., wave basin tests are used for assessment of structural stability. Also in cases with rather irregular bottom contours in front of a breakwater the 3-dimensional testing is relevant.

Such tests are basically carried out in the same manner and with application of the same methods as in flume testing.

Fig. 5 shows a 3-dimensional breakwater model for the fishing port of El-Kala in Algeria and Fig. 4 the same breakwater seen from the shore during a heavy storm.

Study of Construction Stages

Hydraulic models are also excellent tools for the study of breakwater construction stages.

Breakwaters are always more vulnerable during construction. The purpose of hydraulic tests is therefore to assist with the determination of a proper construction strategy to minimize the risk of damage and delays.

Examples of Breakwater Studies

In recent years, it has been proposed to make rather simple breakwater profiles, for instance with thick layers to ease construction and to make structures less sensitive to inaccuracies. Furthermore, attempts are made to design profiles that actually use all gradations from the quarry to minimize waste of material. Two typical examples are presented below:

Zwara Port, Libya, 1979

The profile for this major port developed in collaboration with the consulting engineers, Sir Alexander Gibb & Partners, U.K., is shown in Fig. 6. The client is General Ports and Lights Authority, Tripoli, Libya.

The following features of the breakwater are especially noticeable:

- the armour layer is supported by a large quarry stone berm
- a 3.5 m thick layer of 0.5-4.0 t stones is used as filter layer (secondary armour)
- the armour layer in front of the crown wall has a 5 m wide horizontal shoulder. This is important for reduction of wave impacts on the wall and of the risk of damage to the superstructure in case the armour layer is damaged
- the superstructure has a 'heel' to improve the stability

Puerto de Carboneras, Spain, 1982

The project for Puerto de Carboneras has been made in cooperation with Christiani & Nielsen A/S, Copenhagen, for PUCARSA, Madrid. Fig. 7 shows the profile developed for the major part of the outer trunk of the breakwater in depths of 10-20 m. This type of breakwater profile, shown in Fig. 7, proved to be economicly more feasible than a traditional design with a superstructure or a lower structure with large units, also on the crest and rear side.

The breakwater has the following remarkable features:

- the toe and lower part of the seaward face is supported by two very large berms, each with a width of 10 m. The lower berm is placed directly on the bottom and damage is acceptable. The berm is easy to construct and does not require a high degree of construction accuracy
- the main armour layer consists of two layers of 57 t rectangularly shaped concrete blocks on a slope of 1:2. The sublayer is 4 m thick and consists of 0.3-2.0 t quarry rock
- the upper part of the seaward armour layer, the crest and the rear side armour consists of 2-4 t quarry rock. Model tests have proven that this solution has adequate stability. The crest and rear side will be stable for extreme waves due to the high crest elevation resulting in little wave overtopping during such conditions
- depending on the characteristics of available quarry run for the core, an extra filter layer may be required

Fig. 4. Breakwater in El-Kala, Algeria, during a heavy storm.

Fig. 5. 3-dimensional breakwater model study for El-Kala, Algeria.

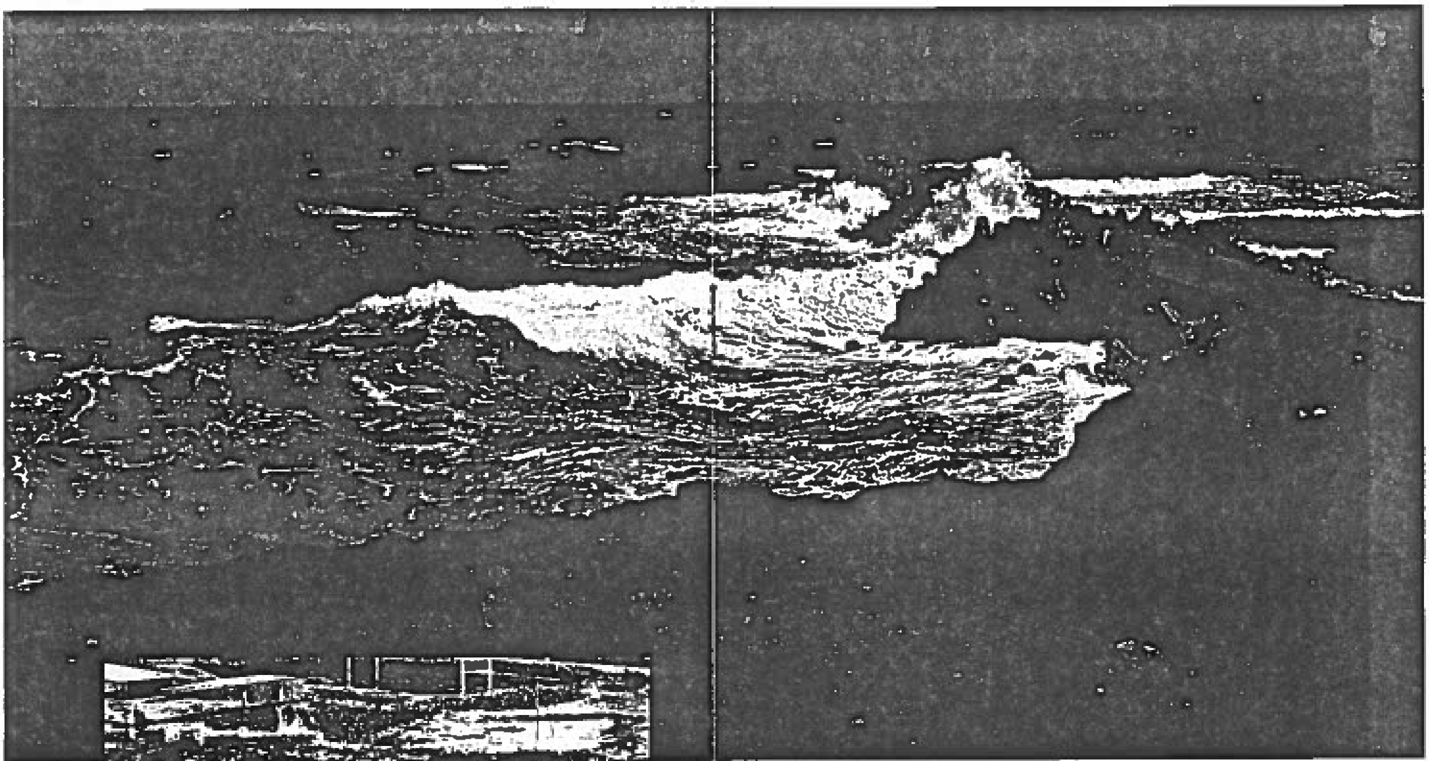


Fig.4

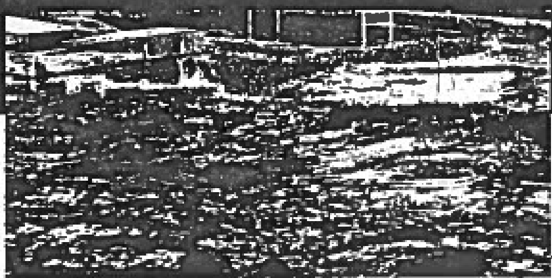


Fig.5

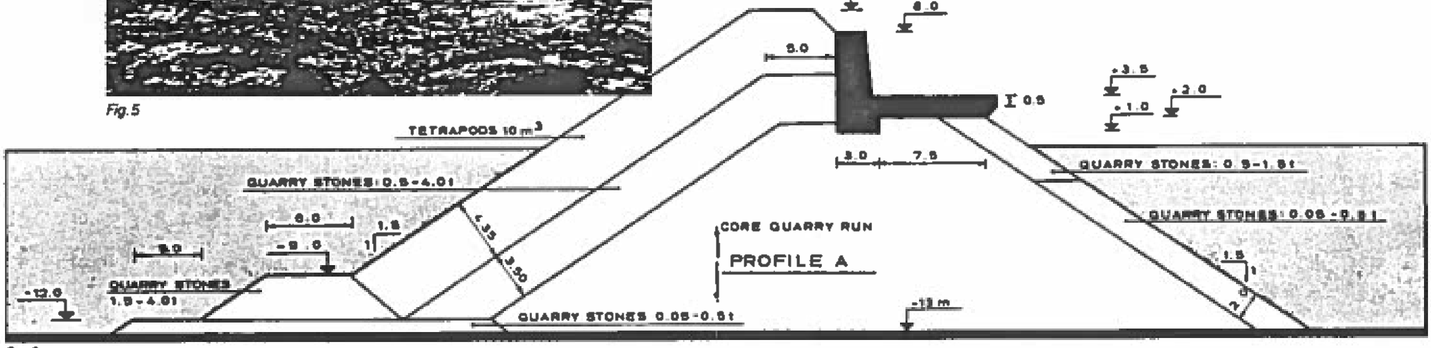


Fig.6

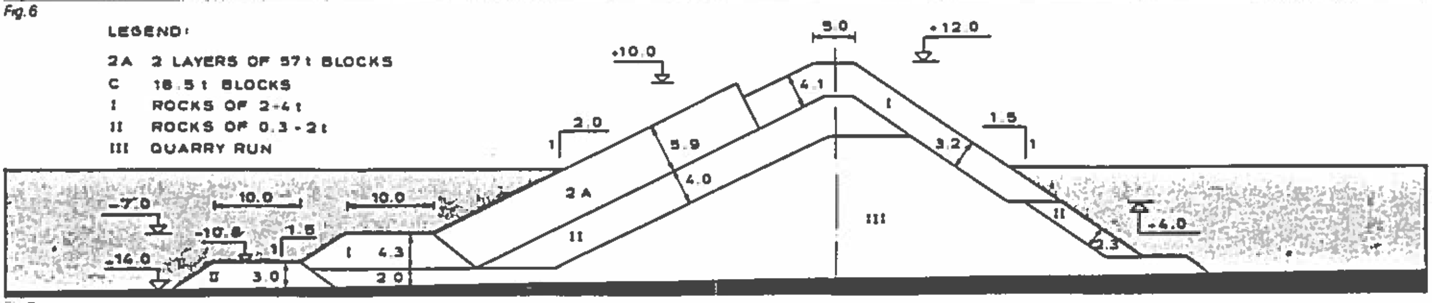


Fig.7

Fig. 6. Breakwater profile for Zwara Port, Libya.

Fig. 7. Breakwater profile for Puerto de Carboneras, Spain.

DANSK VANDBYGNINGSTEKNISK SELSKAB

Seminar om Nye Havne

HYDRAULIC STUDIES FOR BINTULU DEEPWATER PORT

by

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&

A. Hasle Nielsen, Deputy Director, Danish Hydraulic Institute

HYDRAULIC STUDIES FOR BINTULU DEEPWATER PORT BY J. Kirkegaard, Chief Hydraulic Engineer, Ports and Marine Structures and A. Hasle Nielsen, Deputy Director, Danish Hydraulic Institute

1. INTRODUCTION

The present paper describes the hydraulic investigations for Bintulu LNG and general cargo harbour carried out by the Danish Hydraulic Institute (DHI) in 1977-79. The investigations formed part of the feasibility study and the detailed design studies carried out by Stanley Consultants Inc., Iowa, U.S.A. for Public Works Department, Kuching, Sarawak. DHI's studies were coordinated by Dravo Van Houten Inc., New York, U.S.A. who acted as subcontractor to Stanley Consultants on marine engineering problems. The approval of the Authorities as well as of the Consulting Engineers to make this presentation is greatly appreciated.

The hydraulic investigations included studies of sedimentation conditions in the approach channel and port basis, studies of wave disturbance at the LNG jetty and wharves and of the breakwater design.

The paper concentrates on the most important aspects of the studies which were the sedimentation analysis and the investigations on vessel mooring problems in long period waves generated by wave groups.

2. BINTULU DEEPWATER PORT PROJECT

A new port facility at Bintulu in Sarawak is under construction. This port is intended to serve as an export facility for liquified natural gas and as a general cargo harbour, Refs. /1/ and /2/. The harbour is located in the shallow bay, Telok Plan, south of Tanjong Kidurong, see Fig. 1. The port and terminal facilities are connected with deep water through a 5.5 km long, 240 m wide and 16.5 m deep approach channel. An outer breakwater almost 900 m long protects the LNG jetty which is designed to handle 130.000 m^3 tankers. These vessels are approximately 180 m long and have a draft of 11.5 m. They are characterized by a large freeboard, and thereby a large wind surface. For this reason safe navigation requires special attention in regard to channel width and size of turning area. The general cargo harbour is located in the inner part of the bay and is separated from the LNG facility by a safety distance of 1000 m.

3. OUTLINE OF HYDRAULIC INVESTIGATIONS

The investigations were planned in cooperation with the consulting engineers with the objectives of supporting the following aspects of the feasibility and engineering studies:

- Design of approach channel.

  Methodology: Field investigations to describe sediment transport patterns in the area followed by calculations of sedimentation rates in the approach channel and harbour basins using mathematical modelling techniques.

- Design of harbour basins including optimization of LNG jetty location and layout of outer breakwater.

  Methodology: Wave disturbance studies including wave climate investigation and physical model study with irregular waves for determination of mooring forces and movements of moored vessels in the harbour. The effects of channel configuration upon wave propagation was of significance and hence modelling of a large part of the channel was required.

During the study it was found that additional tasks had to be carried out to resolve disturbance relating to long period wave activity in the area. These tasks included field measurements of long and short waves and mathematical model studies.

- Design of breakwaters.

  Methodology: Model studies in wave flume and in wave basin on stability of armour layers of breakwater trunk sections and breakwater head.

4. FIELD INVESTIGATIONS

The hydraulic investigations on site were aimed at establishing an adequate basis for the subsequent theoretical analysis and model investigations. Earlier investigations had already provided a certain insight in the site conditions and the results of several years of wave measurements were found to provide a sufficient basis for the wave climate study. However, indications of long periodic waves in the area had to be verified and better knowledge of wave directions had to be obtained. Otherwise, the supplementary field measurements emphasized sediment characteristics and sediment movements in the area.

The first phase of the field investigation programme took place in the beginning of the NE-monsoon period, November-December 1977 and comprised:

- Sampling of bottom sediments to describe sea bed characteristics.

- Collection of water samples to determine content of suspended sediments at various elevations, notably close to the sea bottom.

- Simultaneous wave and current measurements to determine correlation between sediment suspension and hydrographic conditions.

- Tide and long wave recordings.

Measuring and sampling positions are shown in Fig. 2.

The data analysis revealed the existence of long period waves with more critical wave periods than originally anticipated, namely 3 min. periods, instead of 10-12 min., and the result of model tests emphasized the importance of the long waves (see Section 7). A second field investigation programme was therefore carried out in the NE-monsoon period 1978-79 with the objective of finding an explanation of the generation of the long waves recorded in the area. This programme comprised measurements by an OTT seiche recorder located in the bay of Telok Plan (position y in Fig. 2) and a bottom mounted DHI pressure cell recorder located at 13 m depth (position x). The pressure recorder measures pressure variations caused by surface waves as well as by long waves and tides. This type of instrument was therefore selected in favour of an accelerometer buoy system which cannot detect waves with periods greater than about 20 s.

5. METEROLOGICAL AND HYDROGRAPHIC CONDITIONS

Meteorological Conditions

The climate of the region is characterized by the monsoon seasons with predominant north-easterly wind directions during the winter and south-westerly winds during the summer. In the coastal areas the general monsoon wind system is normally superseded by the breeze resulting in winds from sea (NW) during daytime and from land (SE) during night.

Although typhoons do not pass over Bintulu it has been found that they contribute strongly to extreme wave conditions at the harbour site. Typhoons typically appear in the winter period, when they travel westwards from the Philippines over the South China Sea towards the Asian Continent where they are deflected towards NE. The winds on the front side of the typhoons are northerly and strong and they generate waves that propagate towards the south.

Wave Conditions

The wave climate is characterized by swell generated by distant storms over the South China Sea and superimposed by locally generated wind waves. The recordings at Tanjong Kidurong shows significant wave heights generally less than 2 m and mean periods of 7-9 sec. The directional distribution was determined on the basis of offshore wave and wind statistics from ship observations. The result of the analysis is presented by the distributions in Fig. 3. The graph shows that wave heights expected to be exceeded in 1 hour per 1 year, 10 years and 100 years are H_S = 4.0m, H_S = 5.1m and H_S = 6.1m, respectively.

Deep water waves from the north are typically refracted to about 330^o off the harbour site and these waves were found to be more important than waves from the westerly directions.

Tides and Currents

The tides are mixed diurnal and semidiurnal with the diurnal component predominating, Fig. 4. The maximum range is 1.7 m and minimum 0.3 m. The tidal currents off the harbour are running along the depth contours, the ebb current towards 30^o and the flood current towards 210^o. One percent of the time the current speed (mean hourly) exceeds 0.4 m/s.

Long Period Waves

The second field measuring programme was planned to explain the transfer of long wave energy in deeper water to the long waves in the shallow bay of Telok Plan. The long waves recorded in Telok Plan had typical periods of about 3 minutes and height of 0.2 m as shown in Fig. 6. This particular record was obtained shortly after the typhoon Kim passed the Philippines on November 14, 1977, cf. Fig. 5.

The measuring procedure was designed to determine whether the long waves in the bay originated from long waves already existing in deep water ("free" long waves) or were locally associated with wave groups propagating into shallower water.

The hypothesis of free long waves arriving at the site could not be excluded due to the lack of typhoons during the first measuring period. However, the correlation between the long waves in the bay and the wave heights outside the bay, see Fig. 8, convinced to conclude that grouped short waves, cf. Fig. 7, is the predominant source of the measured long waves at Bintulu. The effect of the recorded long waves would therefore be inherent in properly executed model tests with irregular waves reproduced from natural wave records.

The consistency of the measured long wave period and the stability of the oscillation indicate that the long waves are trapped waves.

Wave groups may generate trapped waves if the following preconditions are present.

1) Amplification of the long period wave energy when approaching the site.

2) Local sea bed topography favourable for waves of long period trapping.

The first precondition is attained when the development of the wave group induced long period energy is considered. It is known that a long period wave is associated with wave groups, see Fig. 7.

The amplitude of this long period wave is inversely proportional to the square of the water depth, and it is therefore considerably higher in shallow water, Ref. /9/. At Bintulu an amplification of 3 to 4 in the long wave amplitude occurs between the position of the wave recorder and Tanjong Kidurong.

There are two possibilities for wave trapping at the site of the recorder.

1) Coincidence between the periods of the long waves and the natural periods of the bay formed by Tanjong Kidurong to the north and Bintulu to the south. Such resonance effects are frequently referred to as seiches.

   It is found however, that the natural periods in the bay are significantly higher than the observed long period waves of the order of 3 minutes. Therefore this mechanism is ruled out as a significant factor in the amplification of long waves.

2) Waves propagating along the northern shore being reflected by the local reefs and refracted shorewards again due to the gently sloping sea bed. At Bintulu the conditions are favourable to this trapping of waves along the northern coastline.

The stability of a trapped wave existing along the northern coastline will be affected by the varying water depth normal to the coastline. In fact only waves of certain periods can be stable for a beach of constant slope, Ref. /3/. Waves of these periods are known as edge waves.

Assuming the period of 3 minutes which is dominant in the long wave measurements, the first order edge wave has a wave length that corresponds to a trapped wave along the northern coastline of Telok Plan (Fig. 9).

It will be seen that the amplitude of the edge wave drops off sharply with distance from the coastline. This is consistent with the fact that maximum long wave activity occurs at low water levels. The relative edge wave amplitude levels at the positions of the seiche recorder at high and low tide are schematically illustrated in Fig. 9.

6. SEDIMENTATION IN THE APPROACH CHANNEL

The bottom sediments are very uniform throughout the area of the approach channel and in Telok Plan and consist of soft silty clay. Average concentrations of suspended material close to the sea bed were found to be related to the water depth and the wave conditions. Typical concentrations 15 cm above the sea bed were 200-2000 mg/l for waves up to H_S = 1.2 m and the settling velocities of suspended sediments were between 10^{-4} and 10^{-3} m/s. Results from the field investigations are shown in Figs. 10 and 11.

The concentration of suspended sediment in a thin layer close to the bottom is almost exclusively determined by the wave agitation because the waves exert a much higher shear stress and turbulent diffusivity at the bottom, from which the material is brought into suspension, than the current. Above the thin wave boundary layer, there is a transition to the part of the flow where the diffusivity created by the current is dominant.

Assuming that sea bed material brought into suspension by wave action and transported by currents is the major source of sedimentation in the dredged approach channel, the expected amount of sedimentation was determined by considering the two mechanisms described below:

The first mechanism refers to accumulation of sediments in the approach channel caused by reduction in current speed and wave orbital activity at the bottom of the deeper channel.

The introduction of the channel results in a changed equilibrium concentration profile of suspended sediments, characterized by a lower concentration at the channel bed than at the natural sea bed. As the concentration profile changes and the current velocity decreases, the transport capacity is reduced in the dredged region which results in sedimentation. The other, but less important, sedimentation mechanism is density-currents, induced by the layer of high sediment concentration at the sea bed. When this layer is moved across the side slopes by the current, it runs down into the channel because of the greater density of the sediment than the water.

Model for Sedimentation due to Change of Depth

A mathematical model has been developed at DHI, Ref. /4/, in which the sedimentation rate is calculated by expressing equilibrium between convection, diffusion and settling of the suspended sediment. This model has been applied for various approach channel projects, a.o. Karachi in Pakistan, Ref. /5/ and Forcados Entrance, Nigeria, Ref. /4/. The model is based upon the approximation of applying a constant viscosity, , which has to be combined with a finite velocity at the bottom, the so-called slip velocity, V_b, Ref. /6/. This is very convenient description for analytical treatment of the problem. The general expression for the sedimentation rate, q_r, is

$$q_r = \left\{ q_{10} \cdot \left(1 - e^{-\frac{W}{\varepsilon} \cdot \frac{W}{V_b} \frac{D_1}{D_2} \frac{B}{\cos\alpha}}\right) - q_{20} \cdot \left(1 - e^{\frac{W}{\varepsilon} \frac{W}{V_b} \frac{B}{\cos\alpha}}\right) \right\} \cos\alpha$$

where:

q_{10} = transport of suspended sediments equilibrium conditions at depth D_1.

q_{20} = transport of suspended sediments, equilibrium conditions at depth D_2.

W = settling velocity of suspended material.

$\mathcal{E}$ = current eddy viscosity

V_b = "slip velocity" at the bed.

B = channel width at the middle of the slope.

α = angle between the direction of the current and normal to the channel alignment.

D_1 = present depth.

D_2 = channel depth.

By applying this model, the sedimentation has been determined as a summation of sedimentation rates for combination of settling velocity fractions, water depths, current velocities and wave characteristics and using the statistical distributions of the hydrographic conditions obtained during field investigations.

The estimate of average annual sedimentation for a 240 m wide, 16.5 m deep and 5.5 km long channel was a total of 1.1 million m^3/year with a distribution of average infill as shown in Fig. 12. However, large variations in sedimentation may occur from year to year depending upon the actual hydrographic conditions.

The method applied made it possible to investigate in detail the variation within practical limits of geometrical parameters such as:

- channel width
- channel depth
- side slope
- channel orientation

Conclusions are presented below:

Width :	Sedimentation rates (per unit area of channel) decreases only little with increased width which means that the total sedimentation increases almost linearly with the width.
Depth :	Sedimentation rates increase slightly with the depth.
Side Slope:	Flatter side slopes lead to increased effective channel width and consequently to increased sedimentation.
Orientation:	The sedimentation rate is almost constant for variations in channel alignment up to $\pm 30^0$ relative to the shortest channel perpendicular to the depth contours. Changed orientation, however, results in a longer channel and therefore in increased total sedimentation.

Among the conclusions from the results above is that a reservoir for sedimentation should be provided by overdredging rather than by widening the channel.

7. WAVE DISTURBANCE IN THE HARBOUR

A basic layout for the harbour had been prepared from a number of considerations including topographic, port operational and navigational aspects. It was considered necessary to protect the LNG jetty by a breakwater and it was decided to study the necessary length and orientation of this breakwater in a hydraulic model. The effect of approach channel layout, LNG jetty location and mooring system layout were also studied in this model.

The technique adopted in the optimization study was to determine the movements and mooring and fender forces for a model LNG carrier moored in the harbour. The behaviour of the vessel for different layouts and various combinations of waves and wind provided the basis for determination of an adequate layout in regard to the criteria specified for the project.

Criteria for Design

Two criteria were given for the design:

- An operational criterion set by the flexibility of the loading arms. This flexibility was maximum 2.5 m from the neutral position and this was not to be exceeded more than once in 10 years.

- A safety criterion stating that the vessel should be able to remain at the berth during a 100 years storm including the worst combination of waves and south-easterly wind during 100 years. The mooring forces should be less than 60% of the breaking load (64 t) during this event.

  The safety criterion specifies a very extreme event because of the long approach channel and the hazardous cargo.

Model Set-up

The model was constructed to a linear scale of 1:110 in a 30x60 m model basin. The tanker model was moored with elastic moorings and against fenders simulating the characteristics of prototype mooring equipment, Fig. 13. The irregular waves in the model were simulated from selected long duration natural wave records (corresponding to 1 hour in the prototype), Ref. /7/. These wave records introduce natural wave trains in the model with the only simplification that they have two-dimensional characteristics at the wave generator.

Natural grouping of the waves is thereby ensured and hence the associated water level set-down as described in Section 5. This is valid when compensation of parasitic long wave-phenomena is introduced as was done in the present study, Refs. /8/ and /9/.

Test Results

Mooring Arrangements

The vessel mooring system was tested in order to determine the best layout of mooring lines to ensure a balance between movements and forces that would fit the design criteria. In principle a very stiff system (steel hawsers) will allow little movements and give large forces, while a soft system with synthetic lines will give large movements and small forces. It was found that steel hawsers with 5 m nylon tails and an arrangement consisting of breast lines and spring lines would provide optimal characteristics, Fig. 14, system M2.1.

Influence of Wind

Analysis of wind conditions during extreme wave conditions shows that winds from south-easterly directions occur just as frequently during extreme sea states at Bintulu as westerly winds. Tests with various combinations of wind directions and waves proved that only winds from SE (forcing the vessel away from the wharf) will give rise to larger movements and forces than waves alone. Therefore this unfavourable combination was selected as design condition. The increased movements and forces for this wind direction are due to reduced contact between fenders and the vessel.

Influence of Long Waves

Although the height of the long waves is in the order of typically only 0.2 m, the model tests showed that the long waves were of governing importance for the movements of the moored LNG vessel, notably for surge. The movements are mainly caused by the oscillating slope of the water surface and less by the oscillating currents associated with the long waves.

Dredging limits proved to have significant influence upon the effects of the long waves.

Breakwater Length

The length of the breakwater is an essential element in the economy of the project. Foundation conditions are so poor that the soil has to be excavated down to -35 m and replaced by sand back fill.

The test results demonstrated that it was not possible to provide sufficient protection unless the breakwater was extended into the proposed dredged channel, Fig. 15. For the channel directions tested an important part of the wave energy is travelling along the north slope of the channel as will be described below. For this reason an extension of the breakwater over the side slope is relatively effective in preventing wave energy in entering the port. Because of this effect it was not considered feasible just to displace the channel towards the south to allow for a longer breakwater.

Approach Channel

Waves travelling in a direction with a small angle to the orientation of a channel will be refracted owing to the increased water depth of the channel. When the angle is very small the waves are reflected from the channel side slope, see Fig. 16. This effect leads to a concentration of wave energy (caustic wave propagation) along the side of the channel from which the waves approach and this energy - and associated long wave energy - will enter the harbour basin unless it is effectively cut off by a breakwater. In the present case, where such a breakwater reaching over the channel side slope is not feasible because of the added risk for vessels entering and leaving the port, an alternative method was used to prevent the energy from entering the harbour.

Fig. 17 shows 3 alternatives of this solution which consists of a widening of the approach channel in front of the breakwater. The effect of this is to deflect the wave energy away from the port entrance. As shown in Fig. 1 this layout has been adopted for the present project. Two different channel orientations, 100^o and 120^o were tested and a combination of navigational and disturbance considerations resulted in recommendation of a 100^o orientation.

8. BREAKWATER STUDIES

The breakwater studies included tests on two alternative types of armour for the rubble mound breakwater, tetrapods and quarry stones. Tests were carried out in a wave flume to determine the stability of the trunk section and tests in a basin to describe the stability of the breakwater head. The influence of the dredging in front of the breakwater was considered in the basin model.

It is characteristic for the site that waves are limited in height by extensive breaking due to limited water depths.

In such conditions it is essential to demonstrate that the damage during the expected lifetime of the structure is so small that the breakwater will not gradually be weakened. After having developed the profiles during stability tests with gradually stepwise increasing wave height, the long term stability is documented by a testing procedure using a sequence of varying extreme sea states describing the extreme part of the wave statistics.

As an example the results of a test simulating 87 hours exposure to extreme waves is shown in Fig. 18 & 19. It is seen that the damage on the seaside armour layer occurs during the first few hours exposure and does not develop further, whereas it is the most extreme sea state causing heavy overtopping that gives rise to damage on the rear side of the structure.

The determination of armour size requires a criterion that expresses acceptable maximum damage during the life time of the structure. In this study the criteria were expressed as 1% damaged blocks for tetrapods and 5% for quarry stones. The lower acceptable percentage of damaged tetrapods is due to the greater risk that a tetrapod, when displaced will break itself and maybe other units in the armour layer. In a quarry armour layer a displaced block is not so important because it will normally not break and may still serve well when resting in its new position.

Based on the test results and the criteria described above the recommendations given in the table below were made.

Armour Unit Type		Tetrapods		Quarry Rock	
Expected Lifetime (years)		10-20	50-100	10-20	50-100
Recommended Armour Unit Weight	Roundhead (slope 1:2)	11 t	13 t	15 t	18 t
	Trunk/Bend (slope 1:1.5)	7 t	9.6 t	8 t	10.5 t
Seaside Berm Level, NBD		-6.1 m		-6.1 m	
Trunk/Bend Crest Level, NBD		+6.7 m		+9.8 m	

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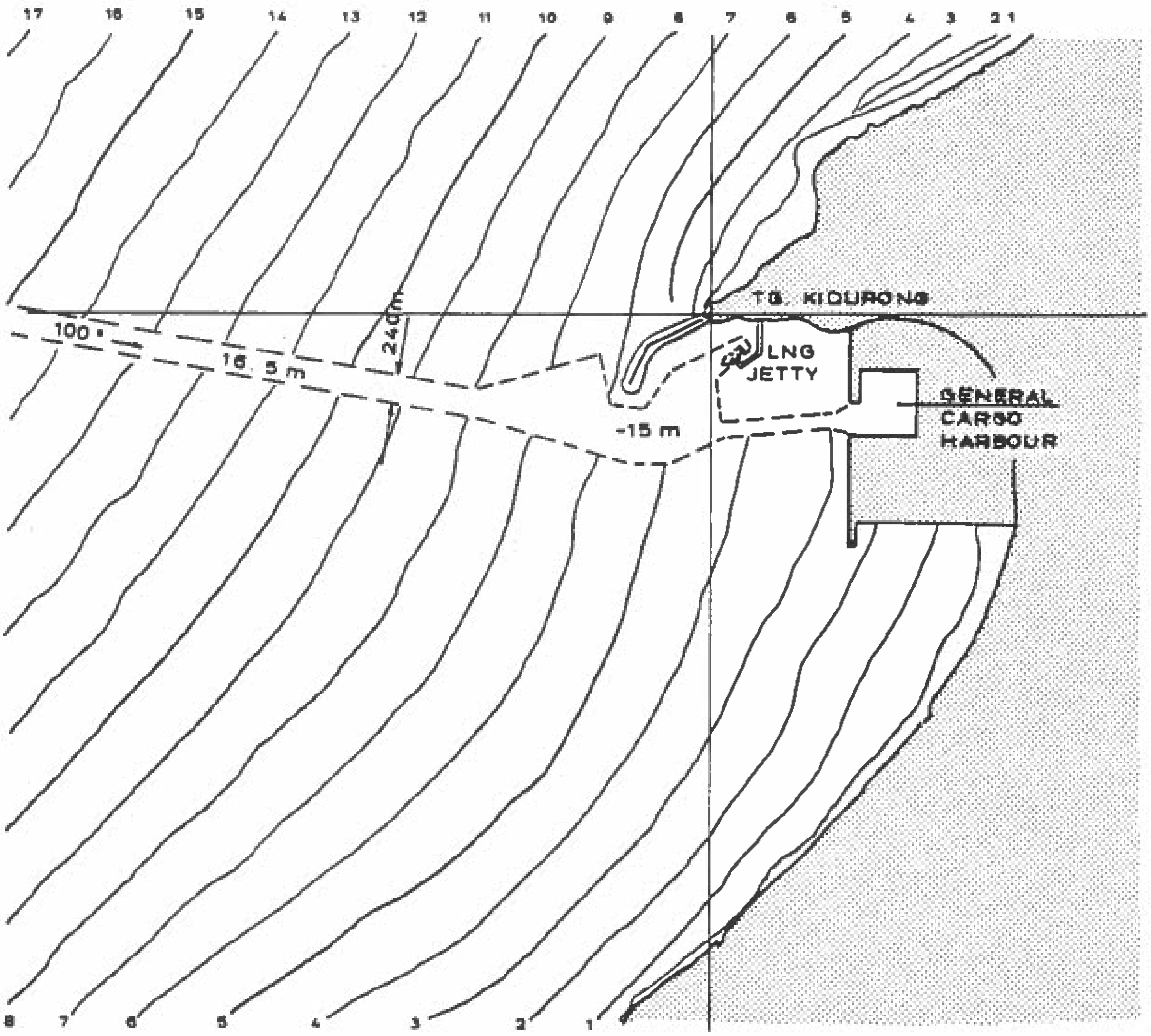

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Fig. 1 Port Layout

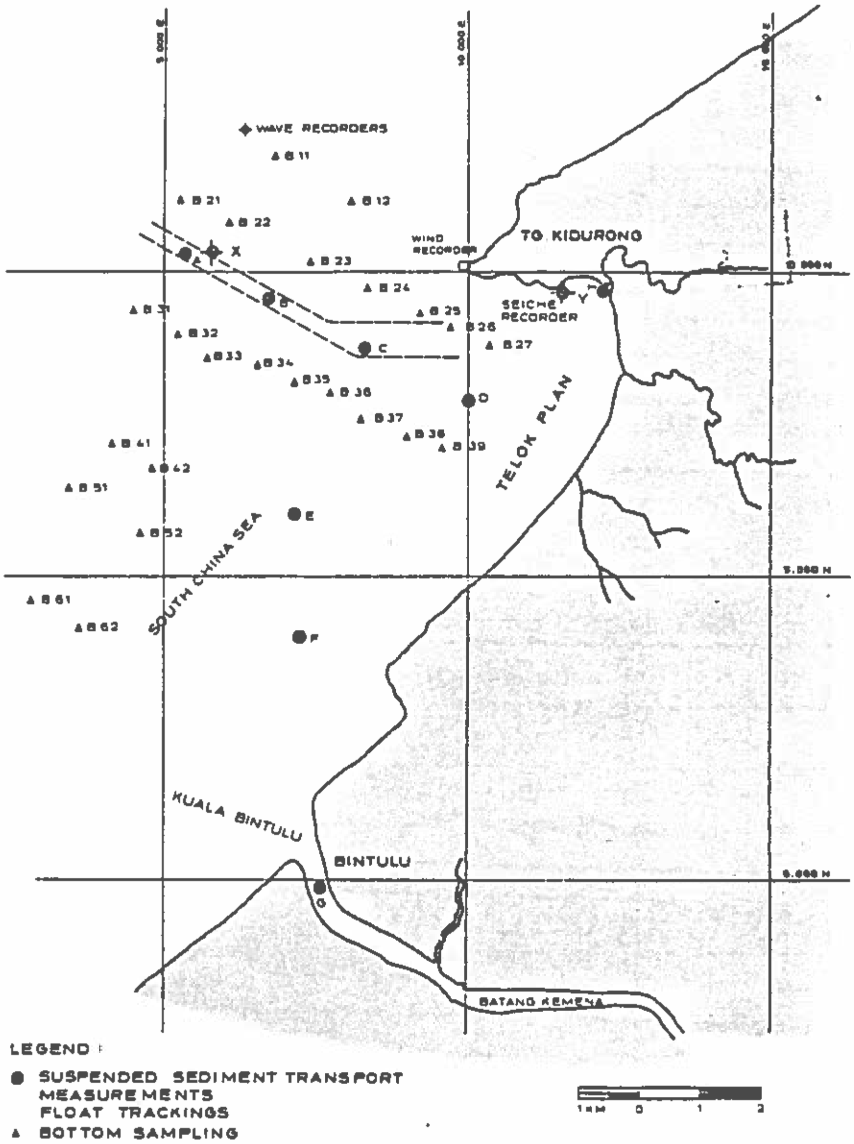

Fig. 2 Field Investigations. Measuring Positions

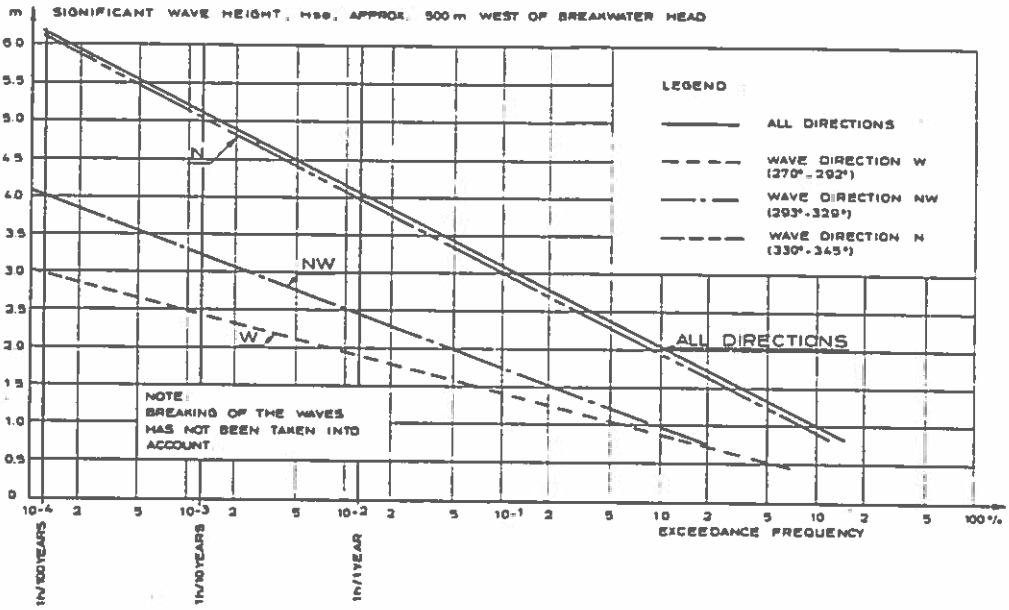

Fig. 3 Wave Height Distributions.

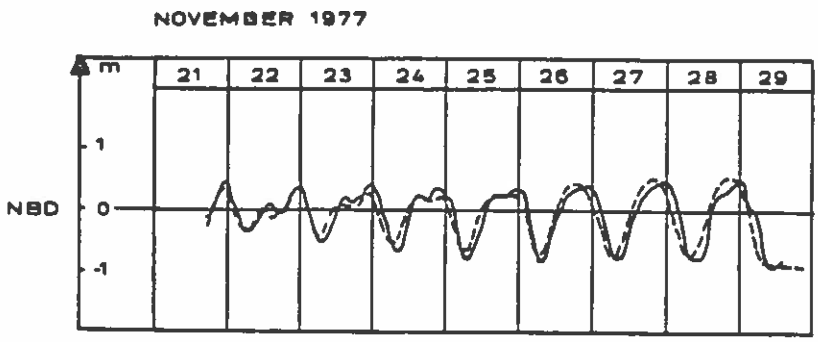

Fig. 4 Water Level Measurements.

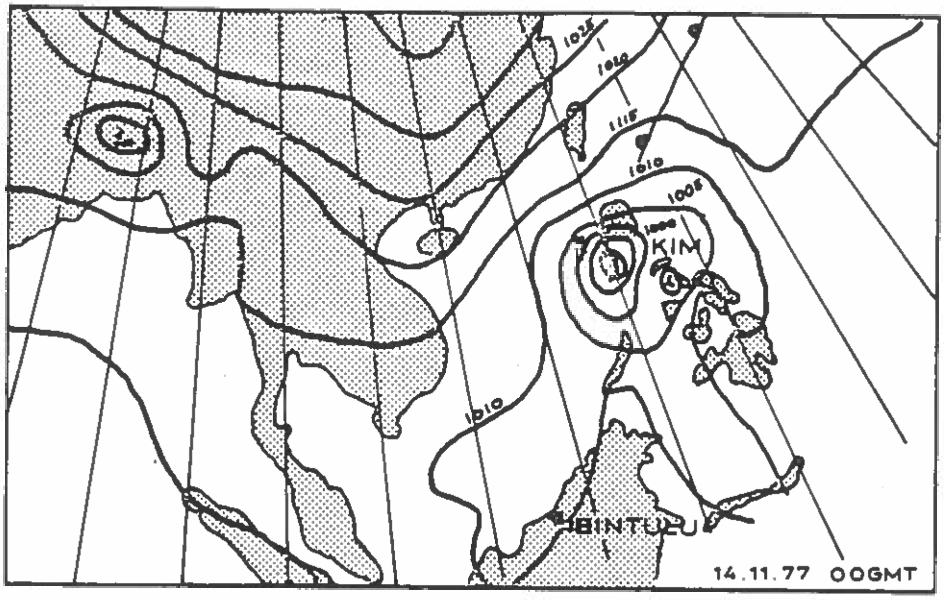

Fig. 5 Synoptic Weather Chart, Typhoon "Kim".

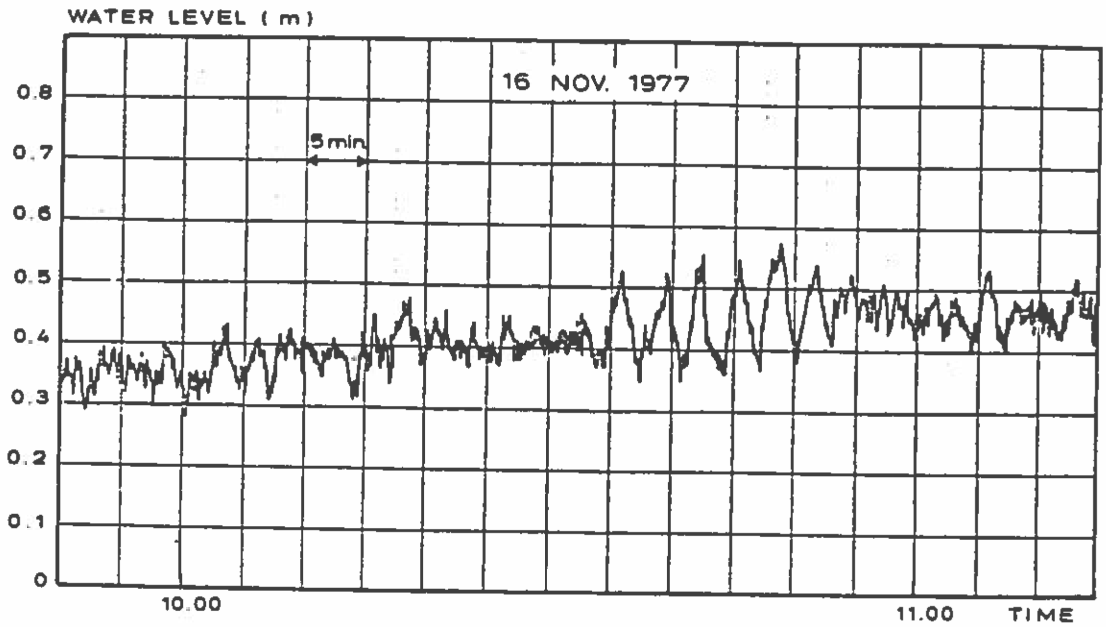

Fig. 6 Seiche Record obtained in Telok Plan.

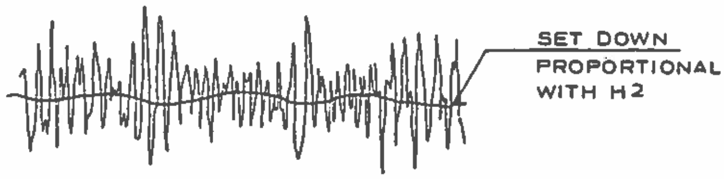

Fig. 7 Wave Groups and Wave Set-down.

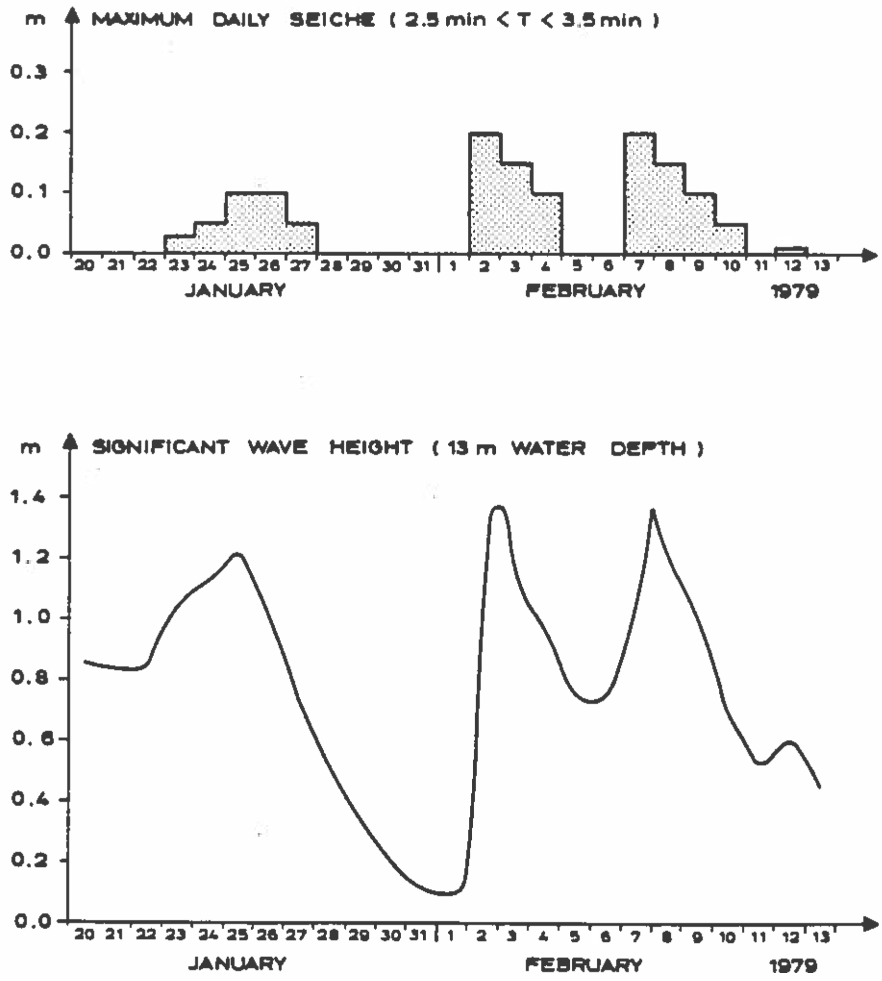

Fig. 8 Correlation between Seiche and Wave Heights.

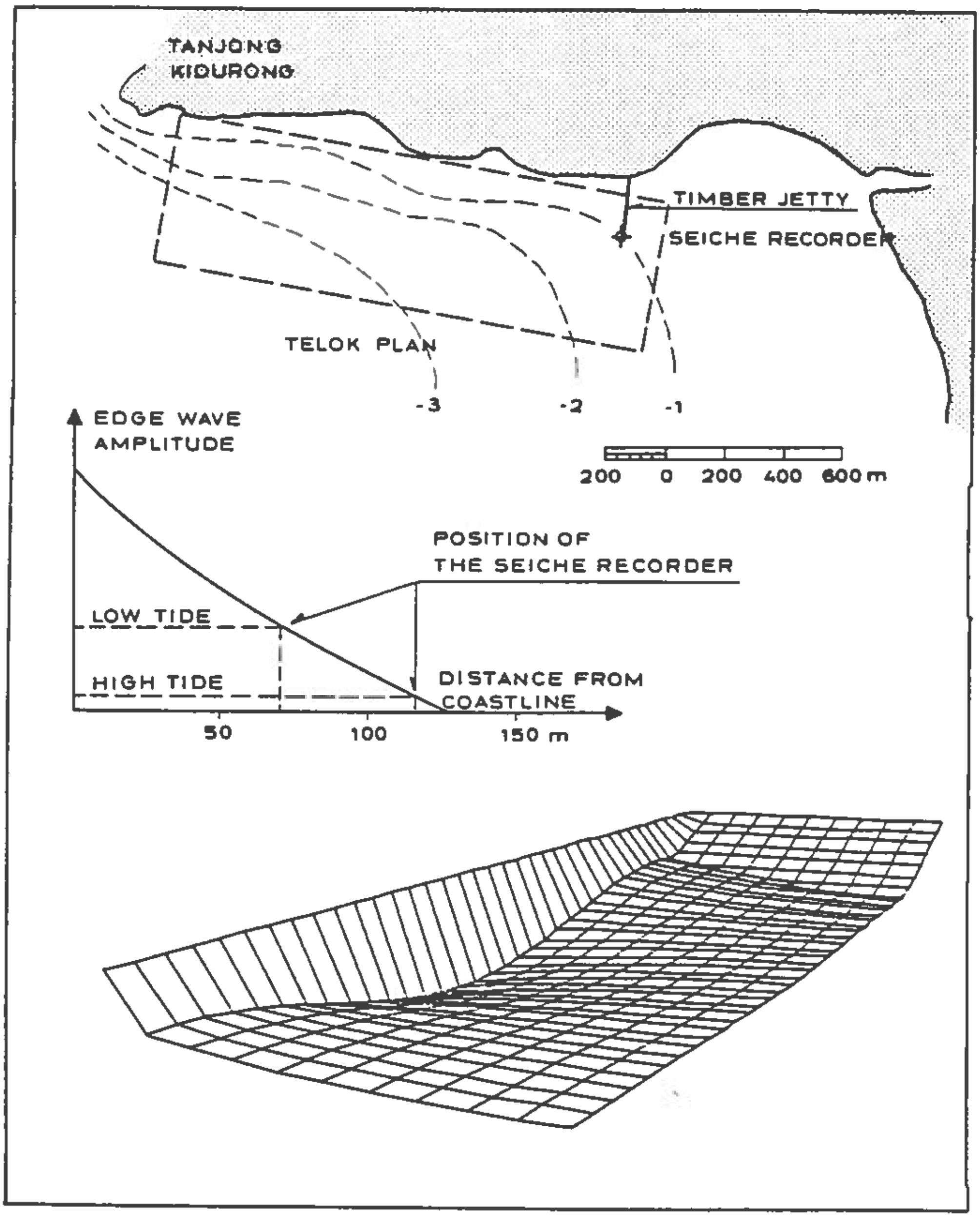

Fig. 9 Edge Wave along northern Shore of Telok Plan.

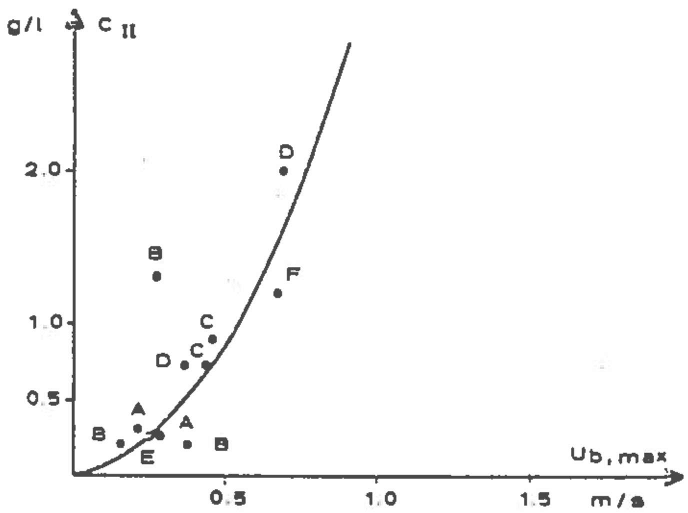

Fig. 10 Concentration of suspended Sediment 15 cm ab. the Bottom as Function of Wave orbital Velocity.

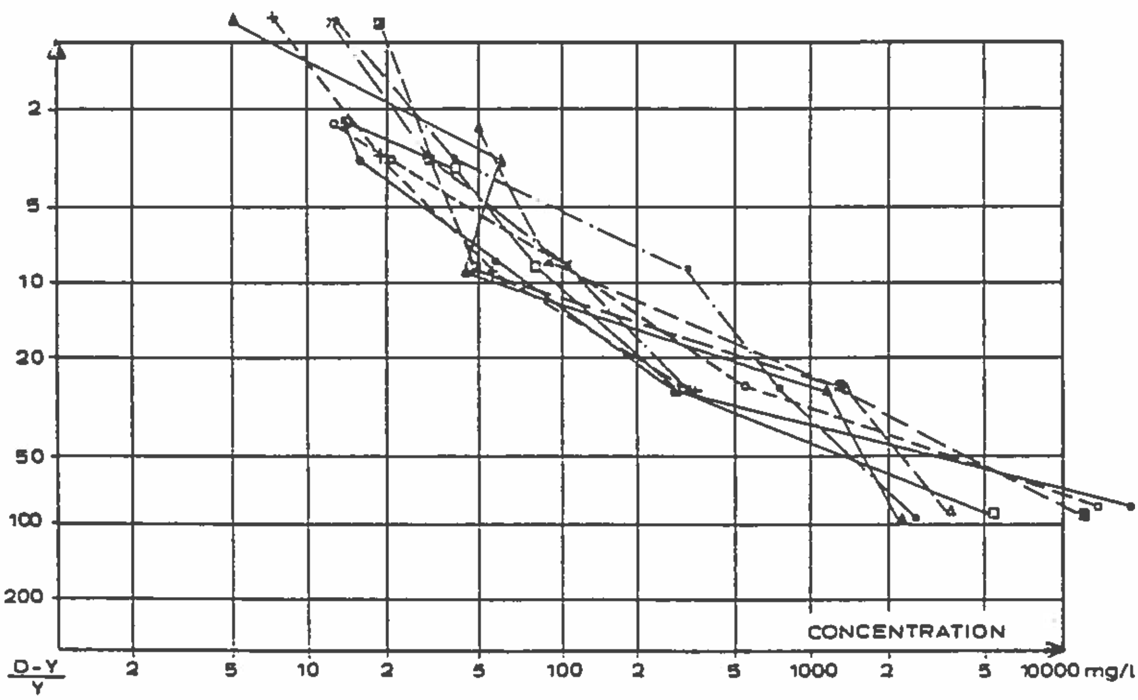

Fig. 11 Sediment Concentration Profiles.

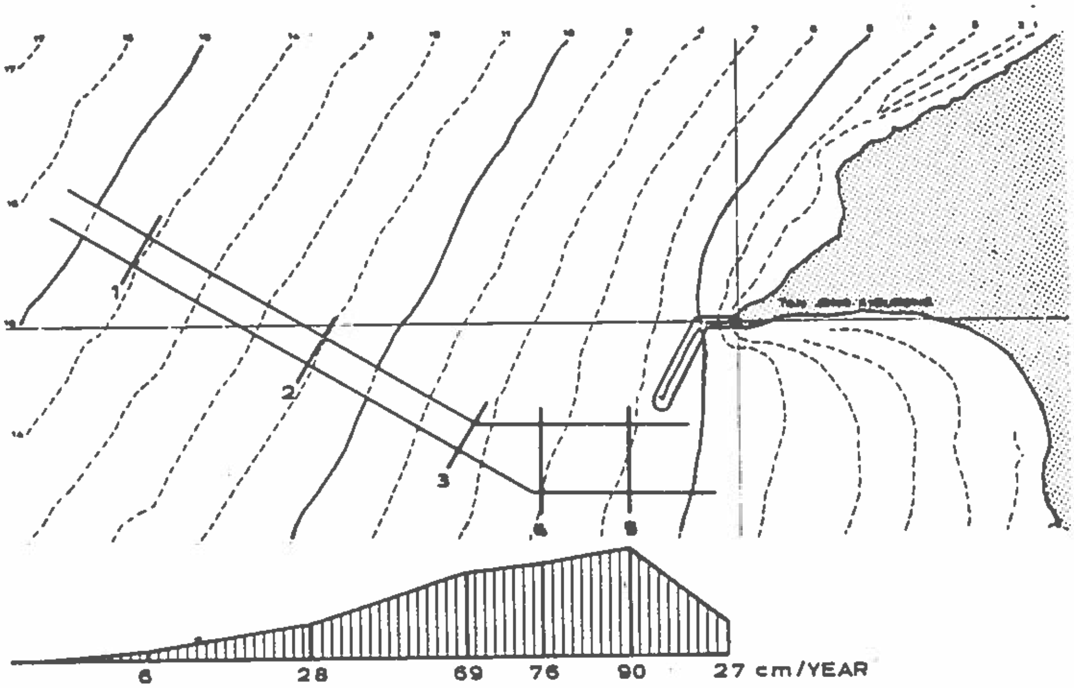

Fig. 12 Sediment Infill, Distribution along Channel.

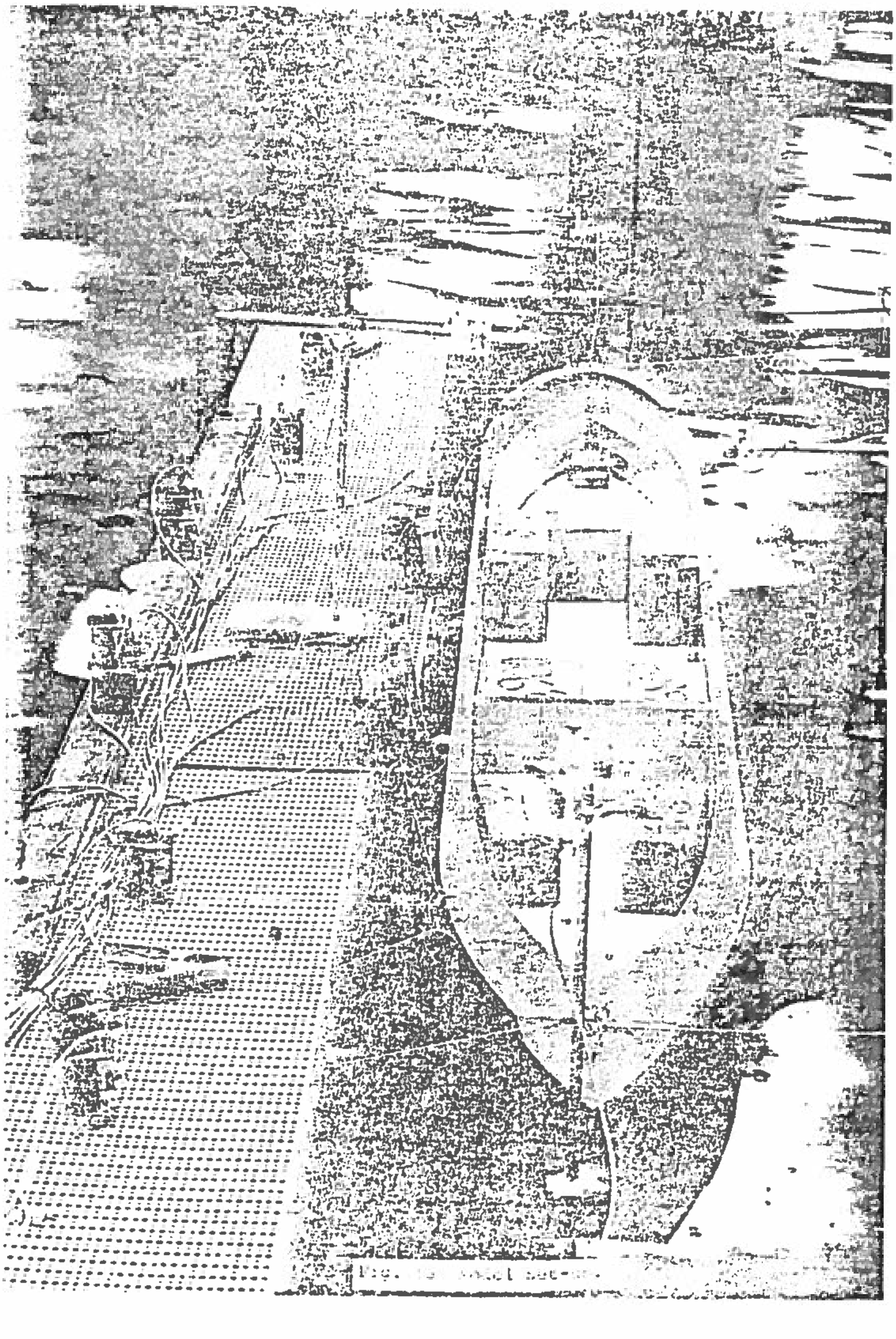

Fig. [illegible] Model set-up [illegible]

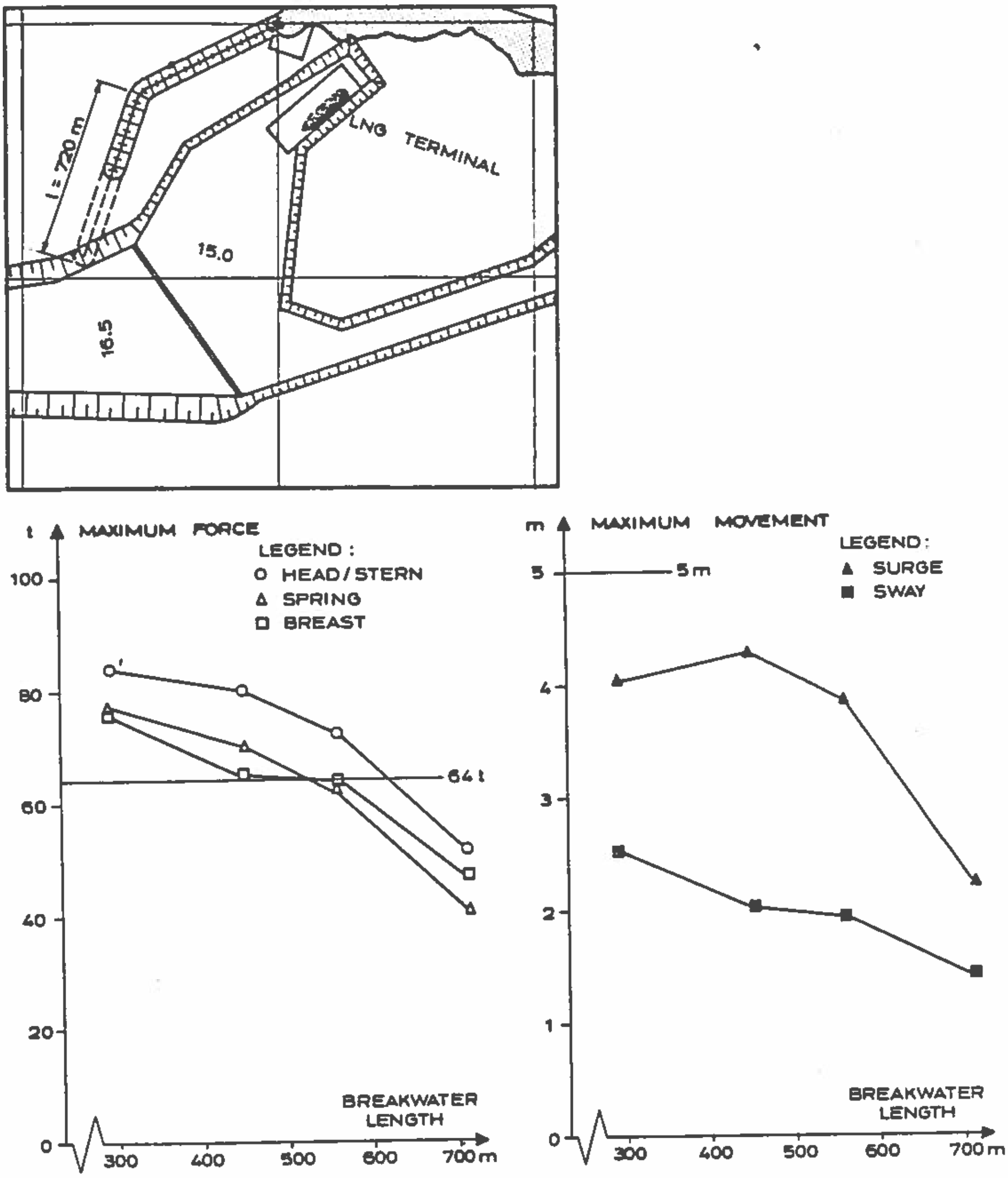

Fig. 15 Effect of Breakwater Length on Conditions for the moored LNG Carrier.

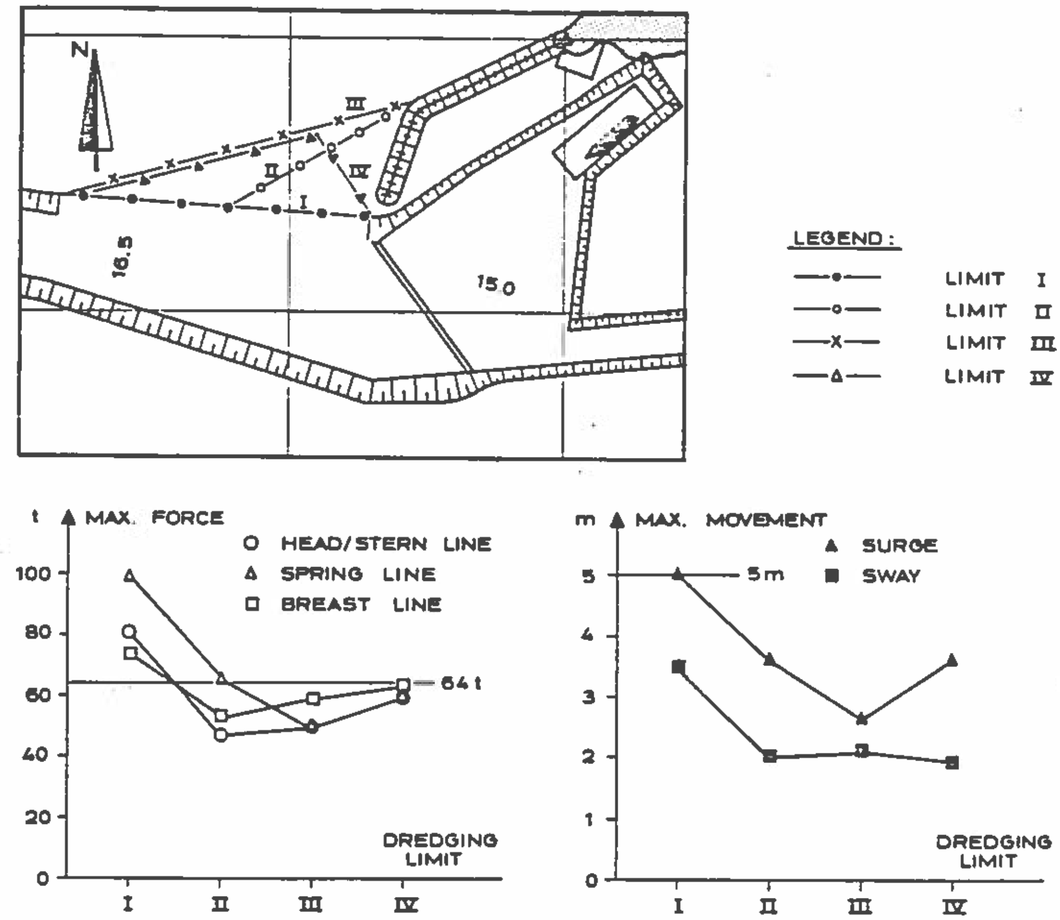

Fig. 17 Effect of Widened Approach Channel.