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CURSO: "INGENIERIA MARITIMA" IX JORNADA DE INGENIERIA CIVIL
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LUGAR: COLEGIO DE INGENIEROS CIVILES DEL GUAYAS
GUAYAQUIL, ECUADOR.

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EL DESARROLLO MARITIMO EN MEXICO

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CURSO:

"INGENIERIA MARITIMA"

ORGANIZADO CONJUNTAMENTE ENTRE EL COLEGIO DE INGENIEROS CIVILES DEL GUAYAS Y LA DIVISION DE EDUCACION CONTINUA DE LA FACULTAD DE INGENIERIA DE LA UNAM.

IX JORNADAS DE INGENIERIA CIVIL

TEMA I:

"EL DESARROLLO MARITIMO EN MEXICO"

"EL ARTE DE LA INGENIERIA OCEANICA"

ING. ROBERTO BUSTAMANTE AHUMADA
12 AL 17 DE JULIO DE 1982.

GUAYAQUIL, ECUADOR

EL DESARROLLO MARITIMO EN MEXICO

En el desarrollo marítimo de México en esta primera plática hablaremos de la evolución de la propia ingeniería marítima y cómo la necesidad de la explotación de los recursos del mar se han venido incrementando día con día; es de todos conocidos que la actividad portuaria, y me refiero en especial a ello, porque fue la inicial; el encuentro con los fenómenos oceánicos tuvo una época de auge a fines del Siglo pasado y a principios del presente Siglo, en esa época, y lo entendemos en forma clara, la tecnología aplicada a los proyectos de construcción y operación era motivo de condiciones -- a grupos externos dado que en México no existía ningun antecedente del trato con el fenómeno del mar, sin embargo, ahí se inicia la historia de la Ingeniería en México y surgió --- un grupo muy distinguido de Ingenieros mexicanos que fueron producto de esa época cuando se construyó el puerto de Veracruz, el de Tampico y el de Puerto México, ahora Coatzacoalcos, Salina Cruz y Manzanillo.

Viene por el movimiento social de nuestra revolución en -- 1910, una total descontinuidad al grado que renace esta necesidad de asomarnos al movimiento portuario hasta la época del -- General Cárdenas y por lo tanto a quienes les tocó reiniciar -- la actividad marítima no tuvieron el eslabón de liga en aquellos Ingenieros que a finales del siglo pasado y a principios - de éste, habían participado en la construcción de muy importantes obras portuarias, se puede decir que del año de 1936 en -- adelante se reinicia con cero de antecedentes nuestra participación en la Ingeniería Marítima. Qué es lo que sucede ?, que venía como primer paso la rehabilitación de los puertos existentes; para ese entonces, el Puerto de Salina Cruz, por un fenómeno tristemente celebre podríamos decir, se encontraba absolutamente azolvado, cerrada su dársena en ese sitio. Se conoce que era el lugar donde se juega Beisbol, lo que ahora es la dársena, había un hermoso campo hecho por el -- azolve natural del oleaje y el Puerto totalmente fuera de operación .

En esa época al final del regimen del General Lázaro Cárdenas, se reabre el Puerto de Salina Cruz,pero tenién dose el problema costero de contínuo azolve con volúmenes muy importantes a dragarse para mantenerlo en operación;- desde el aspecto de los fenómenos físicos,cuál ha sido la evolución por otros motivos y con la organización de De-- pendencias Oficiales contábamos con una buena estadística de vientos; ello en en el aquél entonces, llevado por la - Secretaría de Agricultura. En épocas modernas ese fenó- meno, el conocimiento de los vientos, ha sido incrementado por otro tipo de actividades, fundamentalmente la navega--- ción aérea y también la marítima;. ese fenómeno tan importan te en la concepción de los puertos, pues era un elemento -- con el cual podíamos decir que se contaba otro fenómeno fí- sico de gran importancia; era el de las mareas, a esos prin cipios y ahora hablo de la época del año de 1952, el fenóme no de las mareas era una meta por alcanzar; el que tuvie-- ramos información real y veridica. Entonces qué fue lo - que originó este importante fenómeno, que es para uste-- des obvio el que el fenómeno de las mareas se refiere para- el análisis de una serie de diseños de obras o estudios de fenómenos físicos;en aquel entonces nuestro conocimiento -- de las mareas era muy relativo, teníamos mareógrafos que - operaban en tiempo atrás en un plan absolutamente burocrata con datos no confiables, etc. Únicamente y exclusivamente -

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en los puertos ya mencionados anteriormente viene una promoción y el apoyo de la UNAM, de hacerse cargo de la toma de las mareas en forma directa y posteriormente de su análisis,publicación de predicción, etc., que es la situación que a la fecha tenemos. Nuestra red de mareógrafos, podemos decir que a la fecha es de suficiente amplitud, que tenemos un conocimiento preciso, en cada sitio elementos suficientes para que cuando nos toque analizar el fenómeno de mareas en cualquier punto de nuestro litoral, existen a uno y otro lado, en su cercanía, información ya con valor estadístico del fenómeno de las mareas

Todos conocemos nuestros libros de Predicción de mareas y hemos confirmado la buena calidad de este trabajo, pero nos encontramos también con otro fenómeno muy importante para la concepción y la solución de problemas de cualquier tipo, decimos de la actividad portuaria en su plan de reestructuración, fue el motor que reinicia la Ingeniería Marítima en nuestro país.

Sin embargo, qué sucede ? que día con día se ha venido ampliando la actividad ligada al mar, cierto es que la portuaria continúa con el proyecto de construcción y operación de nuevas unidades en muy diversos sitios en nuestros litorales; viene de ralativamente pocos años a la fecha la actividad pesquera en forma muy importante que no es solamente el refigio del puerto pesquero, sino la explotación de las lagunas litorales con los fenómenos físicos inherentes a su comportamiento hidráulico. En la actividad petrolera no solamente interesa el aspecto costero en el área marítima litoral,en su campo de transportación y de puertos, sino que se da el paso adelante y es la actividad en la cual por necesidad se participa en el estu-

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dío del fenómeno oceánico en sí, por el hecho de tener es-- tructuras de plataformas, de exploración y explotación francamente en mar abierto, ello dá origen a una técnica muy -- especial y a la necesidad de contar con elementos de tipo - estadístico de los diversos fenómenos entre los cuales destaca el oleaje, el que nos interesa por un lado, para el diseño de las obras de protección, los rompeolas, obras de protección de playas, el regimen costero o sea todo aquel fenómeno que modifica la morfología de nuestras playas, sea creando - fenómenos de azolvamiento o fenómenos mucho más graves de -- fuertes erosiones que afectan a áreas urbanas, a otras instalaciones y todo ello tiene como denominador común el conoci-- miento del parámetro - oleaje - Qué es lo que ha sucedido en él, pues bien, así como en el aspecto Vientos, resulta simple el tener información que en el aspecto :marea, también se ha - avanzado satisfactoriamente. No podemos decir lo mismo en el aspecto - oleaje -, el primer paso que se dió fue el de utilizar estadísticas publicadas por otros paises; como es el ----- SEA AND SWELL CHARTS, muy conocido por la mayoría de los presentes; el OCEAN WAVE STATITICS y que al no tener alguna informa -- ción directa del fenómeno del oleaje, no quedaba más que el recurso de utilizar este tipo de información que si bien, tiene - valor estadístico, porque es proporcionado por las embarcacio-- nes que navegan por cierta área oceánica, dividiendo los océa-- nos en una cuadricula de 5°x 5°o sean grandes áreas oceánicas, - considerabamos que al no haber nada, eso era representativo - del fenómeno del oleaje en donde obtenemos estadísticas de di-- recciones de oleaje en forma mensual, estacional y anual etc; información también de alturas de ola y períodos de la misma y habíamos considerado que se había dado un paso adelante en el avance del oleaje. Claro que de no disponer de ninguna in-- formación al momento de tener esa información indudablemente --

que fue un avance.

En épocas posteriores y ésto promovido por quienes tenían necesidades expresas, hablo en su principio de PEMEX, C.F.E., por la necesidad de también construir rompeolas para sus obras de toma de sus diversas Plantas Termoeléctricas y de la Nucleoeléctrica de Laguna Verde.

Se enfrenta también el mismo problema de falta de información respecto al oleaje, adicionalmente y todavía a algo que podemos decir es hoy o es ayer, más bien hoy, se inicia la toma de datos directos de oleaje para apoyar los muy importantes proyectos de Puertos Industriales.

Así tenemos cuatro estaciones y dos más en proceso de instalación en donde tenemos información directa de ese fenómeno. Al igual y con anterioridad en el Puerto de Dos Bocas que proyecta y construye PEMEX, pero qué es lo que nos sucede con esta información de oleaje ? hemos llegado a una primera conclusión: que su relación con los datos estadísticos que se tienen de la información muy general del SEA AND SWEEL CHARTS y del OCEAN WAVE STATICS difieren mucho de aquella que se ha venido tomando en forma directa tanto en magnitudes de oleaje como en direcciones y resulta que es lógico esperar ese tipo de resultados dado que tenemos por decir algo, del Puerto de Salina Cruz, oleaje muy importante en el área oceánica, representada por esta estadística internacional de dirección procedente del Norte;que es lo que nos pasa a la hora de medir la ola directamente ,la dirección que proviene del Norte,viene de tierra o sea que ese oleaje es inexistente para las playas directas;qué nos interesa conocer? incidencia del oleaje, en esa forma observaremos como es una necesidad imperiosa el tener nuestra estadística directa de oleaje en sus características básicas, que son alturas de oleaje, períodos

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de oleaje, direcciones de incidencias y ahora energía del -- oleaje a través de formaciones de trenes de ola, situación que nos resulta de más interés en su aplicación a otra arma que afortunadamente ya tenemos en nuestro país, que son los modelos hidráulicos y este fenómeno de captar nuestra situación real de oleaje, es un reto al cual nos enfrentamos ac-- tualmente; estamos tomando oleaje. Aquel que lo necesita, - por decir algo, la Secretaría de Comunicaciones por un lado y por otro lado, PEMEX, Comisión Federal de Electricidad por otro lado y son ellos,porque son las Dependencias que tienen fuerza económica; no porque otras Dependencias no tengan mismas necesidades, podemos poner un ejemplo.

El aspecto de los puertos pesqueros, qué sucede ?, al final - de un puerto pesquero, el fenómeno marítimo, la magnitud de - la ola, etc., son idénticas que si se trata de hacer el puerto grande, entonces misma necesidad se tiene para la concepción - y localización de nuevas unidades pesqueras que estén en franco desarrollo y al igual en una actividad que está por nacer - como es el puerto turístico; nos sucede exactamente misma si-- tuación. El fenómeno físico feneral de oleaje, mareas, corrientes, vientos, etc. inciden en idéntica forma al tener un- puerto de juguete, como podríamos llamarlo, que es el puerto turístico, pero con idéntico concepto en su concepción, en análisis de resolver en forma económica y adecuada el problema físico que hay que resolver, de protección, que no erosione zonas adjuntas, etc. y que obviamente la actividad turística que no - ha tenido tradicionalmente una vocación hacia el puerto de de-- portes náuticos, pues a su vez carece de medios, este reto - simplemente lo pongo a consideración de este distinguido audi-- torio.

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Una necesidad nacional: el que organicemos a nivel país todo lo que corresponde a ir planeando esta estadística de oleaje y que en función del tiempo, así como en aspectos hidrológicos, veamos hermosas estadísticas . De mucho tiempo atrás - el día de mañana nuestra información directa de características de oleaje tengan mayor estadística y podamos tener las tesis - para equivocarnos menos.

Otro aspecto interesante es el aspecto teórico, --- cuál ha sido la información en el conocimiento del subsuelo tanto en tierra como en zonas cubiertas por el agua.

Ha sido una situación parecida a la que hemos expuesto de los otros fenómenos.

Recuerdo épocas en donde la geología era considerada como algo de brujería, algo en que no había necesidad de porqué gastar dinero; encontramos en nuestra actividad profesional una total y negativa reacción a hacer estudios geológicos porque anteriormente no se usaba,en que hubiese este tipo de análisis. - Ahora tener una conciencia completamente clara de su importante - necesidad.

Qué sucedía al no contarse con ese tipo deinformación, pues necesariamente errores grandes y cuando se empieza a avanzar en ésto, se participa de la geología en zonas de tierra y en zonas de mar. Era muy complejo y a la fecha sigue siendo complejo, no porque no se tenga la idea de qué es lo que requerimos, si - hablamos de los datos geológicos, me refiero a la mecánica de -- suelos. Cuando a mí me tocó estudiar,esa materia no existía.

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Surge en esa época como una materia optativa, quiero decir ésto, que de esa época, estoy hablando del año de 1950 eso es cuando participamos en estudios geológicos y todos los -estudios correspondientes a la mecánica de suelos. Cuál es --nuestra situación actual? bien, la importancia de la geología-es un concepto claro para todo tipo de proyecto, pero el problema de obtener datos geológicos en la zona de mar sigue siendo a la fecha, un problema que solamente en aquellos proyectos muy -importantes es factible que téngamos ese tipo de información debido en forma simple, a lo especializado del equipo que se requiere y en consecuencia a lo costoso de ese equipo. Esto se ha ---ameritado y se ha hecho el esfuerzo económico de tener información geológica para los proyectos muy importantes de Puertos Industriales. Difícilmente en algún proyecto de algún puerto -pesquero, y no dígamos turístico ,pues es información la cual -sólo con supuestas analogías, etc., es que en los reconocimientos muy superficiales, tenemos que aplicar el ingenio a efecto de -tener que resolver las obras sin los suficientes elementos de juicio al respecto.

Allí tenemos ese otro aspecto que va encaminado fundamentalmente a la enfilación y participación de grupos que se dedican a la Ingeniería Oceánica para ir contándose en función del tiempo, con mayores elementos, y que podamos hacer los estudios que se requieren en mejor forma y participando en forma directa con equipo propio, con equipo nacional y que no téngamos que recurrir,al carecer en nuestro país, de recurrir al apoyo de otros paises y que necesariamente al tener que traer embarcaciones especializadas, en fin, pues nos resultan acciones caras,pero no queda otra solución más que ella ,mientras el propio país no disponga de los medios --para ese tipo de acciones.

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En el aspecto de dragado, también viendo la enfilación del dragado, Qué es lo que sucede ?, que ese atorón de varias - décadas que se tuvo en la operación portuaria, pues obviamente - la actividad de dragado tuvo misma situación, lo primero que -- surge qué es ?, el dragado de mantenimiento; el que puertos como Salina Cruz, operacen, el que puertos sujetos a un contínuo azolvamiento por estar en un río, me refiero al Coatzacoalcos, pero básicamente al Río Pánuco, pues esa actividad nace como una necesidad imperiosa ,qué pasa en ello ?, bien el tipo de dragas para dragado de mantenimiento, sonde la concepción de utilizar el material dragado para rellenar terrenos, como es en el caso de proyectos nuevos, el tipo de draga que se empieza a utilizar para -- esa rehabilitación y habilitación de puertos existentes, pues el tipo de draga auto propulsada, o sea draga con tolva que tendría que salir mar afuera y depositar en algún sitio adecuado.

Ese fenómeno de detectar cuál era el sitio adecuado fue - y resultó ser motor para participar en los estudios del comportamiento general de depósitos en " X " sitio y hacía dónde es que se irían, y caminar por la acción de las fuerzas vivas,principalmente de oleaje, de corrientes dirigidas por el viento, de mareas, etc., y surge como necesidad de análisis, el aspecto del comportamiento de ese tipo de dragado.

Datos específicos: Tampico, como lo he dicho y básicamente el de Salina Cruz que era un puerto que seguía siendo victima de un muy importante azolve en muchas épocas, por encima de las necesidades que el país tenía para sacar ese material que se había introducido a la dársena.

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Viene la participación por falta de elementos, por falta de medios de que hay que dar solución a este tipo de problemas y poco a poco se va resolviendo al grado de que podemos pensar que a la fecha esos problemas han quedado absolutamente bajo control.

En el caso de Salina Cruz, por una clara concepción del -fenómeno costero de la acción de la ola sobre la conformación --playera. Por la experiencia con datos de volúmenes que se tenían ya por muchos años de aquello, que año con año se dragaba; la --idea clara de cuál era el gasto sólido litoral, etc., y se llegó -a una solución que a la fecha no existe ningún problema de dragado ahora con la construcción del muy importante Puerto Petrolero y -a corto plazo, del Puerto Industrial en esa zona, resulta que ese problema, es un problema que podemos decir, que pasó a la histo--ria con una participación importante de la técnica mexicana, habiéndose tenido necesidad de recurrir a sistemas ingeniosos de tipo ---económico a base interpretación de planos de oleaje, habiendo creado el sistema gráfico de comportamiento de playas que nos daban ---ideas muy generales,pero volviendo a lo mismo que de eso o nada,eso era preferible, viene un avance más importante cuando a nivel país empezamos a participar en la Ingeniería Experimental encaminada --precisamente al comportamiento de playas y al comportamiento de las estructuras, para mejor resolver los problemas de agitación y al -diseño ya al detalle de los canales de ola, para definir la sección transversal de su estabilidad, para diferentes condiciones de rom--peolas u obras mas pequeñas, para protección de playas;.

Este fue seguramente el avance más firme que en los últimos 20 años se ha dado y su evolución a épocas recientes donde ya contamos con diversos laboratorios cada vez con más presencia técnica, con más intercambio; como está el del Instituto de Ingeniería de de la U.N.A.M., el del I.P.N., el de la Dirección General de Obras Marítimas,dependiente de la S.C.T. y tenemos inicios de laborato--

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rios de otros lugares de provincia donde creemos que este paso haya con fines docentes de difusión ,la técnica de la Ingeniería Experimental y el día de mañana redituarán para una mejor con-- cepción de la obra marítima, para una mayor economía en su cons- trucción,

En el aspecto de estructuras y de ahí hablamos de estruc- turas en zonas de calma dentro de los puertos también ha habido - una evolución, imaginense ustedes , cuando era la epoca que ha- bía que diseñar un (X) muelle y se consideraba que el estu-- dio geológico era inútil, -- que cantidad de errores no fueron los que se acumularon por ese subdesarrollo en el que se inició;- día con día se ha venido avanzando en ello y su participación en- estructuras cada vez más complejas por un lado es la clase de es- tructuras, es el muelle de cualquier tipo, dependiendo de las con diciones geológicas que vamos encontrando en cada lugar por la -- participación de otro tipo de actividades como son las obras de - toma que requiere la Comisión Federal de Electricidad, como son - los diques secos que a la fecha construye Astilleros Unidos en su actividad de infraestructura para fomentar la muy necesaria indus tria naval en México, en sus aspectos de reparación de buques y de construcción, ya que el país está entrando en serio en esa activi- dad, pues ha sido satisfactorio que ese tipo de estructuras - son estructuras que en su construcción, son totalmente nacionales - o sea que son producto de la técnica mexicana, de los constructo-- res mexicanos que están atacando proyectos que en otra época podría mos considerar como muy ambiciosos.

Bien, en resumen la infraestructura portuaria, haciendo a un lado - lo que queda fuera de la costa, podemos resumir que básicamente te- nemos dos tipos de problemas a resolver, uno que corresponde a las obras de protección que son los rompeolas en sus aspectos de esta- bilidad por un lado y de equilibrio del regimen de la costa, dado que cualquier obstáculo que pongamos viene a alterar ese regimen -

natural del transporte de material, por efecto fundamentalmente del oleaje y eventualmente de corrientes.

Ese es un capítulo muy bien definido, la obra de protección - en su concepción conjunta, repito, su situación geológica en el aspecto estructural, el aspecto de su comportamiento para resolver la agitación y dejar zonas operables dentro de áreas internas protegidas; los rompeolas y su aspecto de no alterar negativamente a ese regimen litoral. En ello podemos resumir a qué le vamos a enfocar para las obras de protección.

En el aspecto de dragado, necesariamente viene el otro capítulo importante que es adecuar las profundidades para las necesidades que se requieren, día con día, por desarrollo general del país; estamos cayendo a la necesidad quizá un poco en forma -- anacrónica porque lo hubieramos de haber hecho anteriormente - de adecuar las profundidades de nuestros puertos a las necesidades comerciales de navegación. Esto traé en sí como consecuencia que.la actividad del d-agado es muy importante, primero por el excavar el material bajo el agua, el segundo, el - tratar de aprovecharlo para habilitar zonas bajas que tenemos alrededor de los puertos, o en su defecto, si nos vemos obligados a depositar el material en el mar, que sigue habiendo casos de este tipo en que los depositamos con toda seguridad en - aquellos sitios donde no va a originar daños y problemas a terceros.

Bien, este aspecto del dragado nos enfrentamos a que ahora hay que dragar en períodos muy cortos volúmenes mucho más importantes de aquellos tradicionalmente lo hacíamos; también traé una consecuencia en su evolución, en la medida en que disponíamos de equipo. Ese ritmo de desarrollo del país y ese ritmo de - desarrollo del dragado a la fecha, al tener que resolver en - lapsos cortos problemas de gran volúmenes de excavación, aje- nos otra vez a la necesidad de recurrir a quien tiene el equi- po adecuado y que en todo caso son empresas extranjeras en -- nuestro país, tenemos ahorita en este preciso momento varios ejemplos de ello y necesariamente es una labor conjunta de la concepción de la programación del dragado y la participación- de grupos privados y oficiales para resolver en mejor forma -- en beneficio de nuestro país, este fenómeno del dragado que ha tenido una expansión brutal; podríamos decir en los últimos -- años, pero qué se bislumbra, que ésto va a continuar en los- próximos años, tanto en proyectos que están en proceso , como por proyectos que están a nivel de estudios justificativos.

Buena parte de las conferencias que ustedes van a escuchar -- irán al análisis del detalle del aspecto estructural de los - rompeolas. Ahí también hemos tenido una evolución en el sen- tido de las obras de protección que nos tocaba realizar en años pasados, eran obras relativamente pequeñas y ahora nos presen- tamos a obras a mayores profundidades, mayores volúmenes de -- obra y de menor tiempo para su ejecución. Ahí tenemos otro re- to que es el adecuar la tecnología mexicana en su gama total - de su concepción, al uso de la Ingeniería Experimental hasta su construcción y operación , en su caso, entonces ahí verán las - distintas técnicas empleadas y también observarán que ha habido una cierta evolución contando en todo momento con lo que a tra- vés de la literatura mundial accesible, se tiene para resolver- estos problemas el avance de la Ingeniería Experimental y cuál es la situación actual en la que vamos, - que necesitamos - seguir avanzando para obtener proyectos absolutamente eficientes y seguros a menor costo en las siguientes conferencias; lo que aquí he expuesto en una forma general, seguramente va a ser analizado en diversos capitulos en forma más detallada, para - que tengamos el concepto global de adónde vamos; qué es lo que tenemos que apoyar cada uno de nosotros en el campo en que estamos colaborando para que nuestra Ingeniería Océanica, día con - día continúe avanzando como hasta la fecha .

Por las propias necesidades del desarrollo del país, ha sucedido al aumentarse las necesidades , necesariamente el reto, básicamente para ustedes, me refiero muy específicamente para los jóvenes, es un reto amplio, de gran responsabilidad y que sola--- mente con el esfuerzo personal y con la colaboración del conjunto, podemos salir adelante.

G r a c i a s .

CURSO:

"INGENIERIA MARITIMA"

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LOS ASPECTOS RELEVANTES EN EL DISEÑO, LA CONSTRUCCION Y LA SUPERVISION DE ROMPEOLAS

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R E S U M E N

Las fases del diseño y la construcción de un rompeolas, requieren la debida coordinación a través de la Residencia de Construcción, la cual tendrá la responsabilidad de que la obra se ejecute precisamente de acuerdo con los planos del proyecto, respetando y cumpliendo las especificaciones técnicas y de control de calidad de los materiales, así como los relativos al transporte y la colocación de estos últimos.

Se definen las funciones de los elementos constitutivos de un rompeolas y se precisan los criterios básicos para el suministro de roca, la fabricación de elementos precolados de concreto y la fabricación de roca artificial. Se comentan las posibilidades que ofrece el empleo del suelfacreto y el uso de los geotextiles. Se hacen recomendaciones para el transporte de la roca y los elementos precolados y se tratan los aspectos más importantes de las operaciones de colocación de materiales en el rompeolas.

Finalmente se hace hincapié en la necesidad de contar con la Residencia de Construcción.

Se incluye la Bibliografía de estudios y publicaciones reciente sobre la tecnología del diseño y la construcción de los rompeolas.

" Se ti addiviene ti trattare delle acque, consulta prima la esperienza e poi la ragione ". (*)

Leonardo da Vinci
1452 - 1519.

1. INTRODUCCION

Como en toda obra de ingeniería, el proyecto y la construcción de un rompeolas a talud comprende diversas fases, desde la concepción y definición de su finalidad hasta su construcción. En este proceso cabe distinguir dos etapas sucesivas: el diseño y la construcción. La primera incluye la ingeniería básica, la ingeniería de detalle, la preparación de las especificaciones y la formulación de los documentos necesarios para concursar la obra. Con esta última información, se inicia la segunda etapa; o sea, la de construcción, durante la cual las empresas constructoras que participarán en el concurso se enteran del tipo de trabajo a ejecutar mediante los planos, especificaciones, requisitos de calidad y el plazo requerido para ejecutar la obra. Con esta información las empresas están en condiciones de formular el presupuesto y preparar la proposición solicitada por el dueño de la obra. Una

(*) Cuando tengas que lidiar con el agua, consulta primero la experiencia y después razona.

vez elegida la empresa constructora, procede ésta a la movilización de sus recursos para iniciar la construcción.

A fin de coordinar debidamente las fases de diseño y construcción, resulta imperativo organizar oportunamente la Residencia de Construcción que tendrá a su cargo la responsabilidad y la autoridad para supervisar la realización de la obra a fin de que ésta se ejecute de acuerdo con los planos, las normas de control de calidad y las especificaciones definidas en la fase de diseño, mediante lo cual será posible satisfacer la finalidad que debe cumplir la obra en cuestión.

2. CRITERIOS BASICOS DE DISEÑO Y CONSTRUCCION

Una vez definidas las funciones de la obra, se establecerán los criterios básicos de diseño que regirán a lo largo de la fase de ingeniería de detalle y los cuales debe conocer y hacer respetar la Residencia de Construcción durante el proceso de ejecución del trabajo. Es muy importante que el constructor conozca y esté conciente de que la obra que realice cumpla con los criterios básicos de diseño y respete las normas de calidad contenidas en la documentación de Concurso, que a su vez, le sirvieron de base para formular su oferta.

Resulta de capital importancia que durante la fase de diseño se tomen muy en cuenta los aspectos constructivos, a fin de que el proyecto pueda ejecutarse en el plazo previsto y al más bajo costo posible, considerando los diversos procedimientos de construcción, así como los equipos y recursos disponibles para el tipo particular de obra.(18) y (19) (*)

(*) Las Referencias Bibliográficas se identifican con un número entre paréntesis.

Concretando, en el caso específico de un rompeolas, su finalidad primordial estriba en constituír una barrera permanente contra el oleaje de mar, a fin de crear en forma artificial una zona protegida y en calma en la que puedan realizarse en forma expedita y económica las operaciones portuarias, en un sitio en que las condiciones naturales no lo permiten.(13), (31) y (8) Para lograr esta finalidad, el rompeolas deberá ser capaz, en primer lugar, de resistir durante un largo tiempo el embate de las olas. En segundo lugar, es indispensable que tal barrera sea lo suficientemente impermeable para impedir que se transmitan a la zona abrigada del puerto las fluctuaciones de nivel que ocurran en el lado externo por efecto del oleaje. Las funciones antes definidas se obtienen económicamente con un rompeolas a talud, constituído simplemente por un dique de enrocamiento formado con dos elementos básicos: la coraza y el núcleo. La coraza consiste en rocas o elementos precolados de concreto, con el peso y el espesor necesarios para resistir el embate del oleaje. En virtud de su alto costo se procura reducir al mínimo el volumen de este elemento. El núcleo , que no estará expuesto a la acción directa del oleaje, puede y debe construírse con los materiales que en forma más económica se disponga cerca del rompeolas, como arena, grava o el producto integral de una cantera, con tal que, mediante una razonable graduación de sus partículas, se obtenga una masa relativamente impermeable y poco deformable. (Ver Fig. 1).

Para asegurar la formación y permanencia del núcleo deberá cubrirse con un material que actúe como filtro, para evitar que sus partículas de menor tamaño emigren a través de los grandes huecos de la coraza. Comúnmente a este filtro se le denomina capa secundaria.

El núcleo, así construído y protegido, servirá también como la base del camino que se necesita durante los procesos de construcción y mantenimiento de la obra, así como para disponer de un acceso a los muelles que se coloquen adosados a los rompeolas. (46), (10), (13), (9), (8), (12), (43) y (38)

Para reducir el costo del rompeolas conviene que la corona del camino de construcción sea lo más angosta y baja posible, a fin de permitir el tránsito expedito de los camiones y la operación de las grúas que se requieren para colocar por vía terrestre los materiales de la obra. (46) y (39)

Por el lado interior del puerto, en el que el oleaje será menor, se requiere también proteger el rompeolas con una coraza de roca más ligera que la del lado exterior, la que a su vez debe apoyarse en una capa secundaria o filtro que arrope y proteja al núcleo. Cuando el rompeolas sea del tipo rebasable deberá verificarse que tanto la coraza en el lado interior como la corona, no resulten erosionables por este efecto. (8) y (14)

Otro elemento básico en el diseño de los rompeolas lo constituyen los atraques al pie de las corazas, que tienen la doble función de servir de apoyo a estas últimas y actúan como defensas contra la erosión en la base de los taludes de la estructura. (1), (11), (13) y (14).

Por último, dependiendo de las características del fondo marino en que se desplante el rompeolas, convendrá incluir como parte del proyecto la construcción de una plantilla de roca en el desplante de la obra, que restrinja la migración del material fino del fondo a través de los huecos de la coraza. En casos extremos de fondos arenosos muy finos, y según la importancia de la obra, podrá justificarse el empleo de un filtro a base de un geotextil, en vez de la plantilla de roca.(35), (42) y (24)

Para limitar la erosión al pie de la estructura, que puede inducir el deslizamiento de los materiales colocados en el talud de la obra, resulta muy recomendable extender generosamente la plantilla de roca, con o sin el geotextil, para formar un delantal, adelante del pie del talud del atraque de la coraza (14), (39), (10), (11), (13)

Teniendo en mente las funciones básicas que desempeñarán los diversos elementos constitutivos de un rompeolas, que en términos muy generales se han descrito, resulta posible precisar los criterios de diseño para su dimensionamiento, así como definir las especificaciones técnicas para el control de calidad de los materiales y las normas que deben respetarse durante el proceso de colocación de los mismos.(39 y (14)

En los párrafos subsiguientes se comentarán los aspectos más relevantes que conviene tomar en cuenta al seleccionar y fabricar los materiales, así como las recomendaciones relativas a su transporte y colocación en el rompeolas. Cabe advertir que todo lo que será expuesto es igualmente aplicable a la construcción de escolleras y espigones.

3. SUMINISTRO DE ROCA

Dado que la roca es el material que constituye la mayor parte de los rompeolas a talud, resulta de particular importancia la localización y el estudio de las canteras, de las cuales pueda extraerse roca de la calidad adecuada y en cantidad suficiente, que se transporte en forma expedita y económica hasta el sitio de la obra.

independientemente de su origen geológico, la roca que se utilice en los rompeolas debe ser resistente al ataque del agua de mar; de alto peso específico; resistente a la abrasión y con un mínimo de resistencia a la compresión. Para calificar estos requisitos, en la siguiente tabla se indican las pruebas a que debe sujetarse este material y los límites de aceptabilidad correspondientes (39) y (43)

PRUEBAS FISICAS PARA EL CONTROL DE CALIDAD Y LIMITES DE ACEPTACION PARA LA ROCA DESTINADA A ROMPEOLAS

TIPO DE PRUEBA	NORMA ASTM	LIMITES DE ACEPTABILIDAD
Intemperismo acelerado (resistencia a los sulfatos):		
Sódico	C-88	2 a 3 %
Magnésico	C-88	2 a 3.5%
Absorsión	C-97	1.2 a 3 %
Gravedad específica	C-127	Igual o mayor a 2.5 Ton/m^3
Resistencia a la compresión	C-170	Igual o mayor a 400 Kg/cm^2
Abrasión y desgaste	C-135	25 a 35%

La investigación de los bancos de roca comprende estudios topográficos, geofísicos, geológicos y pruebas de calidad. Resulta de particular importancia efectuar un buen estudio de la geología estructural de cada cantera, que permita pronosticar el tamaño máximo de la roca susceptible de obtenerse, así como para investigar la microfracturación que pudiera existir en el material. Adicionalmente, este estudio servirá para proyectar, en forma racional, la apertura de los bancos de explotación.

Es muy recomendable efectuar voladuras de prueba en las canteras durante la fase de investigación de las mismas, variando los patrones de barrenación y la cantidad de explosivos, a fin de conocer los porcentajes probables de los distintos tamaños de roca que pueden obtenerse en forma económica y práctica, lo cual permitirá desde la fase de diseño, ajustar el proyecto a la disponibilidad de este material. En caso de que no sea posible efectuar dichas pruebas de voladuras durante la fase de diseño, habrá necesidad de hacer ajustes al proyecto cuando se conozca la producción real de las canteras, para evitar el desperdicio de este material.

Desde un punto de vista práctico y económico, es muy recomendable que como parte del proceso de explotación de la cantera se incluyan también las operaciones de acopio y clasificación de la roca, de acuerdo con los diferentes tamaños y cantidades requeridas a lo largo de la construcción de los rompeolas.

4. FABRICACION DE CONCRETOS HIDRAULICOS

4.1. Elementos de coraza

Cuando no sea posible obtener de las canteras las rocas de gran tamaño requeridas en la coraza exterior de los rompeolas, se recurre al uso de elementos precolados de concreto hidráulico. El proyectista puede elegir entre una gran variedad de elementos precolados como cubos, tetrápodos, akmon, dolos, etc. etc. (46), (17), (14), (37) y (34)

Resulta de particular importancia la selección de los elementos precolados debido a que su costo, que resulta en general mayor que el de la roce natural, incidirá directamente en el importe de la obra. Esto justifica los esfuerzos realizados por los diversos laboratorios hidráulicos del mundo para desarrollar y ensayar muy diversas formas geométricas que requieran la menor cantidad posible de concreto en cada elemento. Sin embargo, las experiencias obtenidas en los graves deterioros que han sufrido algunos de los rompeolas construídos en los últimos años, muestran palpablemente que ciertos elementos precolados, que basan principalmente su estabilidad en la trabazón de sus elementos sobresalientes, fallan en la práctica debido a la fragilidad intrínseca de los mismos.

Por otra parte, los bloques simples o con ranuras, que implican la necesidad de emplear mayor cantidad de concreto en cada uno de ellos, han resistido mejor el embate de las olas, en virtud de que su estabilidad en la coraza depende fundamentalmente de su propio peso, tal como ocurre en las rocas naturales empleadas para el mismo fin.(29), (34) y (14)

Cualquiera que sea el tipo de elemento precolado que se utilice en la coraza, resulta indispensable que el concreto que se emplee en su fabricación cumpla con los siguientes requisitos básicos: durabilidad, impermeabilidad y resistencia.(3) y (23) Desde el punto de vista práctico, lo anterior se logra con las recomendaciones que se exponen a continuación y las cuales deben servir de base para formular las especificaciones relativas a la fabricación de los concretos hidráulicos.

Los agregados para el concreto deben satisfacer las normas que se especifican para los concretos normales, principalmente por lo que respecta a su resistencia a la compresión, resistencia al ataque del agua de mar, resistencia a la abrasión, complementado lo anterior, con un adecuado tratamiento de lavado, trituración si procede y clasificación.

De preferencia se utilizará en las mezclas de concreto agua dulce que cumpla con las normas de calidad requeridas para los concretos comunes.

Tomando en consideración que los concretos estarán expuestos a los efectos agresivos del agua de mar, deberán usarse de preferencia cementos con bajo contenido de aluminato tricálcico. El cemento tipo V es el más adecuado para este fin, pues especialmente se fabrica para que sea resistente a los sulfatos. Se recomienda también el cemento de escorias de alto horno, de fraguado rápido.(39). En caso de no poder disponer de alguna de estas dos clases de cemento, puede utilizarse en su lugar el cemento porcland puzolánico, fabricado precisamente con un clinker puzolánico. Cuando tampoco se disponga de cualquiera de estos cementos, deberán emplearse cementos porcland tipos I ó II, combinándolos con un cierto porcentaje de ceniza voladora, puzolana de alto horno o puzolana natural activada, con la cual se obtiene una pasta de cementante resistente a los sulfatos, como se ha demostrado en obras marítimas europeas con más de 50 años de vida útil.(33), (47), (4), (15), (5), (6), (7), (45) y (30)

En virtud de que la durabilidad de los concretos, partiendo de que se fabriquen con agregados de buena calidad, dependerá intrínsicamente de la resistencia de la pasta que aglutinará a las partículas de grava y arena, resulta fundamental la utilización de alguno de los cementos antes mencionados, con o sin aditivos puzolánicos.

Debe procurarse que el concreto sea impermeable para evitar que los elementos deletereos del agua de mar, sulfatos y cloruros, penetren en el cuerpo de las piezas precoladas. Esta impermeabilidad se logra dosifican

do y compactando debidamente el concreto (38), (20), (21), (22), (25), (48) y (49)

Por lo que respecta al proporcionamiento del concreto, necesario para obtener durabilidad e impermeabilidad, se recomienda lo siguiente: contenido mínimo de cementante: 270 Kg/m^3, cuando se tengan agregados con tamaño máximo de 80 mm. Relación agua/cemento: entre 0.45 y 0.50. Empleo de un plastificante para mejorar la manejabilidad de los concretos, con la baja relación agua/cemento antes indicada, para facilitar la colocación y compactación del concreto dentro de los moldes, a fin de obtener un concreto denso, con el mínimo de poros. (23), (26), (39) y (48)

Por lo que respecta a la resistencia a la compresión ésta deberá ser como mínimo de 200 Kg/cm^2 cuando se utilicen agregados con tamaño máximo de 80 mm, en la fabricación de bloques y cubos. Para elaborar elementos frágiles, cuya dimensión sea mayor por lo menos al doble de la menor, se recomienda una resistencia del orden de los 400 Kg/cm^2, con agregados de 50 mm. de tamaño máximo (2), (39), (43), (3), (28) y (44)

El empleo de la ceniza voladora, además de impartir al concreto resistencia a los sulfatos, permite bajar el consumo de cemento en la mezcla; lo que a su vez, trae como ventaja importante la reducción en el gradiente térmico que se genera durante el proceso de hidratación del cemento, que se traduce en una disminución de los esfuerzos térmicos durante el proceso de fraguado que inducen la microfracturación de los bloques. Como resultado final se obtiene un concreto más denso, impermeable y de mayor duración. (40), (2) y (49)

Para controlar debidamente la calidad del concreto, la dosificación de los agregados, el cemento y en su caso la puzolana, se harán precisamente por peso. Durante el llenado de los moldes se procurará evitar la segregación del concreto y la formación de juntas frías. Se recomienda también compactar enérgicamente la mezcla, usando vibradores de inmersión y de forma. Los moldes para fabricar los elementos precolados deberán ser robustos y con la suficiente rigidez para que no se deformen durante el proceso de compactación con los vibradores.

Resulta indispensable efectuar un curado adecuado del concreto, proscribiendo para este fin, el uso de agua de mar durante las primeras 24 horas después de terminado el colado. (36).

Finalmente, se recomienda que los bloques permanezcan sin moverse en el patio de colado por lo menos 28 días, antes de ser transportados y colocados en el rompeolas, para dar tiempo a que el concreto alcance la resistencia especificada. Cuando se use puzolana será necesario aumentar el tiempo de fraguado en el patio hasta alcanzar la resistencia de proyecto. (36)

4.2. Roca artificial

Cuando no sea posible obtener de las canteras la roca natural para la capa secundaria o dicha roca resulte a un costo muy elevado, puede emplearse en su lugar la roca artificial fabricada con concreto hidráulico. (18) y (19)

La roca artificial se fabrica como un concreto normal, utilizando como moldes zanjas excavadas en las cercanías de los rompeolas. Una vez fraguado el concreto se procede a fragmentar las losas utilizando explosivos. Variando las dimensiones de la zanja, principalmnete por lo que respecta a su profundidad y combinando diversos patrones de barrenación, así como la cantidad de explosivo se puede controlar a voluntad el tamaño de la roca artificial, según lo demande el proyecto del rompeolas. Por supuesto que el concreto deberá cumplir con las especificaciones ya apuntadas en 4.3, a fin de que el material resultante sea durable y resistente al ataque del agua de mar. Mediante esta solución se obtiene roca artificial que normalmente puede resultar a mayor costo que la roca natural; sin embargo, habrá casos en que resulte aconsejable su uso.

En situaciones extremas, el núcleo también podrá fabricarse con roca artificial, en cuyo caso, para abatir su costo conviene reducir el contenido de cementante a unos 200 Kg/m^3, con una relación agua/cemento hasta de 0.65.

5. SULFACRETO

Bajo este nombre se identifica un material formado por una mezcla de arenas y gravas, como las requeridas para un concreto hidráulico normal, y azufre modificado como material aglutinante. Los agregados se calientan antes de combinarlos con el azufre fundido en la planta mezcladora. Tan pronto como se enfría el azufre fundido se alcanzan resistencias que llegan a ser de 300 a 400 Kg/cm^2 según la dosificación empleada. El peso específico del sulfacreto resulta del orden de 2 300 a 2 400 Kg/m^3.

Esta tecnología aun no se ha aplicado en la fabricación de elementos de coraza, pero ofrece muchas posibilidades debido a que el sulfacreto es resistente al ataque del agua de mar y puede resultar más económico que el concreto hidráulico. Por otra parte, habiendo abundancia de azufre en nuestro país y escasez de cemento, resulta atractiva la sustitución de un material por el otro.

Conviene por lo tanto, propiciar la investigación básica y la experimental de este material para conocer su durabilidad en las obras marítimas, para lo cual se sugiere colar varios bloques de coraza destinados a los rompeolas de algunos de los puertos industriales que se están construyendo en México, para iniciar la observación de su comportamiento en condiciones reales de trabajo. (1), (16), (37) y (27)

6. GEOTEXTILES

En las últimas décadas se ha desarrollado una nueva tecnología para usar filtros de fibras sintéticas en el desplante de obras marítimas. Estos filtros, que genéricamente se conocen con el nombre de geotextiles, se han aplicado con éxito en diversos lugares como en las obras del Plan Delta, de Holanda, así como en los rompeolas de los Puertos de Dunkerque y Zeebrugge. (24, (36) y (42) Su aplicación más importante se está llevando a cabo en el cierre del Brazo Oriente del río Escalda, último eslabón en el Plan Delta de Holanda. En esta obra se colocarán alrededor de 2.5 millones de metros cuadrados en el desplante general de esta monumental obra de ingeniería marítima.

Para colocar el geotextil en el desplante de las estructuras marítimas se ha perfeccionado un sistema a base de las fajinas tipo holandés, de 60 m. de largo y 30 m. de ancho. Este procedimiento resulta práctico para colocar el geotextil en el desplante de los rompeolas, cuando la superficie del geotextil sea del orden de unos 250 000 m^2. En el cierre del Brazo Izquierdo del río Escalda, en que la superficie del geotextil es diez veces mayor, hubo necesidad de desarrollar una nueva técnica para habilitar en fábrica secciones de geotextil de 30 m. de ancho y 200 m. de longitud, con elementos precolados de concreto como lastre. El geotextil de estas dimensiones se enrolla en un tambor de 10 m. de diámetro y un poco más de 30 m. de ancho que lo transporta flotando hasta una barcaza especialmente diseñada, que lo extiende en el sitio requerido por la obra. Se afirma que este sistema resulta económico cuando se tengan que colocar por lo menos 1 millón de metros cuadrados de geotextil.

Cabe advertir que es posible desarrollar otros sistemas para colocar el geotextil, diferente a los dos antes descritos que ya han sido probados con éxito en su aplicación práctica, con la condición de que se asegure un adecuado posicionamiento del mismo y se eviten daños en la tela al verter sobre ella la roca del rompeolas.

7. TRANSPORTE DE MATERIALES

7.1. Transporte de la roca

En cada caso debe estudiarse el medio de transporte más económico para llevar hasta el sitio de construcción de los rompeolas la roca previamente clasificada en los patios de las canteras. Desde un punto de vista económico, conviene dar preferencia al transporte por vía marítima o fluvial, cuando ésto resulte factible, aun cuando para utilizarlo se requiera hacer inversiones en la construcción de un puerto para el embarque de la roca en las cercanías de las canteras, así como de un puerto de servicio en el sitio de la obra. El transporte marítimo y fluvial ofrece la ventaja adicional de permitir descargar el material directamente desde el chalán al rompeolas, evitándose así las maniobras de descarga de la roca en el muelle del puerto de servicio, el transporte de la roca al patio de almacenamiento y la nueva carga que precede a la operación de colocar la roca en la obra. Lo anterior aumenta las ventajas de tipo económico en este medio de transporte, en comparación con el que se realice por carretera o ferrocarril. Por lo que respecta a estos últimos, conviene planear adecuadamente la selección del tipo de unidades de acarreo, así como los equipos para la carga y la descarga, tanto en el patio de la cantera como en las cercanías de la obra.

7.2. Transporte de elementos precolados de concreto

Para reducir los cargos por concepto de acarreo, convendría fabricar los elementos de concreto precolado para la coraza en un sitio cercano a la obra, cuando se disponga en el lugar de agregados a bajo costo y no resulte oneroso el transporte del cemento; en caso contrario, resulta viable fabricar los bloques en lugares cercanos a los puertos ya existentes en donde sí se cuente con un adecuado suministro de cemento y agregados.

Una vez fraguado el concreto de los bloques, económicamente podrán transportarse hasta la obra por vía marítima, fluvial o terrestre hasta el puerto de servicio del rompeolas en construcción.

Debe procederse con especial cuidado en la manipulación de los bloques, tanto en los patios de colado como durante las maniobras de carga y descarga a los chalanes, a fin de evitar que sean dañados innecesariamente durante estas maniobras. El transporte de los elementos precolados del tipo cúbico o paralelipédico se facilita en virtud de que se pueden apilar unos encima de otros sobre el chalán, sin tener el problema de que se traben entre sí, como es el caso de los dolos, tetrápodos, etc. Estos últimos elementos tienen también la desventaja de que ocupan más espacio en la embarcación y por tanto encarecen y dificultan su transporte y descarga. (18) y (34)

8. COLOCACION DE MATERIALES

La construcción propiamente dicha del rompeolas implica la operación de colocación de los materiales, que incluye como elemento primordial el control por peso de los mismos.

A continuación se describen a grandes rasgos los diversos sistemas constructivos que se utilizan para colocar los elementos de un rompeolas.

8.1. Vertido marino

Para efectuar esta operación se pueden utilizar chalanes de cubierta plana, empujando la roca con tractores o traxcavos. Se dispone también de equipos más especializados como son los gánguiles de vertido por fondo y las embarcaciones para el vertido lateral controlado, bien sea con un sistema de vibradores o con eyectores transversales.

La construcción del núcleo puede hacerse económicamente utilizando los chalanes normales y los gánguiles de vertido de fondo. La colocación de la roca en los atraques al pie de la coraza conviene hacerla de preferencia con embarcaciones de vertido lateral controlado, para garantizar su depósito con un mayor grado de precisión.

En todos los casos se requiere disponer en las embarcaciones de un sistema de posicionamiento confiable y suficientemente preciso que permita verter la roca de acuerdo con las líneas y niveles que marcan los planos de proyecto, respetando las tolerancias que se indiquen en las especificaciones correspondientes. (37) y (43)

El procedimiento de colocación de roca mediante el vertido marino es el sistema más económico y tiene como única limitación la necesidad de disponer de un tirante mínimo de agua, arriba de la roca previamente colocada, suficiente para que las embarcaciones naveguen y descarguen sin peligro de vararse sobre el rompeolas en construcción.

8.2. Colocación a volteo

Este sistema se utiliza preferentemente para construír el núcleo. Debe tomarse en cuenta que el talud natural de la roca así colocada resulta de alrededor de 1.3 : 1 a 1.4 : 1 como máximo. Durante el avance en la construcción del núcleo se procurará que los taludes queden protegidos con la roca de mayor tamaño que se haya especificado para este elemento del rompeolas, a fin de que disponga de cierta protección contra el oleaje, antes deque se arrope con la capa secundaria de mayor tamaño. El material de menor tamaño especificado para el núcleo deberá colocarse de preferencia en el centro del mismo, para limitar en lo posible la pérdida del material fino a través de las capas secundarias. (18), (19) y (43)

En caso de que en el proyecto de núcleo se exija que su talud exterior sea más tendido que el natural de la roca colocada a volteo, resulta imprescindible completar la sección utilizando una charola para roca operada con una grúa de capacidad adecuada. (43)

Para evitar la pérdida de material del núcleo durante el proceso de construcción se procurará ir colocando la capa secundaria de protección lo más cerca posible de su extremo, con tal de evitar la interferencia entre la operación de los camiones de volteo y la grúa con charola para la colocación de la capa secundaria.

En caso de amenaza de mal tiempo el tramo de núcleo ya construído deberá protegerse con la roca de la capa secundaria y en casos extremos se cubrirá esta última con rocas o bloques de la coraza para prevenir su degradación. (18) y (43)

8.3. Colocación con charola

Como antes se indicó, resulta indispensable utilizar este procedimiento para colocar la roca en la parte exterior del núcleo y en la capa secundaria, cuando el proyecto requiera que estos materiales queden con taludes más tendidos que el natural de la roca depositada a volteo. (43)

Es muy importante hacer hincapié en la necesidad de que el límite exterior del núcleo quede rugoso, aun cuando ésto implique salirse un poco de la frontera teórica que marcan los planos, a fin de que la capa secundaria quede acuñada y ligada al núcleo. Por lo tanto, el Residente de la obra debe prohibir que se afine la superficie de contacto a que se hace referencia, pues con tal afine se propiciará la formación de un

plano de falla que resulta muy perjudicial para la estabilidad de la capa secundaria. La anterior observación es aplicable también a la superficie exterior de la capa secundaria, la cual debe quedar muy rugosa, con algunas piedras que sobresalgan o queden abajo de las líneas teóricas que marcan los planos, para que en esta forma la coraza, bien sea de roca o de elementos precolados, se ligue y acuñe en la capa secundaria, para reducir la posibilidad de un deslizamiento del manto exterior de protección del rompeolas. (10), (11), (12) y (14)

8.4. Colocación con grúa

Cuando no resulte práctico manejar con charola la roca de la capa secundaria, resulta indispensable colocarla pieza por pieza, mediante una grúa. Este mismo procedimiento se hace extensivo a la colocación de roca de coraza y los elementos precolados de concreto para la misma. (43)

Durante el proceso de colocar los bloques precolados en la coraza, hay que respetar la especificación que marque el proyecto sobre el número de elementos que deben colocarse como mínimo, en dos capas, sobre una superficie dada del talud del rompeolas, a fin de obtener una predeterminada porosidad en dicha coraza y se cubra totalmente la capa secundaria.

Durante la fase de colocación de los elementos precolados debe utilizarse un dispositivo de izaje que no los dañe y al mismo tiempo facilite la operación de descargarlos en el talud del rompeolas.

En el caso particular de los bloques ranurados, bien sea del tipo denominado VS, empleado en Antifer ó el cubo ranurado tipo A, recomendado para los Puertos de Dos Bocas y El Ostión, se recomienda que los bloques queden con sus ranuras aproximadamente perpendiculares al talud de la capa secundaria. En la práctica se ha encontrado, especialmente en Antifer, que mediante esta precaución los bloques se mantienen mejor en su lugar (18), y no propician el rebase de las olas.

Durante la fase de diseño deben analizarse las dimensiones de las grúas de distintos tipos y capacidades que será posible utilizar para colocar los elementos de coraza, en virtud de que ésto influirá en el ancho mínimo de corona requerido durante la construcción. Al ancho necesario para la grúa deberá aumentarse, cuando así convenga, el correspondiente a un carril para el tránsito de los camiones que colocarán a volteo el material del núcleo ,adelante de la grúa. (39)

Es muy recomendable que la colocación de los bloques de coraza se realice con la grúa apoyada precisamente sobre la corona del rompeolas, ya que cuando la grúa se monta sobre un chalán, resulta muy imprecisa la colocación de los elementos de coraza. Como solución alternativa a este procedimiento, puede emplearse una grúa montada en una barcaza autoelevable (Jack-up), aun cuando cabe advertir que la utilización de este sistema debe analizarse cuidadosamente desde el punto de vista económico.

9. RESIDENCIA DE CONSTRUCCION

Como se dijo en la Introducción, las fases de diseño y construcción de un rompeolas requieren para su debida coordinación, de la sólida organización oportuna de una Residencia de Construcción, la cual será responsable de que la obra se construya precisamente de acuerdo con los planos y especificaciones del proyecto. Tendrá su cargo el ejercicio estricto del contrato; vigilará el control de calidad de todos y cada uno de los elementos constitutivos del rompeolas y supervisará todas las fases de colocación de los diferentes elementos que lo forman.

Para cumplir sus funciones la Residencia de Construcción deberá disponer de suficiente personal especializado, que a la vez que conozca a fondo los criterios básicos que normaron el diseño del proyecto, tenga experiencia en el campo y mente abierta y flexible para estudiar, rechazar o aceptar, en su caso, los procedimientos de construcción que proponga el contratista, de acuerdo con sus recursos disponibles, siempre y cuando el producto final responda a los requisitos de diseño y calidad preestablecidos y se logre ejecutar la obra en el menor tiempo posible y al más bajo costo. Solamente bajo estas condiciones será posible que la obra que se construya responda a la finalidad básica para la que fue diseñada y estará en condiciones de cumplir satisfactoriamente sus funciones a lo largo de la vida útil prevista para la misma. No debe olvidarse que el rompeolas es un elemento de infraestructura básica cuya falla parcial o total podrá entorpecer o aun llegar a paralizar las labores de operación portuaria, que al final de cuentas, constituyen la meta básica a alcanzar en cualquier puerto.

Toda la atención y esfuerzo que se preste a la supervisión y control de la obra, a la larga justificará con creces el costo de la Residencia de Construcción, pues mediante su intervención se controlará su ejercicio presupuestal, al mismo tiempo que se garantizará su correcta construcción conforme al proyecto, lo cual se reflejará a la larga, en un menor costo de mantenimiento y, lo que es más importante, se disminuirá el riesgo de que ocurran fallas catastróficas como las registradas en algunos rompeolas de los nuevos puertos petroleros e industriales recientemente construidos en Europa y Africa.

Tomando en consideración el riesgo intrínseco que implica la construcción de obras como los rompeolas, que estarán expuestas al embate del mar, vale la pena reflexionar en el sabio consejo que nos dejó Leonardo da Vinci: " Cuando tengas que lidiar con el agua, consulta primero la experiencia y después razona".

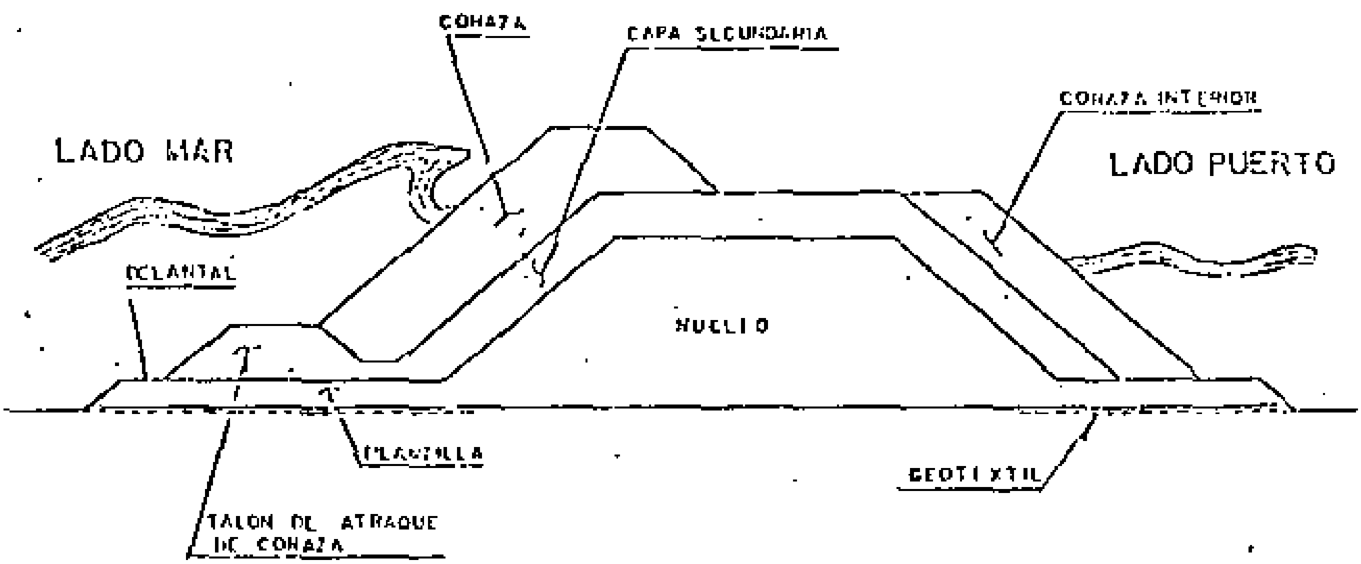

FIG. - 1 ELEMENTOS BASICOS DE UN ROMPEOLAS

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CURSO:

"INGENIERIA MARITIMA"

ORGANIZADO CONJUNTAMENTE ENTRE EL COLEGIO DE INGENIEROS CIVILES DEL GUAYAS Y LA DIVISION DE EDUCACION CONTINUA DE LA FACULTAD DE INGENIERIA-UNAM.

IX JORNADAS DE INGENIERIA CIVIL
TEMA III:

INCIDENCIA DE LA INGENIERIA OCEANICA SOBRE EL ECOSISTEMA MARINO.

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12 AL 17 DE JULIO DE 1982.
GUAYAQUIL, ECUADOR.

Incidencia de la Ingeniería Oceánica sobre el Ecosistema Marino

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Desde los comienzos de la civilización, los márgenes costeros de los continentes y las islas ofrecieron al hombre una amplia variedad de recursos naturales. Inicialmente, estos ambientes representaron áreas productoras de alimentos y vías de acceso al transporte y operaciones comerciales. Al principio, las regiones seleccionadas para tales propósitos se localizaban en márgenes costeros protegidos o bién en el interior de estuarios; sin embargo al incrementarse las transacciones comerciales, se hizo evidente la ampliación de puertos y la construcción de estructuras que asegurasen el uso continuo de dichos puertos; con ello, el hombre comenzó a interaccionar directamente con los procesos naturales del Ecosistema Marino. Con la ampliación de canales naturales se dió lugar a prácticas rutinarias de dragado las cuales dieron paso posteriormente a operaciones de construcción de muelles, rompeolas y canales artificiales. De esta forma, el hombre inició la modificación del ambiente acuático, creando así procesos ecológicos muchas veces, unidireccionales e irreversibles.

En la actualidad los grandes núcleos poblaciones del mundo se hallan concentrados precisamente en los márgenes costeros de los continentes y las islas, lo cual evidencia la urgente necesidad de establecer un uso y un manejo racional de estos dos importantes ambientes ecológicos.

Las zonas costeras constituyen para muchos países, la principal fuente de recursos naturales y la vía de acceso al comercio internacional. La continua modificación de estas áreas y el creciente acumulo de productos de desechos de la industria y centros urbanos son en el presente, motivo de gran preocupación para científicos, planificadores y legisladores. Bajo estas circunstancias, se hace imperativo el estudio y la comprensión de los procesos naturales que tienen lugar en el Ecosistema Acuático, teniendo en mente, que un Ecosistema no constituye una simple abstracción, sino que significa un complejo sistema de organización en el cual interactúan componentes bióticos y abióticos; ambos componentes mantienen entre si, un delicado equilibrio, gracias al cual, dichos Ecosistemas Acuáticos (marinos-estuarinos) mantienen su estructura y su funcionamiento óptimo.

En la última década se ha dado realce y gran difusión a la conservación de la naturaleza; sin embargo es oportuno clarificar algunos conceptos ecológicos que pueden contribuir a la introducción de puntos de vista ambientales en la enseñanza y en la formación de profesionales cuya decisiones repercutirán directamente sobre la evolución de los Ecosistemas Acuáticos. Para lograr este objetivo, es importante disociarse de todo sensacionalismo amarillista y reconocer en la Ecología, a una disciplina científica cuyo carácter objetivo y multidisciplinario nos permite interpretar el desarrollo y funcionamiento de los Sistemas Vivos como partes de un todo.

Atendiendo a los intereses de este Seminario de Ingeniería Oceánica, a continuación se ofrece una versión resumida de los principales conceptos sobre Ecosistemas Acuáticos, los cuales servirán de base para la exposición oral sobre el tema.

Introducción al estudio de los ecosistemas.

. Concepto de ecosistema.

El ecosistema puede entenderse como un sistema ecológico constituído por elementos vivos y no vivos que presentan un intercambio de materia y energía dentro de un proceso dinámico de interacción, ajuste y regulación, con una estructura y función características, y cuyo resultado es la evolución a nivel de las especies y la sucesión a nivel del sistema entero.

La comunidad, o elementos vivos, que integran el ecosistema se designan con el nombre de biocenósis; se llama biotopo a los elementos no vivos, representados por el ambiente. Al sistema formado por la biocenósis más el biotopo se le denomina ecosistema. Las primeras concepciones de ecosistema incluían esta sencilla definición, pero otros autores ecológicos han tratado de redefinir y conceptualizar al ecosistema de una forma más completa, como podría ser la que aparece en el primer párrafo.

El ecosistema es el nivel de organización biológica más complejo, y constituye la unidad fundamental básica para la ecología.

. Estudio analítico del ecosistema.

El estudio de los ecosistemas es sumamente complejo; partiendo de su concepto, el estudio del ecosistema debe contemplar el exámen de las relaciones que se establecen entre determinados factores ambientales con una o varias especies, cómo determinan la existencia de ciertas poblaciones y las formas de interacción que se establecen entre las especies (competencia, simbiosis, predación).

Podemos empezar por establecer las características ambientales y fijar posiciones de interacción de cada uno de los elementos que componen al ecosistema; de esta forma se obtendría una visión detallada o "microscópica del mismo, lo cuál sería sumamente complejo, costoso y a muy largo plazo.

Sin embargo, podemos hacer afirmaciones de tipo estadístico, sobre las variables del ecosistema, y dar una interpretación general sobre el comportamiento del mismo, utilizando conceptos como biomasa, diversidad, productividad, que en gran parte son el resultado de la dinámica del ecosistema, y que, al darnos una visión general o "macroscópica" del mismo, nos permiten entender su funcionamiento al relacionarlo con su estructura, y obtener una visión de síntesis de los procesos biológicos ecológicos que en él ocurren.

Desde un punto de vista funcional, un ecosistema puede analizarse adecuadamente en términos de los siguientes (s. Odum):

1) circuitos de energía,
2) cadenas de alimentos,
3) tipos de diversidad en tiempo y espacio,
4) ciclos biogeoquímicos,
5) desarrollo y evolución, y
6) control y estabilidad.

. Tipos de ecosistemas.

En general, podemos distinguir tres tipos de ecosistemas:

a) los llamados insulares; ecosistemas relativamente confinados, con un ciclo de materia que se encierra casi en su interior, que sólo reciben energía solar de fuera, y que dependen muy poco de sistemas exteriores. Ej: lagos.

b) ecosistemas que se repiten una y otra vez en distintos lugares y siempre bajo condiciones similares ambientales. Ej: ecosistemas bien definidos como arrecifes coralinos.

c) ecosistemas que alteran gradualmente sus características en el espacio, pasando en forma paulatina por condiciones o características de composición y equilibrio diferente. Ej: la mayoría de los ecosistemas, inmaduros como el océano, lagunas costeras, manglares, etc.

Margaleff distingue otro tipo de clasificar a los ecosistemas, atendiendo a su estado de madurez; así, si la diversidad se considera como una medida de la madurez, diferencias locales de las estructura de las comunidades, pueden interpretarse como diferencias en la madurez del ecosistema, y entonces podemos reconocer 2 tipos de ecosistemas:

a) ecosistemas anisotrópicos, como aquellos con diferencias locales en la madurez, ya que presentan fuertes gradientes de diversidad, y las partes adyacentes están relacionadas en un ciclo trófico, y es posible distinguir direcciones muy significativas en el transporte de energía, del estado más diverso al menos diverso. Ej. comunidad del plancton oceánico.

b) ecosistemas isotrópicos, que presentan en todas sus partes igual grado de madurez, ya que su diversidad se encuentra distribuída homogeneamente.

Estructura del Ecosistema

. Componentes Bióticos

Del punto de vista funcional, el ecosistema tiene dos componentes:

a) Componentes Autótrofo (Productores)- Donde predomina la fijación de la energía, el uso de sustancias inorgánicas simples y la formación de sustancias complejas.

b) Componente Heterótrofo (Consumidores)- Donde predomina la utilización, reorganización y la decomposición de materiales complejas.

Los hterótrofos de acuerdo con su alimentación se dividen en:

a) Fitófagos ó Herbívaros - organismos que se alimentan de plantas.

b) Zoófagos ó Carnivoros - organismos que se alimentan de animales.

c) Detritófagos ó Decomponedores - organismos que se alimentan de materia muerta.

d) Omnívoros - organismos que se alimentan tanto de vegetales como de animales.

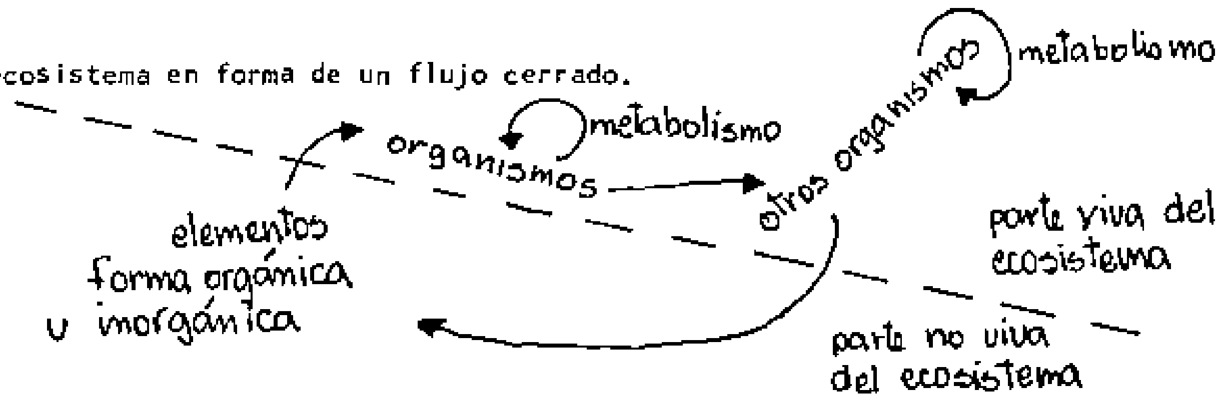
Función del Ecosistema

. Ciclo de materia. (c. biogeoquímicos)

Todo ecosistema presenta dos ciclos: uno de materia y otro de energía, que representan el funcionamiento del ecosistema.

El ciclo de materia es más ó menos cerrado: determinados elementos son asimilados por determinados organismos, toman parte en el metabolismo de éstos, pudiendo pasar a formar parte de otros organismos cuando éstos ingieren a los primeros, y tarde o temprano vuelven a quedar en el medio en forma inorgánica, o de compuestos orgánicos sin formar parte ya de la materia viva, para después volver a ser asimilados por otros organismos.

Así, todo elemento completa un ciclo a través de la parte viva y no viva del ecosistema en forma de un flujo cerrado.



. Flujo de energía.

El ciclo de materia va siempre acompañado de un flujo de energía; el flujo energético no constituye un flujo cerrado, ya que las transformaciones de energía ocurren en un sólo sentido debido a que la energía se degrada y no es recuperable por los organismos. Se puede entonces hablar de un ciclo abierto de energía que impulsa al ciclo cerrado de la materia.

En las sucesivas transferencias de energía, ésta circula en forma de energía química, es decir, asociada a compuestos químicos cuya energía procede, en último término, de la captada y asimilada por los productores primarios. Entonces, los organismos cuyos procesos de síntesis dependen de la energía solar se denominan productores primarios, y aquellos cuyos procesos de síntesis dependen de la energía derivada de otros organismos son los, productores secundarios.

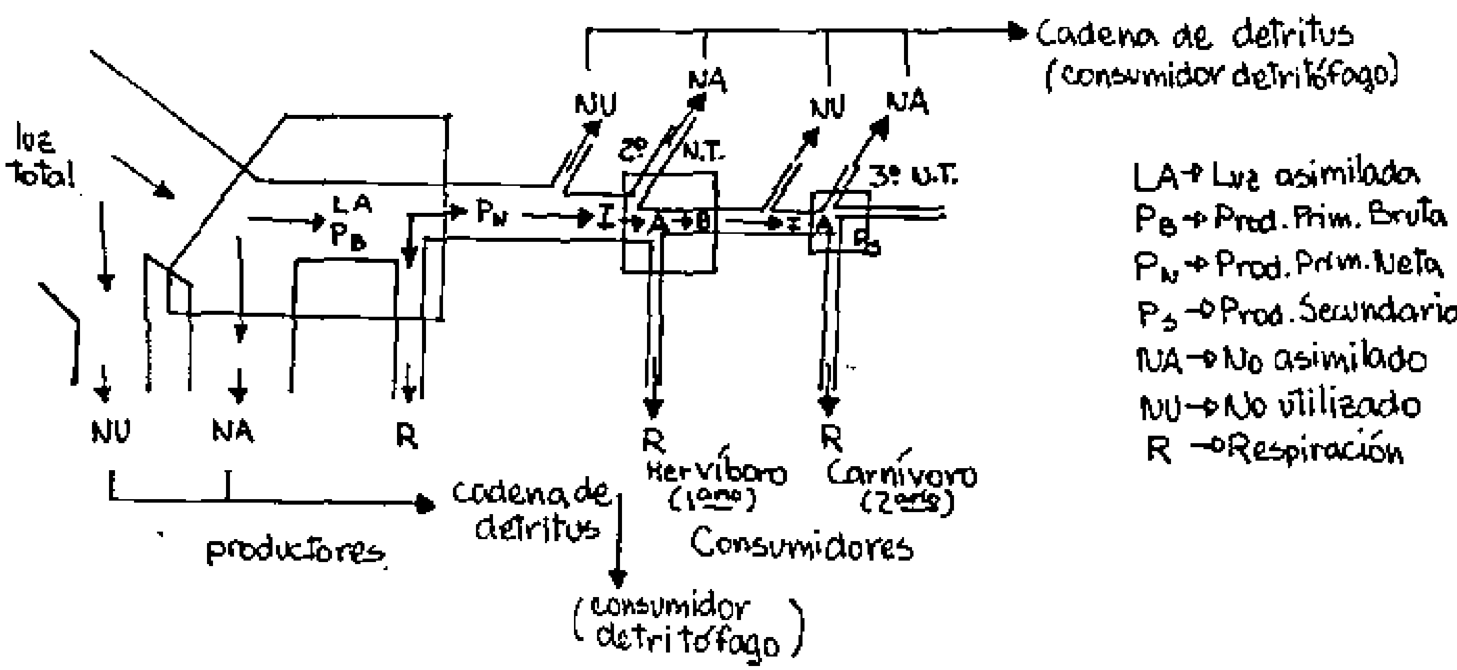
La producción de materia orgánica por los productores primarios constituye una fuente de energía potencial susceptible de ser utilizada por organismos de niveles tróficos superiores, por lo tanto, la entrada de energía al sis tema constituye la producción primaria; la producción secundaria es aquella que comprende a los niveles tróficos superiores, y que representa la utilización de la energía transformada y acumulada en el primer nivel trófico.

La velocidad a la que es almacenada la energía por la actividad fotosintética, es la productividad primaria; y en este sentido, se puede hablar de la productividad secundaria, que representaría la velocidad de la producción secundaria. La productividad primaria se puede considerar de dos formas:

PPB (productividad primaria bruta) - velocidad total de la fotosíntesis, incluída la respiración.

PPN (productividad primaria neta) - velocidad de almacenamiento de materia orgánica (sin incluír respiración).

Partiendo de estos conceptos, podemos utilizar un modelo sencillo para entender el flujo de energía en el ecosistema, y las transformaciones energéticas que ocurren en cada nivel trófico.



Las entradas de energía en cada nivel compensan a las salidas, tal como lo expresa la la. ley de la termodinámica, y cada paso de energía va acompañado de una dispersión de energía en calor no reutilizable por los organismos, tal como lo espresa la 2a. de estas leyes.

Madurez del Ecosistema

. Sucesión y clímax.

La sucesión ecológica se puede considerar cono un proceso de la ocupación de un área por los organismos, proceso de auto-organización en el cuál el ecosistema puede pasar a través de diferentes estados; cualquier cambio que le lleve a un estado más resistente es inmediatamente asimilado. El estado siguiente en una sucesión puede predecirse, pero sólo a nivel macroscópico.

La sucesión no es necesariamente un proceso contínuo; además, pueden existir estados de "regresión", o bién, debido a que el ecosistema no presenta aún un dominio del ambiente, éste último puede impedir que el ecosistema avance en la sucesión.

Margaleff opina que el proceso de sucesión es equivalente a un proceso de acumulación de información. En el tiempo, la información adquirida se expresa en una nueva organización del ecosistema; esta organización toma en cuenta los cambios predecibles del ambiente, y más aún, controla al ambiente, de modo que en el futuro, pequeños cambios en la comunidad serán necesarios para que el ecosistema siga ocupando su área en forma estable.

Puede considerarse que la coloniación de un área es el primer estado de la sucesión, y ésta (la sucesión) puede existir indefinidamente hasta que, por lo menos, el ecosistema no llegue a un estado máximo de madurez y estabilidad. Por esto, existen ejemplos de ecosistemas que se encuentran en un estado cualquiera de sucesión, sin que puedan llegar a la madurez hasta después de un tiempo considerablemente grande (como las comunidades planctónicas de aguas templadas).

Al nivel macroscópico, pueden reconocerse ciertos cambios del ecosistema en el proceso de sucesión, o cambios que se espera que sucedan conforme el ecosistema va avanzando en madurez. Los cambios más importantes serían: incremento de biomasa, producción primaria y diversidad; multiplicidad de nichos, alargamiento de las cadenas alimenticias, especialización, estabilidad del cociente PP/B, tendencia a reducir la tasa de reproducción con una major protección a los jóvenes, dispersión organizada de especies, incremento de la cadena de detritus, y, en términos energéticos, una mayor eficiencia en la utilización de la energía como sistema.

La sucesión es un proceso asintótico, y la entrada de especies al sistema tiene un límite real por lo cuál la sucesión no puede ir más allá. Entonces, la sucesión tiende a un estado que podríamos considerar estacionario (entendido como de estabilidad dinámica), que en ecología se le denomina comunmente como clímax. Sin embargo, por la dificultad que representa el poder definir un estado de clímax en cualquier ecosistema, se prefiere hablar de los ecosistemas como más ó menos maduros.

. Estabilidad

El concepto de estabilidad ha sido muy discutido, pudiendo distinguir dos conceptualizaciones:

a) hay quienes opinan que la estabilidad es la consecuencia de las interacciones que ocurren dentro del sistema, en donde el sistema tiende a un estado estable bajo condiciones constantes.

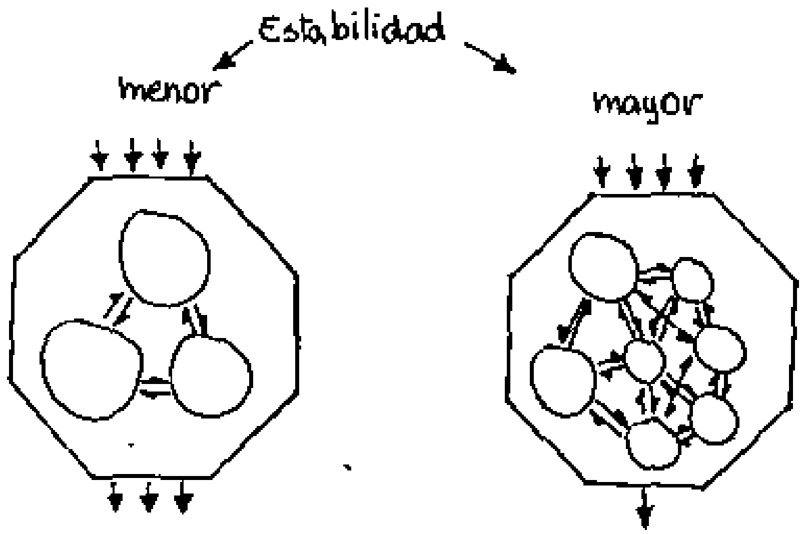
b) sin embargo, la estabilidad debe de tener que ver con las variables exógenas al sistema; entonces el término estabilidad se usa para designar la habilidad del sistema para permanecer razonablemente similar a sí mismo, absorviendo dichos cambios.

El concepto de estabilidad de Mac Arthur, es el más comunmente empleado en ecología, y corresponde a la segunda concepción (b) en la que estabilidad refleja la gran resistencia del ecosistema a los cambios externos.

De acuerdo con Mac Arthur, el ecosistema es estable, si sobrevive a muchos cambios, pero preserva ciertas características esenciales (que hay que definir), lo que consigue cambiando especies, variando las proporciones entre unas y otras, usando vías alternas en las redes tróficas, etc.

El concepto de estabilidad puede entenderse con un esquema en el que se represente al ecosistema como un conjunto de puntos.

Cada uno de ellos correspondería a un elemento del ecosistema. Entonces, un ecosistema constituído por un mayor número de elementos presentaría menos fluctuaciones, y el flujo total de energía por unidad de biomasa sería también menor; si las especies son más numerosas, puede existir una especialización mayor, un mayor aprovechamiento de lo ingerido, un menor despilfarro de energía debido a la disminución de la tasa reproductiva, y menores fluctuaciones de las poblaciones.



Entonces, con una mayor posibilidad de relaciones entre los elementos del ecosistema, se tendrá una estabilidad mayor, con una eficiente utilización de la energía.

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ORGANIZADO CONJUNTAMENTE ENTRE EL COLEGIO DE INGENIEROS CIVILES DEL GUAYAS Y LA DIVISION DE EDUCACION CONTINUA DE LA FACULTAD DE INGENIERIA. U.N.A.M.

IX JORNADAS DE INGENIERIA CIVIL

TEMA IV:

D R A G A D O

ING. J.P.C. VAN DEN KIEBOOM
ROYAS BONKAL WESTMINSTER NV.

12 AL 17 DE JULIO DE 1982
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Esta ponencia ha sido preparada por el suscrito, con la ocasión del Simposio de Oceanología, organizado por PEMEX - Petróleos Mexicanos en marzo de 1982 - Ciudad de México.

Ing. J.P.C. van den Kieboom
Royal Boskalis Westminster N.V.

ECOLOGIA RELACIONADA AL DRAGADO

INTRODUCCION

En los últimos años, la descarga de deshechos humanos e industriales dentro del ambiente marino, ha ido en aumento convirtiendo las áreas dragadas, en depósitos de material contaminado.
Aunque en general el dragado y eliminación del material puede crear muchos problemas en el medio ambiente, en este particular caso puede ser solo atribuído al dragado en el sentido de que el existente problema puede ser agravado disturbando el estatu quo de un lugar en particular. En la mayoría de los casos, los efectos de las áreas dragadas, como un fenómeno momentáneo, podría ser menos objetable que las causas naturales, tales como: tormentas, inundaciones, las que podrían conventirse en beneficiosas a largo plazo.
Si las intencionadas operaciones de dragado presentaran aspectos dañinos u objetables, se deberá de elaborar una evaluación detallada del proyecto, concediendo apropiada consideración a los beneficios del proyecto así como los efectos en el medio ambiente.
No se considerarán aqui los efectos evidentes por sí mismos - principalmente de naturaleza psicológica - tales como: ruido, obstáculos visuales y olfato así como turbiedad causada por las operaciones de dragado. Esto también es aplicable a las consideraciones económicas.

GENERAL

Las consecuencias de la eliminación del material dragado en el medio ambiente, consiste en la presencia de contaminantes y específicamente el posible transporte de aquellos a otro medio ambiente (la movilidad). El medio ambiente se deberá de identificar aquí como: agua freática, cosechas, organismos animales, etc. Como resultado del método de dragado aplicado, método de transporte del sedimento y de procesos químicos/físicos, contaminantes tales como: materias anorgánicas, metales pesados, petróleo y otras materias alienantes al medio ambiente; podrán ser liberadas y movidas. Los efectos en el medio ambiente por la descarga en el mar, lagos y tierra serán considerados en esta exposición.

Asímismo se puede también investigar, qué pasaría si los trabajos de dragado tuvieran que ser detenidos (alternativa cero) y qué posibilidades puede brindar el desarrollo de depósitos alternativos y métodos de tratamiento.

Se hará uso de información de distintas fuentes, tal como un reporte preliminar completado en 1978 con relación al efecto del medio ambiente de depósitos de dragado en el puerto más grande del mundo: ROTTERDAM

Cantidades

Corrientes de agua.

Para poder entender la provisión, cantidad y composición del material dragado, es necesario saber acerca del movimiento del agua.

El movimiento de aguas en ríos, mares y océanos dependen especialmente del origen y dispersión de los contaminantes.

Provisión de sedimento

A este respecto, se deberá de entender que una lengua de agua salada penetra interiormente cerca del fondo de los ríos, teniendo una gran influencia en el asentamiento del sedimento transportado por éstos. Debibo a esta lengua de agua salada, el sedimento del mar será depositado en los puertos y vías navegables que se encuentran más cerca al mar.

Como un ejemplo tenemos: En Rotterdam los rios proveen 30 - 50 mg de sedimento por litro durante una descarga normal de agua (2200 m3/sec.) Esta corriente de sedimento no se encuentra completamente sincronizada al movimiento del agua. Sin embargo, en la ausencia de un método más preciso, los cálculos para la provisión y distribución de las diversas corrientes, están basados en los movimientos conocidos de agua. En el caso de Rotterdam los cáculos muestran que, del sedimento proveído a través de las descarga de los ríos en el delta del Rín, aproximádamente el 40% se asienta en los puertos y accesos portuarios, De esta manera, los ríos proveen por año de 3 a 4 millones de toneladas de sedimento.

El mar provee aproximádamente 7 millones de toneladas de materia en suspenso a los puertos y vías navegables que se encuentran cerca del mar.

La cantidad total de sedimento dragado es de un volumen anual de 21 millones de m3 de material mojado (11 millones de toneladas, peso de material seco).

Para poner estos datos en perspectiva, podría ser de interés observar que, Pequegnat y Smith (referencia), cuando manifiestan que la mayoría de sedimento de grano fino alcanza el océano a través de los ríos. Ellos estiman que alrededor del mundo, una cantidad anual de 8×10^9 de toneladas métricas es depositada en el talud continental y su ascenso contiguo. Por lo menos el 40% de éste es arrastrado por, más o menos, una docena de rios mayores. Solo el rio Mississippi transporta anualmente 200 millones de metros cúbico de sedimento a y más allá de su delta. También estiman que solo en la dársena Argentina se dan entre 30 y 300 millones de toneladas métricas de material suspendido.

Composición granular

La composición granular de los diferentes tipos de material dragado (variando de lugar de dragado) no es constante. La composición es importante cuando se clasifica y compara la naturaleza y características de las diversas muestras.
A este respecto, se debe de comprender que la contaminación del material dragado será determinado principalmente, por las partículas más finas.

Contaminación

Del sedimento del Rín, data de los siglos 15 y 16, se ha establecido que los metales pesados se dan naturalmente. Sin embargo, hoy en dia el contenido de metales pesados así como otros contaminantes, son mucho más significativos.
Los siguientes contaminantes será revisados en esta exposición:

- metales pesados
- cuerpos extraños al medio ambiente, tales como pesticidas
- petróleo
- cloruro
- cuerpos orgánicos
- alimento, tales como: nitrógeno y fósforo
- micro-órganos tales como: virus y bacterias

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Cuando se tiene en cuenta metales pesados, se prestará atención a aquellos elementos que por su acción tóxica, pueden ser probados o esperados.
Cuerpos extraños al medio ambiente son aquellos que nunca o casi nunca se dieron anteriormente en sedimentos. Acciones humanas han causado su distribución. Estos cuerpos, o no se desintegran, o lo hacen muy lentamente. La acumulación de estos cuerpos y sus derivados revelan extraordinariamente aspectos del problema.
El petróleo consiste de muchos componentes con variables característi-cas peligrosas. Estas son muy amenudo dificiles de desintegrar. Alimentos tales como nitrógeno y fósforo son también considerados como contaminantes. Ellos pueden causar un crecimiento excesivo de algas.
Las grandes cantidades de contaminantes pueden ser removidos a través del dragado de mantenimiento, dependiendo del grado de contaminación del material.
Se verificó de cálculos globales designados a apreciar los distintos contaminantes, que en Rotterdam se podía remover:

- 10% a casi 100% de materiales pesados, de preferencia aquellos unidos al sedimento con el material dragado.
- 50 - 60% del aceite, incluyendo derramamiento en el área portuaria.
- menos del 10% de los hidrocarburos.
- casi el 100% de los cuerpos orgánicos y fósforo.

Comportamiento

Comportamiento de la contaminación.
La contaminación puede ser compuesta de varias formas: por ejemplo, disuelta en el agua, fijadas intercaladamente a partículas sólidas o como compuesto químico estable. El grado de composición es determinado por condiciones químicas, físicas y bactereológicas.
Lo mas conocido es acerca de metales pesados.

Descarga en el agua

Como consecuencia de la descarga del material dragado en el agua, se da mezcla y dilución. En el mar, las partículas pequeñas, relativamente las más contaminadas, serán desplegadas sobre una gran área. La mayoría de los metales pesados disueltos, serán transformados en finos cuerpos sólidos suspendidos (óxido metal) causado por el oxígeno y/o acidés alzada. Las partículas de sedimento pueden actuar como "filtro de polvo". Finalmente, la contaminación será apilada en el sedimento del fondo.

Descarga en tierra

Dependiendo de la composición del material dragado a ser almacenado en tierra, los contaminantes libres pueden unirse con otras partículas de arcilla o humus o ser tranformados por procesos biológicos de demolición. La extensión en la que los contaminantes son liberados, depende de: lluvia y evaporación, porosidad del material dragado y características del subsuelo.

Posibilidades de liberación de los contaminantes

Del despojo dragado, se distinguen los siguientes puntos principales:

a) en el mar	:	separación, baja acidés, agua salada, dispersión amplia.
b) en lagos	:	separación limitada, agua fresca o salobre, dispersión y dilución limitada.
c) en tierra	:	no hay separación, transporte limitado, por: absorción, vegetación capa superior rica en oxígeno.

EFECTOS EN EL MEDIO AMBIENTE POR DESCARGA EN EL MAR - ZOCALO CONTINENTAL O ALTAMAR

Debido al considerable volumen de material dragado de ciertos lugares portuarios, la descarga en el mar de un material dragado relativamente limpio, es inevitable.

Ha sido establecido que en Rotterdam, especialmente en condiciones severas de tiempo, una parte del material dragado descargado en el mar, regresa al acceso portuario.

El presente

Los mares y océanos son usados para diferentes finalidades. En este contexto la pesca es importante.

Se conoce muy poco acerca de la calidad de los fondos acuáticos y sedimento de los mares y océanos.

Plancton es la base para la vida marina. Plancton es la fuente de alimento para los pescados quienes a su vez son la fuente alimenticia para aves y mamíferos. Plancton que muere regresará a la cadena alimenticia, usualmente a través de los organismos del fondo acuático.

Efectos

Cuando se descarga el material dragado, la vida del fondo acuático del lugar es destruído y la formación del plancton es obstaculizada.

La provisión de sedimento por ejemplo por ríos, es parcialmente un proceso natural, más no así el bombeo del material dragado contaminado en un lugar cualquiera. La influencia de contaminación de plancton es difícil de establecer debido a la distribución de estos organismos. El plancton se restituye así mismo muy rápidamente en un nutrido ambiente marino. Los efectos de especies de fondos más amplios, tales como: choros, ostras, langostas, etc. pueden ser calculados con precisión y pueden ser relacionados, hasta cierto punto, al origen de contaminación. De estas especies, especialmente, los choros y ostras pueden ser severamente dañados por la resencia de contaminantes filtrados y absorbidos por el agua.

Las consecuencias en los pescados también son conocidas pero debido a su movilización es dificil de establecer una relación en el origen de contaminación.

Los pescados son vulnerables, especialmente cuando estos son aún pequeños y por ello necesitan de una maternidad limpia.
En las capas gruesas de los pescados, aves y especialmente mamíferos, los contaminantes tales como metales pesados y otros cuerpos extraños pueden acumularse.
Cuando estas capas de grasa son usadas como alimento de reserva, puede darse una muerte masiva.
Los mamíferos transmiten a la prole, por medio de la leche, la contaminación acumulada en el cuerpo de la madre.
Los hombres también pueden experimentar los efectos negativos a través del consumo de estos pescados y mariscos. En Japón han ocurrido casos muy serios.
Actualmente no es posible relacionar la descarga en el mar de material dragado contaminado con un posible incremento de efectos peligrosos.
Sin embargo, se sabe que si no es controlado, estas descargas aumentarán la contaminación en el ambiente marino considerablemente, haciendo que tales consecuencias se den más marcadamente.

Altamar

Esto es definido por Pequegnat y Smith (referencia) como:
Esa parte del océano al y más allá del margen exterior de la plataforma submarina - descanso de la plataforma. Este descanso puede darse a profundidades de hasta 200 m. Ellos sugieren que estos descansos a esa profundidad o más, son adecuados como lugares de depósitos del material dragado.
Teniendo en consideración que la tierra está cubierta de agua en un 71% del cual el 80% tiene una profundidad de 2000 metros, quedará claro que el área de la plataforma submarina es relativamente pequeña por cuanto, menos adecuada que altamar como lugar de depósito.
Más aún, los océanos producen menos del 1,5 al 2% del alimento mundial marino.
Tait y de Santo '72 (referencia) predicen que altamar nunca contribuirá con más del 1% del total mundial de pesca marina.
El complejo de la plataforma interior del estuario es el mejor productor de alimento marino.

EFECTOS EN EL MEDIO AMBIENTE POR DESCARGA EN TIERRA

Historia

En varios países, y principalmente en los Paises Bajos, muchas áreas han sido rellenadas con material dragado. Esto también ocurrió en el área de Rotterdam material el cual provenía de la creación de nuevos puertos por tanto no contaminado y otra parte del dragado de mantenimiento de puertos y vías navegables existentes en el área.
No obstante, se requiere de métodos de maduración muy perfeccionados y de grandes superficies para el depósito de material dragado (aproximádamente 150 hectáreas por año por millón de metros cúbicos de material mojado; en áreas sub-tropicales y tropicales, ésta podría ser menor dependiendo de: lluvia, subsuelo y vegetación).

Situación actual

Actualmente, la locación y paisaje del área de descarga será determinada por los factores tales como: espacio, erosión, sedimentación, condiciones de tiempo, etc. También será determinado por la vida racional, animal y vegetal, por el cultivo de tierra y el género humano.

Efectos

Cuando se descarga el material contaminado en tierra, la calidad de agua en la superficie y del fondo, organismos y flora del suelo marino, pueden ser afectados y finalmente a través de la cadena alimenticia, los animales y el género humano. Metales pesados y cuerpos extraños al ambiente pueden poseer características tóxicas agudas y acumularse en órganos generando cáncer, etc. Con relación a los metales pesados debe de notarse que esta acción puede ser aumentada a la presencia de otro metal (cadmio, zinc). Los efectos de esta contaminación se hacen visibles a través del reducido crecimiento y deformación de plantas, por síntomas de intoxicación crónica en los animales y consecuentemente seres humanos.
Pueden darse consecuencias indirectas comparables cuando el agua del fondo está contaminada debido al considerable asentamiento de capas de material dragado.

Agri y horticultura

No existen reglamentos que dicten la composición del suelo destinado para la agri y horticultura.

Vivienda, Industria

La contaminación del agua de superficie y fondo jugará un rol específico, cuando un área para depósito de material sea usada para vivienda e industria. El saneamiento del área será de importancia.

Recreación

Cuando la zona es destinada para uso recreativo, la naturaleza de la ubicación determinará los efectos a ser esperados. No existen líneas generales de calidad de suelo para zonas de uso recreativo.

General

La población costera seguirá creciendo alrededor del mundo. Consecuentemente la necesidad de espacio para vivienda y la utilización de regiones costeras competirán con áreas para producción de alimentos. En general, la zona costera es en donde el despojo de derivados marinos tendría que ser almacenada en tierra.

EFFECTOS EN EL MEDIO AMBIENTE POR DESCARGA EN LAGOS

Historia

Ya que es difícil en Holanda de encontrar áreas en tierra apropiadas para la disposición de material, se está considerando la posibilidad de disposición en lagos (especialmente aquellos creados por la remoción de arena, agregados y arcilla).

Situación actual

Los lagos arriba referidos tienen generalmente, un gran volumen y podrían contribuir considerablemente con el problema de descarga.
La composición y funcionamiento de vida sobre los lagos, es comparable a aquella en el mar, la sucesión de plancton, animales del fondo, pescados, aves y humanidad. Aquí el plancton puede tener un desarrollo explosivo a través de los alimentos, o pueden morir y hundirse, originando falta de oxígeno en el agua. La provisión de fósforo activará especialmente este proceso. El material dragado puede contener considerables cantidades de fósforo.
El agua en un lago está generalmente en contacto con el agua en la superficie, por lo que se da la distribución. De acuerdo con la estructura del subsuelo, el agua del lago puede encontrarse también en contacto con capas porosas. En estos lagos, como en este caso, las capas sin oxígeno pueden darse a grandes profundidades durante condiciones de tiempo de verano. Así debido también a la ausencia de luz a grandes profundidades, la vida biológica en estos lagos, especialmente la presencia de los animales del fondo, es limitada a pequeñas zonas al margen de los lagos.
Cuando se deposita el material dragado en lagos, se pueden esperar efectos del agua de calidad química y bactereológica, vida hidrobiológica, el agua del fondo en la vecindad del lago y la claridad, salinidad y valor nutritivo del lago mismo.

Efectos temporarios

Los efectos se pueden dar directamente por la disolución de contaminantes en el agua durante el acto de disposición. El transporte de agua

puede causar también esta contaminación directa.
Durante la disposición es principalmente, el agua pura con sus contaminantes absorbidos, la que influye en la calidad del agua del lago.
Los materiales pesados posiblemente disueltos podrán eventualmente oxidar y hundirse al fondo (en un medio ambiente rico en oxígeno)
Este no es el caso con nitrógeno y cloruro y en un menor alcance, fósforo.
Un gran consumo de oxígeno del agua será utilizado por el óxido químico de material arcilloso.
La contaminación bactereológica es de duración limitada, ya que las bacterias se atrofian rápidamente. Dependiendo del tamaño del lago, esto puede causar más o menos un serio disturbio o aún la destrucción de la vida biológica, en donde el material dragado cubre el fondo.

Efectos a largo plazo

Los efectos a largo plazo en la calidad del lago y sus alrededores son dependientes, entre otros, del contenido de oxígeno en el agua inmediata sobre el fondo. La contaminación (metales pesados) expulsada del fondo, puede quedar suspendida allí donde el agua no contiene oxígeno. En el caso de agua rica en oxígeno, el óxido reducirá el riesgo de la presencia de metales pesados. El agua contenida dentro del área de material depositado será expulsada por el asentamiento de dicho material. Esta agua puede afluir al lago y causar una contaminación gradual, lo cual puede reducirse solamente por el posible reaprovisionamiento del lago.
Los cuerpos extraños se disuelven generalmente pobremente en el agua y prefieren adherirse a partículas de material contaminado por petróleo. No se puede esperar una movilización a aguas abiertas o de fondo, debido a esta fuerte estabilidad química.
No es posible una sistemática revisión, debido a la considerable variedad de cuerpos extraños y sus residuos.
Lo arriba expuesto demuestra que la expulsión de cuerpos extraños y metales pesados es logrado casi exclusivamente a través de la absorción activa por la cadena de alimentos de la fauna y flora del fondo acuático.

Alimentos

Considerando un gran contenido de fósforo en el material dragado, se puede esperar que una cierta cantidad de agua estancada en la superficie del material pueda ser atrofiada y quedar así por un largo período.
La porosidad del material en su fase de inmadurez, es muy limitada (aproximádamente 0,5 - 1,0 mm/dia).

Relleno de lagos

En el caso de que un lago es rellenado con material dragado, se presenta una combinación de posibles efectos en el cual la capa superior (por ejemplo sobre el agua subterránea) puede ser considerado como material depositado en tierra.

EVALUACION

Objetivo

Examinar si es factible determinar que clase de material puede ser descargado y en donde, con el fin de minimizar los efectos negativos en el medio ambiente.

Prevención a la base

No importa como enfoquemos las técnicas de disposición, la conclusión será invariablemente que la reducción y terminación de la descarga de contaminantes en la fuente será la más efectiva.

Clasificación

El material dragado puede ser clasificado de diferentes maneras: por su origen, por su contaminación con metales pesados y por contaminación con cuerpos extraños.
La clasificación por contaminación con cloruro y materias orgánicas, etc. puede ser de importancia solo en casos específicos.

Perspectiva futura

Es importante saber como se desarrollará la calidad de material dragado. Las autoridades gubernamentales serán primeramente vistas para establecer reglas adecuadas para la descarga de contaminantes así como la observación de las regulaciones por contaminadores en potencia.

Efectos en el medio ambiente

Cuando se examinan los efectos de estas posibilidades para descarga de material dragado en el medio ambiente, debemos de considerar 3 criterios importantes:

- Cuál es la naturaleza de la distribución (difusa o concentrada)
- Qué medidas existen para reducir la emisión
- Cómo se puede controlar la disposición del material dragado

Las 3 posibilidades de disposición consideradas, pueden resumirse como sigue:

Efectos en el mar

- Disposición en el mar: dilución considerable y distribución difusa, medidas de control no disponibles, pocos efectos apreciables, peligro de acumulación en la cadena alimenticia.
- Disposición en altamar: igual que lo arriba expuesto, pero existe muy poco peligro de acumulación en la cadena alimenticia..

Efectos en lagos

Disposición en un lago: distribución menos difusa, excepto la dispersión a través del transporte de agua, por el movimiento del agua en el lago y a través del agua subterránea.
La limitación de la emisión es parcialmente posible así como el control de emisión del material ya depositado. Sin embargo, existe el peligro de acumulación en la cadena alimenticia.

Efectos en tierra

Descarga en tierra: una distribución menos difusa que en el mar, es posible la emisión al agua subterránea y superficie. Existen posibilidades para limitar y controlar la emisión en material ya depositado. Sin embargo, debido a la imperfección humana existe peligro de acumulación en la cadena alimenticia.

NORMAS Y REGULACIONES

La ausencia de normas y regulaciones pertinentes se pusieron de manifiesto cuando se intentaba investigar la descarga de material dragado en el área de Rotterdam.
Aunque existen acuerdos internacionales, no hay normas específicas que se atengan al vaciado en el mar.
Entre éstas, solo la "Convención de Londres", realizada en Noviembre 1972, que trató sobre la prevención de contaminación marina ocasionada por la descarga de desperdicios y otros cuerpos, tiene algún significado para la región latinoamericana, lo que fué ratificado por Argentina, Cuba, Chile, México, República Dominicana, Guatemala, Haití, Panamá, Surinam y los Estados Unidos.
La convención trata sobre todos los océanos y mares; prohibe la descarga de algunos cuerpos y requiere de permisos especiales para la descarga de otros. Sin embargo, no se han dado pautas cuantitativas.
Reciéntemente, en Octubre 5/9 de 1981, tuvo lugar la sexta (6ta) reunión consultiva de las partes contratantes. Entre otros asuntos, se discutió sobre los métodos de cuidados especiales que se deberían de aplicar en la descarga de material dragado contaminado. Ellos incluyeron el cubrimiento de los lugares de descarga con material limpio, relleno de yacimientos con el subsecuente cubrimiento, descarga en barrancas marinas en pilones hipersalinos, en regions abiológicas del océano.
Se ha acordado remitir esta técnicas de cuidados especiales, a centros de estudio de investigación.
Otras convenciones similares como la de Oslo (72) y Paris (74), firmadas por los países de europa occidental, prohiben la descarga de algunos contaminantes específicos y la reducción de cantidad de desperdicios contaminados con otros contaminantes específicos.
El contenido de metales pesados en el material dragado puede ser solo comparado a regulaciones para fertilizantes. Generalmente, no existen normas para regular la calidad del agua subterránea.

ALTERNATIVAS

Alternativas relacionadas a:

Variante cero

La terminación de la descarga del material dragado y consecuentemente del dragado mismo (variante cero). En este caso se ha trasado una relación entre el no más funcionamiento del puerto y la transferencia de la actividad industrial a otras localidades portuarias.
El material contaminado últimamente no se asentará más en el área portuaria pero en el mar, al menos en aquellas áreas en donde el sedimento es transportado por los rios en donde el puerto abandonado estuvo situado.

Reducción de la actividad del dragado

Reducción en volumen de material dragado. Se puede llevar la atención a experimentos y planes intencionados para la modificación de la descarga del régimen del rio y reducir consiguientemente el volumen de material a dragarse.

Remosión de contaminantes

La remosión de contaminantes del material dragado. Se puede considerar la remosión de cuerpos solubres en agua. El desalinamiento parece ser un método factible. La salinidad contenida en el material no solo depende del lugar dragado, sino también de la calidad del agua de transporte. La descarga de sal adicional rio arriba, podría ocasionar conflictos con los acuerdos internacionales, como es el caso de Europa.

Aplicaciones alternativas

Se ha probado que es factible usar el material dragado del puerto de Rotterdam para la fabricación de ladrillo. Sin embargo, la dureza del ladrillo no es muy alta debido al gran contenido de cuerpos orgánicos. Más aún, la demanda de arcilla por la industria del ladrillo, es pequeña, con relación a la provisión de material dragado contaminado. Relleno de áreas también puede ser considerado como una solución alternativa. Los problemas más importantes aquí son el de la limitada capacidad de drenaje

(saneamiento) del material y la duración de tiempo requerido para la realización.
El material dragado puede ser también incinerado pero requiere de energía extra, apareciendo los contaminantes luego de la incineración, en las cenizas o residuos.
También es posible la utilización del material como sustituto de la arcilla para el levantamiento o agrandamiento de diques.

Alternativas operativas

Los pozos creados por la remosión de tierra (agregados, etc.) puede ser visto como una alternativa operativa la cual en principio puede ser considerada como depósito de material dragado. Los efectos de éste en el medio ambiente puede ser interpretado como los efectos descritos en descarga en tierra o en lagos.

CONOCIMIENTO E INFORMACION ADICIONAL REQUERIDO

(Para una propia evaluación)

Además de los datos relacionados directamente al "Area del Problema", se necesita tener conocimiento e información sobre diversos asuntos relacionados, los cuales pueden ser subdivididos en 3 grupos principales:

Información básica

- Calidad del material dragado fuera del área portuaria.
- Factores que influyen en la capacidad de transporte de la contaminación.
- Información de la calidad del agua subterránea y del mar.
- La parte del mar contaminada por la descarga de material dragado en relación a la contaminación proporcionada por ríos y otras fuentes (otros orígenes).
- Características biológicas del material dragado.

Normas

x Normas de calidad para el agua de mar.
x Normas de calidad para el suelo de fondo.
x Normas de calidad para el agua subterránea.

Efectos

- Penetración en la acumulación de la contaminación en la cadena alimenticia en un ambiente de agua fresca, no existe conocimiento de la relación entre la dosis y su efecto.
- Acumulación de contaminantes en cosechas no destinadas para el consumo.
- La influencia en la calidad del agua freática, por la descarga en tierra, debe de ser identificada más precisamente.

EQUIPO DE DRAGADO PARA REMOSION DE SEDIMENTO CONTAMINADO

- Podemos distinguir:

1.- Contaminación en el fondo.
2.- Contaminación en suspenso en la columna de agua.
3.- Contaminación en la superficie.

1.- Para remover sedimento contaminado del fondo con turbidez y derramamiento mínimo, las dragas más adecuadas serían:

transporte hidráulico	draga de succión tipo "Dustpan"
transporte neumático	"Air lift" - Pneuma, Oozer
transporte mecánico	Cucharón de quijada impermeable

Huelga decir que la severidad de contaminación, el volumen y la localidad del sedimento contaminado a ser removido, tendrá una gran influencia en la selección del equipo.
Una draga de rosario parece ser entonces una alternativa más razonable que una draga cortadora.

2.- Hasta el momento no se ha identificado un equipo específico de dragado, ni equipo auxiliar, para la remosión de material poluto en suspenso.

3.- En Holanda (IHC-Cosmos), ha desarrollado un equipo auxiliar tipo "espumadera", para la remosión de polución en la superficie del agua (como: derramamiento de petróleo), el cual puede ser acoplado a una draga de succión por arrastre especialmente diseñada, la que es utilizada en un contrato de dragado de mantenimiento de cinco años, en el canal de acceso del puerto de Rotterdam.

EPILOGO

La industria del dragado antes de ser identificada como un degradante del ambiente natural y de la calidad del agua por causar turbidez, se está ocupando actualmente de la limpieza del medio ambiente y se espera que en el futuro juege un importante y creciente rol.

Ing. J.P.C. van den Kieboom
Febrero 1982

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IX JORNADAS DE INGENIERIA CIVIL

TEMA V:

THE HISTORY AND PHILOSOPHY OF COASTAL PROTECTION

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THE HISTORY AND PHILOSOPHY OF COASTAL PROTECTION

by

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Dedicated by the author to:
Andries Vierlingh
Dikemaster, Netherlands,
for his "Tractaet van Dijckagie" ("Treatise on Dikebuilding") 1576-1578.

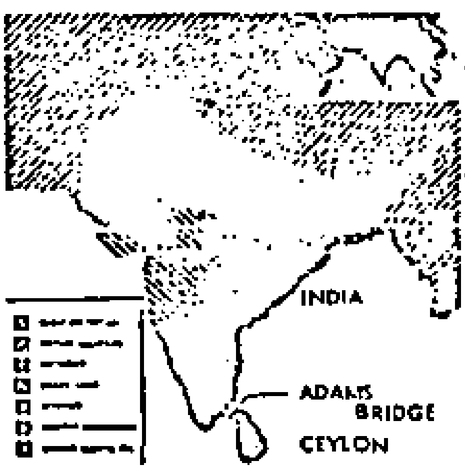
"Water shall not be compelled by any 'force', or it will return that force onto you".

Abstract - This paper gives a brief review of the history of coastal defence as it has developed since the year 1,000. It makes an attempt to outline what may be termed "the philosophy" and its relation to development.

The "State of the Art" is described by figures of characteristic designs including new developments. The paper establishes some general rules for future coastal protection and gives an outlook for the future.

HOW OLD IS THE ART OF COASTAL PROTECTION

We do not know; probably very old. Adam and Eve escaped from the gardens of Paradise located on the island of Ceylon following their blunder in an apple orchard. They crossed the waters on "Adams Bridge" which was not only a "bridge" (reef) but a coastal protection for a major part of the most southern SE coast of India.



Coastal protective works of major order probably first came into existence when man was forced to protect the land which he lived on to avoid the waters digging out the ground under his feet.

COASTAL PROTECTION IN THE LOW COUNTRIES IN EUROPE AND IN ENGLAND

NETHERLANDS

Although a great many training and irrigation walls, dams or dykes were built in the Far and Middle East coastal protection per se probably first developed in the low

TEMA 2

countries in Europe where rivers poured soft materials, mainly clay and silt, out in the ocean for settling. Consolidation was a slow process which made the land settle. In addition sea level was rising. To avoid loss of land by flooding and to protect themselves from drowning the Frisians and the Dutch first built earth mounds. Dyking started about the year 1,000. In the 13th century the Dutch had accomplished major coastal protection and reclamation works, particularly in the Dordrecht area.

Dr. van Veen in his book "The Art of a Nation" (1) writes: "The earliest written records about the Frisians (or Coastal Dutch) describe them as water-men and mud-workers. The Romans found in the North of the country the artificial hillocks upon which the inhabitants, already called 'Frisii', made a living. We shall follow their history, because written records are available about the early reclamation works they made. One and the same race, now called the Dutch, took, held and made the low country.

Pliny, who saw these mound-dwelling tribes in the year 47 A.D. described them as a poor people. He apparently exaggerated when he wrote that they had no cattle at all. Or did he see some much-exposed mounds near the outer shores where the sea had swallowed every bit of marshland? At stormtide, Pliny said, the Frisians resembled groups of miserable shipwrecked sailors, marooned on the top of their self-made mounds in the midst of a waste of water. It was impossible to say whether the country belonged to the land or to the sea. 'They try to warm their frozen bowels by burning mud, dug with their hands out of the earth and dried to some extent in the wind more than in the sun, which one hardly ever sees'.

No doubt the mud Pliny refers to was the peat which was found in the 'wolds', or swamps, some distance south of the clay marshes, where the artificial mounds had been made."

"In all they built 1240 of these mounds in the northeastern part of the Netherlands, an area of a mere 60 x 12 miles. Further East there are more of them in East Friesland. The areas of the mounds themselves vary from 5 to 40 acres; they rise sometimes to a height of 30 feet above normal sea level. The contents of a single mound may be up to a million cubic yards."

"They built their mounds on the shores of the creeks in which the tide ebbed and flowed. In their scows they went (in their language in which the roots of so many English words can be found): 'uth mitha ebbe, up mitha flood' - out with the ebb, up with the flood. The tide bore them towards the peat regions, or perhaps to the woods still farther inland and then brought them back. Or they went out with the ebb in the morning towards the sea, where they gathered their food, and returned in the evening with the incoming tide."

"The Coastal Dutch have now lived 24 centuries in their marshes and of these the first 20 or 21 were spent in peril. It was not until 1600 or 1700 that some reasonable security from flooding was achieved. During these long treacherous centuries the artificial mounds made their survival possible."

"It was a work which might be compared with the building of the pyramids. The pyramid of Cheops has a content of 3,500,000 cubic yards, that of Chephren 3,000,000 and that of Mycerinus 400,000 cubic yards. The amount of clay carried into the mounds of the northeastern part of the Netherlands can be estimated at 100,000,000 cubic yards.

In Egypt it was a great and very powerful nation which built the pyramids throughout a series of dynasties. The aim was to glorify the Pharaohs. With us it was a struggling people, very small in number and often decimated, patiently lifting their race above the dangers of the sea, creating large monuments, not in stone, but in native clay."

"In this Lex Frisionum of 802 there is not yet any mention of seawalls, but the first attempts at dike building must have been made shortly afterwards. Frisian manuscripts still extant, dating from the early Middle Ages, deal chiefly with the following three points: First, the right of the people to freedom, all of them, 'the born and the unborn'. Secondly, the 'wild Norsemen' whose invasions took place roughly from 800 to 1000, and thirdly: the Zeeburgh or Seawall.

This novel means of defence against the sea by means of a continuous clay wall was called a Burgh, or stronghold. The people were apparently very proud of this seaburgh, because they described it in tical language as 'the Golden Hop', the Golden Hoop.

–'This is also the Right of the Land to make and maintain a Golden Hoop that lies all around our country where the salt sea swells both by day and by night'." (Plate 1, Fig. 1).

The spade, the hand barrow and the fork were the instruments used for diking, the fork presumably for the grass turfs which were used to brighten the dykes and make them stronger. Despite the tremendous efforts the sea was the strongest. "This was due partly to our insufficient technical skill and partly to lack of co-operation. For a single night, Dec. 14th 1287, the officials and priests estimated that 50,000 people had been drowned in the coastal district between Stavoren and the Lea. This is a large number considering that this was the area where so many dwelling mounds could be used as places of refuge."

The advances and successes have been tied to a few names. Says van Veen:
"We often wondered who was the master engineer who created the marvellous Great Holland Polder, south of Dordrecht, the work which had included the damming off of the tidal mouth of the river Maas, and the leading of that river into the Rhine. This proved to be William I. He had already finished that gigantic undertaking by 1213. The polder was destroyed in 1421 by the St. Elisabeth's flood, described in a former chapter. William was a man of great conceptions. He surrounded the entire area of Holland-Proper with strong dikes and made several canals intended to drain the vast moors. They also served as a splendid network of shipping canals. It is likely that he made the dikes around the Zeeland islands Walcheren and Schouwen too, and that he established the still-existing administrations for the upkeep of these islands. The other part of his clever and amazing reclamation and construction programme cannot be described here, but it is very clear that he knew the geography of his county by heart. No maps as yet existed!"

The earliest reference to the art of accelerating the natural rate of accretion is the manuscript "Tractaet van Dijckagie" (Treatise on Dikebuilding), written by the Dutch dykemaster ANDRIES VIERLINGH, between 1576 and 1579. VIERLINGH discusses the construction of "cross-dams" on mud-flats which are not yet dry at low water. In this connection he also advises that old ships should be sunk and earth dumped on the top of them so as to make artificial islands or flats which should hold back the silt and sand suspended in the water. These islands should subsequently be connected with low dams. Although this method has not been used commonly it is known that shipwrecks have been used at numerous places to close dyke breaches. These wrecks formed the basis for the fill material which was secured with mats or brushwood. VIERLINGH, however, was much against closing of dyke breaches with shipwrecks due to the non-homogenity they created in the dyke structure. Nevertheless, this method was widely used over a long period of time, not only in Holland but in the (at that time Danish) Schleswig Holstein.

"Vierlingh was found to be a real master of the dikes and waters, a man of great ability and spirit – one of the greatest of his kind. Luckily the greater part of his manuscript has survived. Its ancient picturesque style is a joy to every hydraulic engineer. This remarkable book already shows the special vocabulary of the Dutch diking people in all its present-day richness. In some ways it is even richer.

His advice is simple and sound. The leading thought is:
<u>Water will not be compelled by any 'fortse' (force), or it will return that force onto you.</u>

This is the principle of streamlines. Sudden changes in curves or cross-sections must be avoided. It is the law of action and reaction. And truly, this fundamental law of hydraulics must be thoroughly absorbed by any one who wants to be a master of tidal rivers."

The work by the dykemasters and farmers to protect and to gain land has been remarkable. Plate 1, Figs. 2 and 3 (10) give an impression of how dykes were built up gradually by adding one layer of silt, or silt and sand, shell, willow mattresses etc. on the top of each other. Remains of old ships, brick walls and pile walls were used . No less than two-thirds of the lower part of the Netherlands is manmade, while the other third is just "natural" sea marsh or moorish swamp. Since about the year 1200 the fol[illegible]ng areas [illegible] gai[illegible] according to van Veen:

On the sea shores	940,000 acres	
By pumping lakes dry	345,000 "	
By pumping the Zuiderzee dry	550,000 "	(partly future)
In all	1,835,000 acres	

With respect to the distribution of fill the 100,000,000 cubic yards of earth fill which the Dutch in the early centuries carried to their artificial hills were made only in a small area, covering roughly 8% of the country. Van Veen writes:
"The sea walls or dikes were our second work. In 1860, that is just before the advent of steam dredging, we had about 1750 miles of them, containing about 200,000,000 cubic yards of material. Moreover, there were many old deserted dikes, whose contents may be estimated at 50,000,000 cubic yards. These 250,000,000 cubic yards were practically all transported by handbarrows, wheel-barrows and horse-drawn carts.

The third great work was the digging of the ditches and canals. In the lower half of the country about 800,000,000 cubic yards of earth have been removed, in order to drain the land and separate the fields. Of shipping canals there are about 4500 miles in Holland, for which a figure of 200,000,000 cubic yards would be a fair estimate.

The fourth and greatest task was the digging of peat. This digging served a double purpose: the provision of fuel and the creation of lakes which, when drained, gave more fertile land than the original moors themselves.

In total we have dug according to this rough estimate the enormous volume of some 10,000,000,000 cubic yards. This includes the making of lakes as well as the digging of moors in the higher eastern regions of the Netherlands.

Compare this figure with the dredging of the Suez Canal. We constructed about 100 Suez Canals of the size made by De Lesseps. All this was done by hand, whereas De Lesseps used 60 steam dredges."

But the work would never have been completed without the dykemasters, their foremen and "polderboys", who often were the farmers themselves. Figs. 4 and 5 (Plate 1) show them repairing dykes and building willow mattresses for bottom protection, an old but still active art.

A special kind of dykebuilding was the weed-dykes. Construction was limited to West-Friesland and the Zuiderzee-area, where sea-weed or sea-grass was found in ample quantities along the coast. The West-Frisian sea dyke were for a long time reinforced with seaweed, and so were some of the Wieringen dykes. It is not known with certainty how old the weed-dykes are, but weed dykes were constructed from the 8th century. A 16th-17th-century weed dyke was built at the northernmost point of the Island of Schokland, and another one in 1734 in the Northern part of Noord-Holland. Sea-grass for dyke building was collected offshore in the Zuiderzee and the Wadden area. Following drying, a broad, tough layer was placed on the sea side of the dike.

As ambition grew, dykes also grew. Moving them still closer to the dangers it became necessary to reinforce the dykes by hard surfaces like basalt blocks and/or other structures parallel as well as perpendicular to shore. These reinforcing or supporting structures developed as experience and exposure increased. The gradual reinforcement by structures like seawalls and groynes may have made a contribution to a not fully justified sense of security. It has been claimed that dykes were not raised rapidly enough in step with the sinking of the land and the rise of sea level and that dykes were not subjected to thorough investigation of their structural soundness.

On February 1-4th, 1953, a spring tide whipped up by a raging gale overwhelmed the sea defences, and made tremendous breaches in the dykes (Plate 2, Fig. 6) and most of the islands in the south-west were inundated. 1850 people lost their lives. All the available material and manpower was mobilized and within a year all the gaps in the dykes had been closed and the flooded areas once more reclaimed, (Plate 2, Fig. 7). On November 5th, 1957, the "Delta Bill" was passed, containing plans for closing the tidal entrances in the south-west (Plate 2, Fig. 8). When this project has been completed the Dutch coast will have been shortened by 700 kilometers. The Delta project provides for the closure by means of massive dams of four broad, deep sea inlets, viz. the Haringvliet (1968), Veerse Gat (1961), the Brouwershavense Gat (1972) and the Eastern Scheldt (1978) and for the building of secondary dams in the Zandkreek, the

Gravelingen and the Volkerak. The Rotterdam Waterway and the Western Scheldt will be left open, since they provide access to the ports of Rotterdam and Antwerp respectively. This sequence was chosen after due consideration, since the transition from small to large sea arms enables experience gained to be profitably used in the larger projects. Another reason is the desire to achieve a higher degree of safety for the largest possible area in the shortest possible time. This - the world's largest coastal protection project - is thoroughly described in a number of publications and in the Dutch periodical "Deltawerken" published by the "Delta dienst". The status of this project at this time (July 1972) is that the Veerse Gat and the Haringvliet have been closed according to schedule: Two sections of the Brouwershavense Gat were closed in the spring of 1971, and the work will be completed by 1972. The southern gap was closed by cableway (concrete blocks dumped from cable cars), the northern one by means of 14 caissons. The closing of the last gap meant that tidal currents involving the movement of 317 million cubic metres of water into and out of the inlet (each movement taking about 6 hours) ceased to flow. There remains the dam which will close off the Eastern Scheldt. This will be about 9 kilometres long and will stop tidal currents involving the movement of 1,100 million cubic metres of water into and out of the inlet every twelve hours. The construction of 3 artificial islands was needed to build the dam: the first was completed in 1969, the second in 1970 and the third in 1971. This dam, the last and largest to be constructed (it fills up channels as deep as 35 metres), is expected to be completed by 1978.

The construction of the large sluices presented enormous problems which were solved. Protection of the bottom was obtained by placement of large "Zinkstukken", upholding a 1,000 year old tradition. Although many tool and construction practices have changed, willow mattresses (Plate 2, Fig. 9) are still in use but they may in some cases have been replaced by mattresses of asphalt or synthetic sheets (Plate 2, Fig. 10). The cost of the Deltaproject by 1978 is estimated to be approximately 3,300 million guilders ($ 1.1 billion). It is an expensive project but it ensures greater safety for the entire Southwest of the Netherlands, reduces the cost of dyke maintenance due to the coastline's shortening by nearly 700 km, opens up a whole series of islands, reduces silting, offers fast traffic links across the dams, and improves control of the supply of fresh water in almost the whole of Holland. In addition it provides new recreational possibilities for the vast population in the southwest urban areas and the provision of unique aquatic sports areas.

The development of Dutch groins. - A few remarks should be made specifically on the Dutch groins. The first groins were probably built at the beginning of the 15th century, but groin-like structures may have been built much earlier. We do not know exactly how they looked but the history of development during the latest 100 to 150 years is known (Plate 3, Fig. 11) and represents a continuous line of development of a streamlined structure exposing itself as little as possible to 'the forces' of currents and waves. Although groins have grown in size the principles are the same: stone pitching on gravel or mattress in the middle and stones on mattresses on the sides with two or more pile walls as supports (12).

Today's length is usually approximately 200 meters and space between them is of the same order as described in more detail in a later section. Offshore elevations are about M.S.L. Occasionally groins are provided with piggy backs (Plate 3, Fig. 12) to break the longshore currents. Analyses by Bakker and Joustra (7) have demonstrated the ability of the Dutch groin protection which has not only decreased or stopped erosion in certain areas but has even caused accretion. The reason may be sought in the fact that (tidal) currents combined with swell action provided the shore with material from offshore so that the groins did not suffer starvation as often as is normally the case. While the situation at many other places where groins have been built is that erosion continues outside the extreme ends of the groins this, generally speaking, does not seem to be the case along the Dutch ocean shores. It may be said that nature itself made a demonstration of "artificial nourishment" in Holland. The groins, however, are not corner stone in the protection of Holland. This has always been the dykes. But foreigners who came and saw the results of the Dutch groins sometimes misinterpreted the situation very seriously. The massive Danish North Sea groins, Fig.41, which gradually increased in length to several hundred meters at the Thyboron Barriere due to continued shore recession, is just one of these misinterpretations by

which enormous quantities of materials were sacrificed because of earlier insufficient understanding of the mechanism involved. One may ask: Could they have done anything else? The answer apparently is that it would have been difficult in the past but it is much easier today - for which reason it should be done. Misinterpretations also found their way to the New World, with the groins at Miami Beach (Plate 8, Fig. 45) being one of the most startling examples of how groins alone are inadequate as coastal protection. On the other hand it may be fair to say that the Long Island Atlantic shore groins are examples of efforts by groins to live up to the Dutch example. And there are several other examples where conditions were favorable (Plate 8, Figs.42,47).

"The Art of a Nation" became an export article. The Dutch also carried out many dyke and drainage projects in France. According to van Veen (1) the "Hollandries" were most abundant in Germany, Poland and Russia. Along the Molotschna there were 46 Dutch villages in 1836; the district Chortitza had at that time 20 such villages. In Poland there were about 2,000 villages inhabited by the descendants of the Dutch immigrants; in Posen there were 830 villages. The first great canal in the United States, the Erie Canal, was financed in 1772 by the Dutch and its locks were devised by Dutch engineers. Until 1798 the United States of America had no other creditor than Holland.

ENGLAND

In England coastal protection also has a long history because of the continuous erosion of strategic areas on the South Coast, in Lincolnshire, South Yorkshire and in many estuaries. There is already clear evidence of reclamation works by construction of "walls" (dykes) in the Dungeness area during the Roman occupation, the Rhee wall being the best known example. Historical evidence gives a consistent picture of the incursion of the sea along the Lincolnshire coast, by references to loss of land and damage to "sea banks", which had been a necessary defence since the 13th century. In 1335, according to records, the waves breached the sea banks at Mablethorpe and the land was flooded. By 1430 the sea-wall again needed repair. Erosion continued and the history of this area is one tough fight against the sea.

As in Holland, the first measures against erosion were "sea banks", the design being modified to serve as sea walls according to the local situation. Some were just earth dams, others were fascine or pilewalls (7). Later, vertical bulkheads were developed (Plate 3, Fig. 13). On the English shingle beaches abrasion presented a severe problem and called for the application of flint, basalt, or other suitable materials (backed by concrete) to resist abrasion (Plate 3, Figs. 14 a and b). The block walls at Pett Level (Dungeness) (Plate 6, Fig. 25) and Walland (Plate 6, Fig. 26), are mentioned later as examples of modern sloping walls providing flexibility rather than rigidity and low reflection of wave energy thereby being more considerate to the beach in front than vertical or slightly curved structures (11).

Groins were used as an additional protective measure. They were put into use in early times, probably as a result of observations of the effect of hard points protruding from the shore. This was likely to have been the case at Hornsea, South Yorkshire (9) where during an inquisition held in 1609 concerning heavy losses by erosion it was stated that "there was a peere at Hornsea Beach, during the continuance whereof the decay was very little". In 1664 six groins were built on the heavily eroding Spurn Head, South Yorkshire, at the entrance to the River Humber, where nature's forces were assisted by man's removal of shingle from the beach. The groins were of the King Pile type with horizontal boards which could be adjusted similar to the Withernsea Groynes erected in the 1870's (Plate 3, Fig. 15). They were strutted at the down-drift side to resist the pressure of the accumulating beach on the updrift side. Sheet pile groins were also tested, but the result was less satisfactory than the results with King Pile groins. The former were too rigid and lacked any means of adjustment.

Some enthusiasm seems to have resulted from the result of groin construction works of limited length along the shore but observations about ill effects in the form of downdrift erosion were also made. In a discussion of an article by Mr. J. Murray on "Sunderland Docks" printed in the Proc. of Inst.Civ.Engrs. 1849, Mr. Rennie and Mr. Walker refering to a report of 1832 admit "that groins were, under certain circumstances the best defence for a coast, for wherever the waves brought the sand and shingle in quantities, the seaward side filled up while on the lee side it was generally scooped out, but by a judicious distribution of these groins, such an accumulation of material might

drift and partly to the sides by imbalance in longshore drift or

$$\left| dE_{rosion} \right| = dT_{ransversal\ drift} + dL_{ongshore\ drift}$$

$dT_{ransversal}$ - With reference to long range development transversal drift is caused by sea level rise (disregarding sand drift by wind which in some cases may be of considerable importance).

Sea level rise (refs. 19 and 20) may sound innocent, but, realizing how narrow the beach is compared to the offshore area which is to be nourished by erosion of the beach in order to balance the rise of sea level with an equal amount of deposits of material on the bottom (Plate 4, Fig. 16 and ref. 13), it can be understood how an average rise of just 1/8 in per year may cause shoreline recessions ranging from 2 to 3 ft along the Eastern Seaboard of the United States. A general "rule of thumb" is that the shoreline recedes 1 ft for every millimeter which the sea level rises. There is, needless to say, a phase lag between rise and recession (13). An impression of the most recent sea level rises along the U.S. eastern seaboard may be obtained from the following figures by the U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration:

Ave. rise 1940-1970:

Eastport, Maine	1930-69	.338 cm/yr
Portsmouth, New Hampshire	1927-70	.165 "
Woods Hole, Massachusetts	1933-70	.268 "
Newport, Rhode Island	1931-70	.310 "
New London, Connecticut	1939-70	.225 "
New York, New York	1893-70	.287 "
Sandy Hook, New Jersey	1933-70	.457 "
Baltimore, Maryland	1903-70	.259 "
Washington, D.C.	1932-70	.244 "
Portsmouth, Virginia	1936-70	.341 "
Charleston, South Carolina	1922-70	.180 "
Fort Pulaski, Georgia	1936-70	.198 "
Mayport, Florida	1929-70	.235 "
Miami Beach, Florida	1932-70	.192 "
Pensacola, Florida	1924-70	.040 "
Eugene I., Louisiana	1940-70	.905 "
Galveston, Texas	1909-70	.430 "

It is obvious that the only way in which this erosion can be counteracted is by artificial nourishment replacing the material eroded by other material whether from land or from offshore sources, the latter becoming more and more popular due to shortage of land borrow areas. The integrated transversal transport of material including seasonal and long range movements is usually much larger than the longshore. Earlier planning tended to put the main emphasis on longshore transport and measures like groins, breakwaters and sea walls largely concentrated on a "different distribution" or "redistribution" of the material available - but on average there was always a net loss. Partly due to lack of recognition of that fact and partly due to lack of proper equipment to handle the "transversal problem" effectively it was not until the last two decades that it was realized that ultimately the only way in which erosion may be fought is by artificial nourishment replacing the material eroded by other material. The only place where this general rule may be disregarded is where nature itself provides the nourishment. It was the "New World" which on purely coastal protective basis carried the initiative and it was there that efforts concentrated on the main and large problem of bringing material back to shore to replace the quantity which was lost by "submerging of the profile".

$dL_{ongshore}$ - It is generally accepted and justified by a great number of laboratory and field observations that the longshore transport of material (Q) has an almost linear relationship to the longshore input of wave energy (E) as:

$$\frac{dQ}{dx} = k\,\frac{dE}{dx}$$

when k is a factor which depends upon material and profile characteristics. Only wave induced currents are assumed to be present. This in turn means that the longshore transport depends upon the curvature of the shoreline. In nature this curvature is a

function of natural geological and coastal morphological conditions like the existence of headlands, bedrock and river outlets (Table 1). This may upset the balance equation so that

$$\frac{dQ}{dx} \gtrless k\,\frac{dE}{dx}$$

In case of > accumulation will result
In case of < erosion will result

Nature demonstrates both cases e.g. by updrift accumulation and downdrift erosion at headland.

Table 1. Causes of Erosion by Nature and by Man

Nature	Man
Rise of Sea Level	Dams, dykes and other coastal structures causing rise and concentrations of tides
Protruding headlands, reefs and rocks causing downdrift erosion	Groins, breakwaters, jetties etc. causing downdrift erosion
Tidal entrances and rivers causing interruption of littoral drift	Man-made entrances causing interruption of littoral drift
Shoreline geometry causing rapid increase of drift quantity	Fills protruding in the ocean to such an extent that they change local shoreline geometry radically
Blocking of river outlets carrying sediments to the shore by flood stage barriers, change of location of outlets due to floods, erosion, tectonic movements etc.	Damming up of rivers without providing material sluices Irrigation projects decreasing flow of water and sediments to the shore Removal of material from beaches for construction and other purposes

What destruction effect nature in its abundance has demonstrated man unfortunately has imitated. Man-made erosion is a black spot on man's association with shores. It is a deplorable fact that all coastal protective measures apart from artificial nourishment (may) have an adverse effect on adjoining shores. An example is provided by a group of groins built in the 30'ies on the Danish North Sea Coast which caused severe lee-side erosion, up to 30 ft recession per year (situation corrected at a later date). Dams in California cut off river supply of sediments. Long harbor breakwaters and navigation channels often became almost complete littoral barriers causing severe erosion.

To evaluate the erosion quantitatively, records of erosion of profiles are needed. Such information may be available for areas of limited size but only a few countries have kept continuous records of areas of larger size through a long period of time. This includes the low countries in Europe e.g. parts of Denmark, England, France, Germany and Holland. Mostly only shoreline movements have been followed. A very illustrative example is given by Bakker and Joustra (2) whose paper does not only include data on shoreline movements on the open shore and at tidal entrances but also compares shoreline movements of groined and non-groined shores appraising the effectiveness of groin protection. Similar records are available in a few other places, e.g. in Denmark, Germany and some places in the U.S.A.

Several publications and books give considerable information on the development of erosion and on coastal morphological features (2, 3, 7, 9, 17, 21, 22, 34, 69). A great number deal with seasonal changes (13, 16, 18).

Space photography is now providing a tool for large scale checking of coastal movements (73 and re-references).

A very special type of erosion occur in Norwegian fjords, demonstrating that disasters of tremendous dimensions can occur in subaqueous slopes in fine sand and coarse silt.

The explanation of why these slides frequently reach very large dimensions can be found

be produced, as would effectually protect a shore, or any sea works".

Inexpensive types were devised. Mr. Murray in Proc. of the Inst. of Civ.Engrs., 1847, discusses the design of groins and says "Groins might be formed with stones, timber or fascines, either of the two first-named materials lasted well, but in cases where the deposit was rapid, and of such nature as to entirely fill up interstices, and prevent decay, the latter material would be sufficiently durable for all ordinary purposes".

The entire situation with respect to Sea Protection works was reviewed by a "Royal Commission on Coast Erosion etc." whose report was printed in 1911 by H.M. Stationary Office. One of the most significant references in this report is the statements that sea walls, unless properly constructed are "agents of their own destruction". In particular it refers to scour at the toe and the necessity of constructing a special toe, apron or groin protection in front of the sea wall to prevent undercutting.

With respect to groins advantages and disadvantages were fully realized. "The evidence laid before us goes to show that in many cases on the coast of the United Kingdom groins have been constructed of a greater height than was necessary to fulfil the required conditions, with the result that they have so unduly interfered with the travel of the shingle as to lead to impoverishment of the beach to leeward, causing in many districts serious injury to the coast".

The length of groins and the distance between them is discussed and 1 to 1 ratios are common but "satisfactory results were also obtained by 1 to 2 ratios". Alignment at right angles to the shore was best and provision for adjustment by adding or removing planks was preferable as low groins often proved to be more efficient than high ones being less adverse to downdrift beaches at the same time.

Reading this British document more than 60 years old, one regrets that the wisdom it contained was realized so late elsewhere and that designs as contradictory as possible to the century old British experience were advocated for a long period of time and to some extent still are being promoted. The difference between British and Dutch practice in groin design is related to the grain size of the material which the groins are supposed expected to accumulate. In England a good many beaches are of shingle, and some of them experience a high rate of beach drift and significant fluctuations in beach profiles. A high (but adjustable) groin may therefore be practical. Energy loss along the stem is of less importance due to the coarseness of the material. Conversely in Holland all beaches consist of fine to medium sand which moves easily and fluctuations of beach profiles are of relatively small magnitude. Smooth streamlined cross sectional geometry causing little turbulence is therefore best for such conditions and groins should be low to conform with relatively gentle sand slopes. Groins having high vertical walls would result in scour and lowering of the beach on the either side of the groin.

DENMARK

In Denmark coastal protection started on the North Sea Coast in 1840 with a government project to increase the height of dunes on the Lime Fiord Barriers (3). In the 1870's experimental groins were built on the West Coast using a Dutch design which soon proved to be too weak to withstand the violent wave action on that shore. The design was reinforced and over the next 50-60 years almost 100 massive groins ranging in length from ab. 100 m to ab. 400 m were built in this general area of approximately 50 km length generally using a concrete block design (Plate 8, Fig. 43) of blocks ranging from 4 to 8 tn. now often provided with side slopes of 2 to 6tn granite. Blocks are placed with specially designed cranes. Erosion continued outside the extreme end of the groins and the outer parts were not kept up. The land ends were extended gradually as the dunes and dikes were withdrawn (3). Artificial nourishment from bay or offshore sources has – surprisingly enough – not been applied yet but is urgently needed particularly on the Lime Fiord Barriers.

COASTAL PROTECTION IN NORTH AMERICA

In the New World the professional history on how local problems were solved is becoming old too, but "public history" is new. It may be described briefly with a few notes (4,5).

Before 1930, Federal interest in shore problems was limited to the protection of Federal property and improvements for navigation. At that time, an advisory "Board on Sand Movement and Beach Erosion" appointed by the Chief of Engineers was the principal instrumentality of the Federal Government in this field. In 1930, the Congress assumed a broader role in shore protection by authorizing creation of the Beach Erosion Board. Four of the seven members of the Board were Corps of Engineers officers and the other three were from State agencies. It was empowered to make studies of beach erosion problems at the request of, and in co-operation with, cities, counties, or States. The Federal Government bore up to half of the cost of each study but did not bear any of the construction costs unless federally owned property was involved.

This important first step was followed by a series of improvements, in 1945, 1955, 1962, 1965 and 1968 demonstrating a still increasing interest and involvement in the matter by federal authorities (4, 5). Several states created their own beach erosion and shore development agencies which established co-operation with the local U.S. Army Corps of Engineers District and Division offices and with the Office of the Chief of Engineers. A great number of studies of actual beach erosion problems followed by reports to Congress by the Secretary of the Army authorizations and finally by federal contributions to actual improvements. These efforts were supported by research projects by the Beach Erosion Board and from 1963 by the Coastal Engineering Research Center (CERC). A number of special projects were handled by model tests at the Waterways Experiment Station of the USCE and CERC.

Structurally speaking the art of coastal protection suffered shortcomings compared to the low contries in Europe. Patented more or less useless coastal protection devices as e.g. permeable groins, have had a bigger chance in U.S. business than elsewhere but the newest and most effective measure, artificial nourishment, although not born in the States, was raised there and has so far been most successful in the United States.

Being "philosophical" one may say that the difference between the European low country and the American coastal protection practice lies in "the scale" and in "the degree of involvement". The European is "high", but "short" and often "complex". The American is "long", relatively "low" (excluding hurricane protections) but "relatively simple".

The European practice is tough and silent, the American is flexible and it makes some noise because it is not only a measure but also a "nourishing machine".

WHAT WAS PAST EXPERIENCE? HOW SHALL IT BE UTILIZED?

The combined experience gained through years of struggling may be expressed briefly as:

1) Whatever you do, avoid waves and currents turning their full forces onto you (Vierlingh, 1570's).
2) Don't be nearsighted, think large if you possibly can. It is better to solve problems of some kilometers or miles than only of some meters or feet.
3) Look oceanward, landward and up and down the shore and evaluate carefully how your plans may be influenced by or influence the surrounding areas of land and water.
4) Coastal Protection does not necessarily need to be just coastal defence. Old Dutch experience and military tradition seems to favour defence by attack. In a war it is always best to keep the initiative and not leave it to the enemy.

An American version of this experience may be expressed as "the best protection for real estate is plenty of real estate in front of the real estate you want to protect", an approach which suits miles and kilometers and also matches economy as the general experience is that you (almost) always get a bargain when you order large quantities.

REASONS FOR BEACH EROSION

In order to define and discuss the problem it is necessary to go back to its roots. Erosion is caused by the forces of nature, sometimes assisted by man-made structures or by man's active erosion by removal of material from the shore.

Table 1 is a review of reasons for natural and man-made erosion. If no erosion is to take place from a particular shore there must be a balance between the quantity of material which arrives and the quantity of material which departs. Let us consider a shore which is not in balance but is losing material partly offshore by transversal

in the two characteristic properties of these materials: The first property is the complete loss in strength after a shear failure which is characteristic for loose fine sand and coarse silt. As the result of this property the slide masses assume the character of a viscous liquid and, in the first place, flow downwards from the slide scar and possibly initiate new slides by erosion. In the second place, the disappearance of the slide debris means that the faces of the slide scar are left unsupported, involving the risk of an extension of the slide in an uphill direction by a retrogressive, slice by slice, development.

The second property of submarine deposits of fine sand and coarse silt responsible for the disasterous character of the slides is their exceptionally high erodability. The reason why these deposits are easily attacked by erosion is partly the lack of cohesion of fine sand and coarse silt particles and partly the lack of a protective cover of top soil and vegetation in submarine deposits. The consequence of the high erodability is that if a flow slide descends over this type of deposit, it will cut deep canyons into the slopes and undermine any obstruction which deflects it from its original path. The scars of such an erosion can lead to the development of a new series of retrogressive slides which can contribute both by extending the slide and by adding further liquid sand to the flowing masses. In the postglacial deltas and estuaries occupying the head of the fjords in middle Norway, the fine sand and silt slopes frequently stand steeply. In most cases the deposits are continuously growing as a result of accumulation of sandy and silty material being carried out into the fjords by the rivers. Under these conditions even small man-made fillings may initiate a slide of considerable size. A factor which may contribute to the instability of the submarine slopes in these fjords is artesian pore pressure conditions originating from high water pressures in the fissures of the bedrock beneath the deposits.

For further information on this "soil mechanics" erosion the reader is referred to a paper by Dr. L. Bjerrum, Proceedings of "The First International Conference on Port and Ocean Engineering", the Technical University of Norway, August 1971, pp. 24-26.

NATURE'S COASTAL PROTECTION. MAN'S COUNTERPARTS

By good luck nature has not only demonstrated how to erode but also how to protect. It may safely be said that there is no protection initiated by man which has not beforehand been invented by nature, and nature obtained all the good results as well as all the bad results before man did. Consequently we can learn from nature if we will only make the effort of opening our eyes and looking. It must be admitted that nature has been more imaginative and has had more success than man. Perhaps one reason for this may be sought in the fact that what we see is mainly the result of the successes. In the case of failures little or nothing was left! Coastal geographers and geologists, often unintentionally, describe natures coastal protection (e.g. 24 and 25).

Table 2. Nature's Coastal Protection. Man's Counterparts

Nature	Man
Shore rock	Sea wall
Rock reef	Submerged bulkhead or mound
Rock island	Offshore breakwaters
Headland	Large breakwater perp. to or at an angle with the shoreline
Rock perp. to shore	Groins
Sea floor vegetation	Bottom Mattresses
Sea surface vegetation	Floating breakwater
Dune	Dyke
Material transfer to shore by:	
Wind drift	
Rivers	Artificial nourishment from land sources
Shore erosion	
Longshore Littoral Drift	
Sea bottom transfer	
Natural by-passing of drift at tidal inlets	Artificial nourishment from offshore sources
	Mechanical by-passing of drift at tidal inlets

Table 2 gives examples of nature's protective measures and imitations of them by man. It is obvious that nature has immense resources and is able to play a "full orchestra" where man's instrumentation is somewhat limited by the lack of proper tools and adequate funding, one depending upon the other.

Plate 4 Fig. 17 shows updrift accumulation at a large rock headland, "Portland", on the Icelandic South coast. Plate 4 Fig. 19 demonstrates natural offshore breakwater protection causing tombolo formation in Dorset, England; Plate 4 Fig. 18 pocket beaches formed by outcroppings of coral rock and natural rock sea walls on the east coast of Puerto Rico (Palmas Del Mar) and Plate 4 Fig. 20 huge outpours of material by a glacial river in the Arctic. All glaciers have outbreaks of water reservoirs during the summer season. In Iceland subglacial vulcanoes, when erupting, occasionally cause discharges of the order of 1 billion cubic yards of material including bed load "particles" of 20 ts.

COASTAL PROTECTION A B C

The main cases of need for coastal protection are listed in Table 3, distinguishing between measures to be taken on large and on small scales.

Table 3. Main Cases of Needs for Coastal Protection

	Large Scale	Local (small) Scale
Reclamation of land and protection of the reclaimed land	x	
Protection of property and structures on the coast	x	x
Construction and protection of beaches	x	Pocket beaches

Table 4 is an attempt to establish a detailed classification of the types of coastal protection which are available today. Distinction is made between functional, operational and hydraulic or wave mechanics characteristics.

Table 5. Various Coastal Protection Measures Classified in Accordance with their Ability to Provide Protection of Extensive and Local Areas and their Influence on Adjoining Shores.

	Large Scale	Small Scale	Effect	Influence on neighbouring shores
Groins	(x)	x	May stop or decrease shoreline recession but not if offshore erosion continues	Adverse, often very severely
Sea Walls	x	x	Stop erosion where they are built but do not stop offshore erosion	May to some extent become adverse
Shore parallel breakwater	(x)	x	Will probably stop erosion and build up beach where they are erected	Adverse, often very severely
Artificial nourishment	x		Widens beaches Provides full protection if well maintained	Beneficial

(x) less attractive solution

TABLE 4

[illegible]

Table 5 lists different coastal protection measures and their relative ability in providing the protection, their influence on adjoining shores (beneficial or adverse).

Before we can evaluate which protection is preferable the situation with regard to erosion has to be appraised. This may by done by the introduction of the terminologies: "undernourished", "sufficiently nourished" and "overnourished profiles" and by the terminologies "source" and "drain" of materials (38).

Beach Profiles Classified in Accordance with Nourishment - Considerations on the basis of the development of beach profiles built up of sand with grain size 0.2 to 0.3 mm seem to show that we can distinguish between profiles in another way: that is, between the "overnourished", the "sufficiently nourished", and the "undernourished profiles. These terminologies are especially valuable for an understanding of the problem of what kind of coastal protection should be preferred and how satisfactory such construction will be. The overnourished beach profiles are fed with more material than the waves can shape into real beach profiles. These, therefore, are irregular and often perform as irregular shoals. There are two different types of sufficiently-nourished profiles. At one of them the profiles are not fed with more material than the wave can shape into a profile having the same "equilibrium form". At the other, the loss of material equals the supply of material and the profile still has the same equilibrium form. The undernourished beach profiles are eroded: that is, the coastline retrogrades. The undernourished beach profiles will always keep an equilibrium form but the form may change from one locality to another, depending on the conditions in general. It seems, therefore, that progradation of a coast may take place with or without equilibrium profiles while retrogradation of a shoreline can take place only with equilibrium profiles having a maximum steepness corresponding to the quantity of littoral drift, the waves and the material. An actual equilibrium profile therefore should be defined as a stable profile with maximum steepness. Needless to say, we cannot expect that all undernourished profiles will always have the same shape, but we can expect that all undernourished and sufficiently-nourished profiles will have certain standard forms, which in turn means that where one of the standard erosion forms occurs, we know that the erosion is probably not temporary. This information is very important. If on the other hand no erosion takes place we can expect that only a slight change in the littoral drift balance may start erosion.

A source of materials is a coastal area which delivers materials to other beaches. A source might be an area where erosion takes place, a shoal in the sea, for instance; the shallow area in front of an inlet which has been closed; a river which transports sand material to the nearshore sea territory, or sand drift from dunes to the beach. Artificial nourishment of any kind is also a source.

A drain of materials is a coastal area where materials are deposited. A drain might be a marine foreland of any kind, a spit, recurved spit, a tombolo, angular foreland, etc. It might also be a bay, an inlet, or a shoal. Man-made constructions such as jetties, groins, or dredged sand traps, are also drains.

Both terminologies are used in the section dealing with coastal morphology in relation to problems of beach erosion and coastal protection. In practical coastal protection technology the following general rules are valid:

1). A coastal protection should be built in such a way that it functions as a drain. It should therefore have a source but not a drain on the updrift side. If there is a drain the coastal protection will not be very successful unless material is supplied artificially.

2) A harbor or an improved inlet on a littoral drift coast should not act as a drain. It should therefore have no source but if possible a drain on the updrift side. Meanwhile it is very difficult to find a place where ideal conditions exist, and many other factors play an important role. Most harbors are built in a sheltered area, an inlet, a bay or in a river mouth. In such areas depositions will almost always take place either from the littoral drift or as silting, which means that the harbor actually functions as a drain. This is the case with numerous harbors all over the world, notably on the East Coast of the United States. Protection against the littoral drift can be effected by the construction of jetties, making the "improved inlet". An improved inlet acts as a drain and protects the inlet, but at the same time it cuts off the supply of material to the beaches on the lee side which again means that these

TABLE 6

TABLE 7. DETAILS OF THE PERFORMANCE OF SEA WALLS.

		Comments:
1. What is wanted:	Storm tide and/or extreme storm protection of shore and lands. Protection of specific valuable areas (industry, buildings, highways etc.)	Energy-absorbing wall or revetment on dyke or dune. Any type of substantial wall with as little adverse effects as possible.
2. Layout and geometry:	As streamlined as possible. (It is best to leave and maintain a beach in front of the wall).	Erosion may be stopped at the wall but artificial nourishment may be needed to maintain beach in front of the wall.
	Influence on adjoining shores.	Leeside erosion may result if erosion continues leaving wall as protruding headland or if wall is built too far seaward and is not streamlined in horizontal geometry. Transfer of sand or other nourishment of downdrift shore may be needed.
3. Combination with other coastal protective measures:	Groins.	To break longshore current and possibly build up beach in front of wall.
	Artificial nourishment.	To maintain beach in front of wall and/or to check downdrift effects.
4. Design:	Energy-absorbing (sloping and/or mound type).	Considerate to beach stability due to friction and low reflection.
	Non-energy-absorbing (vertical sheet pile or slab).	May create local erosion due to less friction against currents and more reflection.

TABLE 8. DETAILS OF THE PERFORMANCE OF GROINS.

		Comments:
1. Degree of efficiency wanted:	Just beach stabilization.	Short groins mainly covering the beach.
	Also widening of beach.	Longer groins, possibly extending beyond bar or breaker zone.
2. Layout and geometry:	Streamlined in horizontal geometry. No sharp turns or corners.	Reaction of shore protected: Stable or widening and then stable. Influence on adjoining shores: Usually beneficial or neutral updrift but adverse downdrift.
3. Combinations with other coastal protective measures:	Sea Walls.	To cope with extreme conditions incl. storm surges.
	Artificial Nourishment.	To fill groins and widen beach initially and maintain width.
		To eliminate adverse effects on downdrift beaches.
4. Design:		
Length in agreement with point 1. Height to match beach profile wanted to the practical extent possible.	Impermeable: Energy absorbing. Non energy absorbing.	Less reflection, less loss of sand. More reflection, more loss of sand.
	Adjustable elevation.	May be operated to match fluctuations of beach
	Fixed elevation.	Can not be operated to match fluctuations of beach.
Length/space ratio from 1:1 to 1:4 depending upon quantity of drift and beach material. Most common ratio is 1:3.	Permeable: May be adjustable or fixed.	"To blow and have flour in your mouth at the same time". May provide beneficial results where currents are the main agents in the transport of materials, thus means in flords and estuaries.

TABLE 9. DETAILS OF THE PERFORMANCE OF OFFSHORE BREAKWATERS.

		Comments
1. What is wanted:	Protection, or protection and beach.	If breakwater is built as littoral drift shore both are usually obtained.
2. Layout and geometry:	Parallel to shore or largely following depth contours.	Tombolo formation will result or shore to be protected. Severe downdrift erosion may result due to littoral barrier effect.
3. Combination with other coastal protective measures:	Groins.	This combination is unlikely unless groins are used to check downdrift erosion, thereby transferring problem further downdrift.
	Sea Walls.	May be built to protect against extreme storms and tides or to check downdrift erosion.
	Artificial Nourishment.	May be used to create beach more rapidly if natural supply of material is limited or to check downdrift erosion.
4. Design:	Energy absorbing structures preferable. See Table 7. Combination with natural reefs often advantageous.	

TABLE 10. DETAILS OF THE PERFORMANCE OF ARTIFICIAL NOURISHMENT.

1. What is wanted:	Protection and beach.
2. Layout and geometry:	Follow natural shoreline closely on straight or streamlined shores. Fill in pockets on headland shores and artificial pockets.
3. Combination with other coastal protective measures:	Groins: to create or maintain beach; to eliminate leeside erosion. Sea Walls: to protect wall and/or create or maintain beach in front of wall; to eliminate leeside erosion. Offshore Breakwaters: to create and maintain beach
4. Design:	Nourishment from land or offshore sources. Offshore equipment under development. Various methods suited to actual operation. Sand shall be suitable for nourishment. Main requirement is that sand should be as coarse or coarser than the natural beach material and of no less specific gravity. By-passing - arrangements by fixed or movable plants incl. weirs and floating plants. Movable arrangements preferable.

beaches starve - having no source on the updrift side. It will be seen, therefore, that coastal protection problems are the reverse of harbor problems.

Table 6 is a general outline of the function of the type of coastal protection in relation to the actual situation of the beach and bottom profiles and to source and drain. When these factors are known, it is possible to evaluate the effects of various kinds of coastal protection and in that way determine the type which is most suitable.

Tables 7 to 10 give basic information on various types of coastal protection: putting the protection including sea walls, groins, offshore breakwaters and artificial nourishment in relation to "what is wanted" and giving some specific information on layout and geometry, combinations with other coastal protective measures and on designs. Plate 5 Figures 21-24, with accompanying note sheets following the plate sheets illustrates dyke protection and dune building. Plates 6-9 Figures 25-35 and the accompanying note sheets describe each particular measure: sea walls, groins, offshore breakwaters and artificial nourishment incl. by-passing of sand. Due to lack of space the number of figures had to be limited to characteristic examples of "the State of the Art". Adequate information function is available in the list of references and bibliography which has been separated in sections referring to each particular measure. Space limitations had to be considered also on this matter. By-passing of material is mentioned specifically in the next section on future coastal protection.

HOW WILL COASTAL PROTECTION DEVELOP IN THE FUTURE

In future coastal protection one must think large. It will therefore develop as a function of the combined political, administrative and technical structure. There will be little or no use for "one-man shows". Large groups and large areas will have to be accomodated - by large scale measures. Needs will be concentrated on protective and recreational projects and all combinations thereof. Pressure will increase by the need for recreational beaches. Protection will be achieved simultaneously. The question of which protective measure will be most practical under such circumstances may be answered by just looking at Tables 5-10 which clearly demonstrate that artificial nourishment with suitable material offers the best large-scale protection. This, however, does not mean that it always suffices. It may need support from dikes and/or sea walls because of the possibility of storm surges or it may need groins to break scouring currents running close to shore. One main technical advantage associated with artificial nourishment is that it is "smooth" and "streamlined" and therefore not only has no adverse leeside effects, but, on the contrary, benefits adjoining shores by a gradual release of material. Other measures, particularly groins and offshore breakwaters, have definite adverse effects on neighbouring shores. The importance of streamlining is obvious from the following elementary reasoning: Most littoral drift formulas relate the quantity of longshore drift, Q, to the longshore component of wave energy as:

$$Q = (K\ wa)\sin 2\alpha_b$$

If the breaker angle α_b increases, Q increases too, assuming that α_b changes only one degree up or down. The resulting relative increase (decrease) of material transport within various ranges of α_b is indicated in Table 11.

Table 11. Relative Increase or Decrease of Longshore Material Transport when α_b various ± 1 (one) degree

Range of α_b		Increase or decrease, approx. Percentage
10°	5°	20%
20°	5°	10%
30°	5°	5%

From these figures it is obvious that any (natural or) man-made discontinuity in shoreline geometry may have a considerable effect on adjoining shores. The beneficial effect is welcome but the adverse is not and it is often severe.

The "face" of the coastal protection will vary from place to place, depending upon local conditions. In Holland protection will have the main saying but recreation will become more and more important. In the United States more and more people are moving to the coastal zone. It is estimated (68, Herbich) that approximately 50% of the entire population will live in the coastal zone by the year 2000. In Florida and in most places, along the Eastern Seaboard of the United States coastal protection and recreational beaches will be combined. In California needs are mostly recreational. In Japan the need for recreational beaches is tremendous. In England sea walls and groins are needed for their steep shores but the demands for sandy beaches in lieu of shingle beaches will increase. In Denmark the massive expensive groin protection on the West Coast will be supported and partly replaced or in some cases abandoned by artificial nourishment from offshore and from bay shoals whenever possible. Accepting this inevitable trend of development it will be an increasing demand for just sand. In addition at some places measures will be needed and justified to hold on to the sand to decrease maintenance. In many areas all over the world reclamation will continue and this requires dykes and reinforcement of dykes by seawalls or revetments. From a technical as well as a coastal ethics point of view there can therefore hardly be any doubt that future coastal protection will comprise of the single or combined measures listed in Table 12.

Table 12. Future Coastal Protective Measures
AN = Artificial Beaches and Nourishment
GR = Groins
SW = Sea Walls and Offshore Breakwaters

Large Scale	Small Scale
AN possibly combined with artificial dunes or dykes providing storm tide protection	SW to protect a particular area sloping structures preferable
AN + SW SW to reinforce dyke or dune against extreme conditions of waves and tides	GR may be justified in local areas if well planned and kept filled by nature and/or by man
AN + GR when GR are justified economically to decrease maintenance costs	SW + GR to protect a particular area where groins are needed as current breakers
AN + SW + GR in unusual difficult cases	

The question which now arises is: how do we provide the optimal solution, technically, economically - and aesthetically? This question may be converted to: how do we get fill suitable for beach nourishment in ample quantities most economically? If it is necessary to build supporting structures as sea walls and/or groins, which design is then the most suitable? The problem which we are faced with concerns an optimalization of coastal protection considering all factors, the initial design as well as future maintenance.

As the need for sand increases the possibilities of securing the fill from land, bay or lagoon sources decreases which means that suitable fill to a still increasing extent must be secured from offshore sources. Such material must fulfill the following demands (34, 37, 61):

a) Grain size shall be as coarse or coarser than natural beach sand
b) Material shall be relatively well sorted with a distribution of particle size to cover all grain sizes present in the original environment

It shall include as little fine material (<0.13 mm) as possible and also little coarse material e.g. particles > 2 mm to avoid separation and a steep and unstable - ever changing - beach.

c) It must be resistant against abrasion (quartz, feldspar and similar minerals)
d) Needless to say, it must also be clean without content of clay, silt and organic matters

e) But - very important - not all material needed to fill a beach has to be "first class". It is enough that all the exposed material is suitable. Below the lower level of fluctuation less adequate material may be placed - just to provide volume and support for the upper floor of "beach material".

Where do we find suitable material? - Every artificial nourishment project includes a hunt to locate proper material which can be secured in an economic way. Such material may be found in borrow areas in nearby lagoons and bays where it is usually fairly easy to dredge it and dump it where it is wanted. Most artificial nourishment projects so far have been based on bay, lagoon and land sources. But it may also be secured from offshore sources. The "sand inventory program" carried out by the U.S. Army Corps of Engineers on the U.S. East Coast revealed the existence of such deposits - of varying origin almost everywhere - but not always within an economic pumping distance (61).

Is sand in ample quantities available offshore? - This depends upon the geological structure and the recent - that means the quarternary-geological development. All shores and shore areas have been subjected to changes in sea level. During submerges shores and beaches were drowned. We find old shores including shore material everywhere. Coastal geomorphologist have dealt extensively with ancient shores (69). During emerges materials returned to shore. Land areas which were subjected to glaciations - and deglaciations - and therefore to high fluctuations in pressure moved up and down with the ice load. At the same time oceans subjected to glaciations received tremendous quantities of icecarried material incl. gravel, sand and clays and this was dropped in the ocean when the ice melted away. The North Sea and Baltic Sea offshore moraines and meltwater deposits are typical examples of that. The consequence is that many sea territories are able to deliver materials suitable for beach nourishment in ample quantities - but, this is not enough. The material also has to be available within a reasonable distance from shore and at depths which makes recovery practical and economical. To investigate the availability of material, core samples should be taken up to the depth of the planned borrow. It is selfexplanatory that the borrow pit must not be located so close to shore that it present a danger to beach stability. This question is dealt with in ref. 59. The 20 ft contour may be the boundary for milder conditions but 30 ft should probably be the minimum depth for conditions on the eastern seaboard of the United States.

How will we then bring this material back to shore? - As it cannot creep itself the only practical way of moving it is by hydraulic power, pumps and pipes. For this we need machinery and a device to carry the machinery. For the latter, three different possibilities seem to exist:

a) offshore mining from a surface vehicle (ship)
b) offshore mining from a vehicle operating on the bottom
c) offshore mining from a fixed or movable platform

re. a - Offshore mining from a surface vehicle - A test on mining of sand offshore was run by the U.S. Army Corps of Engineers in 1966 (67). The U.S. Hopper-Dredge "Goethals" was selected for the operation (Plate 9; Fig. 50). The mooring barge used for discharging from the hopper dredge was anchored in approximately 30 ft of water and its discharge pipe was connected to a 28-inch diameter, 2,000 ft long submerged pipeline running ashore. The line between the discharge piping on the barge and the submerged line, to form a connection from the plant to the ocean floor, needed both flexibility and ruggedness to withstand the lateral and vertical movement and the forces anticipated in this severe service. Much experience of operation and equipment was gained by this test by which fifty-two hopper loads, comprising more than 250,000 cubic yards of sand, were pumped ashore along a 7/10th-mile stretch of beach. The sand fill was piled on the beach to elevations about 5 feet higher than existed previously and the beach was extended seaward some 50 feet.

The Corps of Engineers beach nourishment experiment at Sea Girt, New Jersey, demonstrated that a suitably equipped seagoing hopper dredge could pump sand onto an ocean beach from an offshore mooring, thereby further enhancing the versatility and usefulness of this type of hydraulic dredging plant.

In 1971 a comprehensive nourishment from offshore sources was run at Pompano Beach, Fla.

by C.E.Bean, Inc., La. This work was performed between late April and October of 1970 by a cutterhead-suction dredge. During this period approximately 1,100,000 cubic yards of material was pumped on the beach. The material was located approximately 3-4,000 feet offshore in depths of 30-50 feet of water. The depth of sand available seldom exceeded 15 feet and never exceeded 20 feet. The dredge was 215 ft long, 45 ft wide, and 10 ft deep with a displacement of approximately 2,000 short tons. The pump engine was 3700 h.p. The pipeline used was 25" I.D. The floating line was conventional. The Pompano Beach project was described by the contractor as being "routine in every respect, with the exception of sea conditions". The operation was limited by the inability of the floating pipeline to remain intact when the seas exceeded 4-5 ft in height. It is felt by the operators that the dredge could have operated in seas up to 5-6 ft provided the wave period was relatively short. Long-period waves tended to affect the dredge's capability while short-period waves had more effect on the floating pipeline. Subsurface pipes have now been developed for use in cases where a pipeline must be able to remain in position in bad weather.

The largest beach restoration or creation project is probably the 16 million cubic meters beach fill which was carried out in 1971 at Hook van Holland to create a 100-hectare (250 acres) beach north of the north breakwater of the Rotterdam Waterway. The material was dredged in the deep water channel serving Europort, Rotterdam's new gigantic seaport (62).

re. b - Offshore mining from a vehicle operating on the bottom - The underwater dredge is an old dream which appeared at intervals during the latest two decades. Underwater dredging for minerals has been known for long. Submergence of pipelines in the ocean bottom by jet pumps is of recent date. Similar large scale projects for placement of tunnel pipes across the Straits of Dover and elsewhere have been advocated during recent years. The underwater dredge (Plate 10 Fig. 52) which was put in operation on a test basis in 1970 at Ft. Pierce, Fla., was a result of many years of trial and error (66). A total of 61,000 cubic yards was discharged on the beach from the borrow area 1,200 ft offshore. Many improvements still seem to be needed to make such an operation successful technically and economically. 300,000 thousand cubic yards were pumped ashore by a conventional dredge in continuation of this work.

re. c - Pumping from a platform - Another type of offshore dredge is a result of research undertaken by IHC, Holland, over a long period of years, which resulted in the development of the "platformdredge" (56). Using this dredge a high rate of production can be achieved at considerable depth in currents and swells. Plan and side views of the platform are given in Plate 9, Fig. 51, which shows the dredge with the ladder lowered for dredging at the maximum depth of 25 meters. Supported on three legs, the platform can be moved by means of three twin-spud rotors, in any direction. Length of the L-sides is 30 m. The cutter ladder projects about 12 m when in the raised position. The legs are approximately 38 m in length. At a dredging depth of 25 m and a cutter penetration depth of 2 m, the platform can be jacked up to a height of 4 m above water. Table 13 is a list of data predicted by the IHC (56) comparing output capacities of conventional dredges to the cutter platform dredge.

Table 13. Output Capacity by Conventional and by Platform Dredge (56)

	Conventional cutter dredger with spuds	Conventional cutter dredger with swing wires	Cutter platform
Basic output in m^3/hr	1,000	1,000	1,000
Loss factor for overrunning	-	0,3 m	-
Number of pump-hours attainable per year in calm water	3,100	3,100	3,100
Maximum wave height in even swell	0,30 m	0,75 m	2,0 m
Percentage of workable hours	15%	37%	80%
Actual pump-hours per year	465	1,145	2,480
Annual production in m^3	465,000	573,000	2,480,000

Maintenance - Any artificial nourished beach will suffer loss of material. This raises the question of how to decrease loss of artificially nourished material. This may be accomplished by structures. It is generally accepted that groins are able to slow down longshore drift but loss by transversal drift is probably far more severe, particularly on shores with steep offshore profiles. Addition of shoreparallel breakwaters at the extreme end making a "T-groin" or "mini pocket beach" is an improvement which has been used with success e.g. at Deerfield Beach, Fla. (Plate 3 Figs. 46 and 49) on Lido Key, Fla., on Hilton Head Island, S.C. etc. Another solution is the construction of an "offshore sill". Such sills have been used e.g. on some of the Chicago beaches and on Singer Island, Fla. It is in fact some kind of an offshore breakwater. The difference is that while offshore breakwaters are built in single sections sills are continous training walls providing an offset or step in the bottom profile.

Looking at the experience available in Florida, it may be said that many shores in Florida are already protected by some kind of offshore breakwater in the form of the limestone, coquina, and beach rock reefs, which are found along a good part of the S. E. coast as well as part of the lower Gulf coast. It is a known fact that deterioration of some of the offshore reefs had caused increased erosion (e.g. at Jupiter Island, Fla., Atlantic Coast and on Casey Key, Fla., lower Gulf Coast). Model experiments carried out at the University of Florida in 1965 demonstrated the ability of the sill (reef) to make a step in the bottom but also revealed the scour problem, particularely inside the wall. It is, however, quite evident that the offshore bulkhead or training wall has had a beneficial effect on the profile. The result, needless to say, is quantitative, but compared to field experience it indicates the trend correctly.

A very special type of "offshore training wall" has been built at Durban, South Africa. It consists (1972) of an almost 3 km (2 miles) long offshore deposit of 5 million m^3 of sand placed in 1965-1972 on 17-18 m depth, crown elevation at approximately 7,5 m below M.S.L., maximum waves ab. 6 meters (ref. 53 brought up to date by private comm. with Mr. J.A. Zwamborn). This "wall" or breakwater has so far been remarkably stable although it fluctuates slightly, the upper developing gentler slope during storms and steeper slopes during fair weather (swell) conditions. Losses have been small and the breakwater has caused considerable decrease of wave action during storms, benefiting the beaches.

Considering the coastal geomorphological side of the problem, nature has established large pocket beaches (Plate 4, Fig. 18). Improvements of natural conditions may be undertaken taking advantage of natural headlands and extending them by breakwater - additions. Such pocket beaches have been established at several places, e.g. on the Venezuelan shores at Los Caracos and at the Sheraton-Macuto Hotel. Pockets may also be established by stockpiling of sand on the beach at intervals. This will undoubtedly cause some (temporary) slow down of longshore drift creating a (temporary) leeside erosion problem. More material will probably be lost offshore however.

STRUCTURES

With respect to structures - whether groins, offshore breakwaters or seawalls including revetments - it may be expected that prefab. elements will take over to a still increasing extent. The shore-parallel structures will be in the lead because they fulfill requirements of consideration to adjoining property, recreational needs and aesthetics better than shore-perpendicular structures. Where the latter are built they are most likely needed for special "interlocking purposes" as "pocket" or "perched" beaches. Most structures will probably either be mass-produced in elements as large as practical or needed - or mass-produced in sheets of various materials easy to handle and place. This trend is already evident. A sloping sea wall e.g. may be split up in the following units: prefab. toe protection, prefab. mattress, prefab. armor, prefab. wave screen and prefab. overslash protection, totalling five "units". Groins may be made up of prefab. stem elements + possibly a T-head which could also be of prefab. elements.

Bypassing of material - Bypassing of littoral drift at tidal and other entrances cannot be considered "artificial nourishment". It is a re-establishment of natural processes which were disturbed due to man's adverse interference. This may be accomplished by "bypassing plants" or by "bypassing arrangements" (59, 60, 63, 65, 68).

Table 14. Sand By-passing Status in the United States

Location	By-passing arrangement	Status, 1970-1971
Bakers Haulover, Fla.	Bay shoal dredging	Permanent transfer from bay shoal trap suggested
Boca Raton, Fla.	Trap in entrance	Transfer from trap behind updrift jetty connected breakwater suggested
Canaveral Harbor, Fla.	Dredging of channel	Fixed plant to be constructed(?)
Channel Islands Harbor, Calif.	Trap behind breakwater	Operational
East Pass, Fla.	Depressed weir and trap	Weir jetty completed
Fire Island, L.I., N.Y.	Transfer from bay shoal	Has been studied/model study on trap arrangement
Ft. Pierce, Fla.	Transfer from bay shoals	Has been studied/suggested
Hillsboro, Fla.	Depressed weir and trap	In operation since 1952
Houston, Corpus Christi, Tex.	Bay and ocean shoal dredging	Sidecasting in operation
Jupiter, Fla.	Transfer from bay shoal	Depressed weir and trap/proposed
Masonboro, N.C.	Depressed weir and trap	Operation 3 years
Moriches Inlet, L.I., N.Y.	Fixed plant proposed	By-pass of jetties to be extended authorized
Pass, Fla.	Ocean shoal dredging	Occasional transfer from ocean shoals
Newport, Calif.	Undetermined or being studied	Recirculation by trap at lower end of ½-mile reach being studied
Ocean Beach, Calif.	Trap inside updrift jetty	By-pass from trap inside
Palm Beach, Fla.	Fixed plant	Revision planned
Perdido Pass, Ala.	Dredging of channel	Weir jetty completed
Ponce DeLeon, Fla.	Depressed weir and trap	Almost completed
Port Everglades, Fla.	Ocean shoal dredging	Transfer from shoals in ocean and entrance suggested (model)
Port Hueneme, Calif.	Trap behind updrift jetty	Transfer from trap behind updrift breakwater
St. Lucie, Fla.	Depressed weir and trap	Construction recommended
Santa Barbara, Calif.	Transfer from shoal inside updrift breakwater	Extension of west jetty, construction of east jetty and detached breakwater authorized
Sebastian, Fla.	Bay shoal dredging	Permanent transfer from bay shoal trap suggested
Shinnecock, L.I., N.Y.	Undetermined or being studied	By-pass of jetties to be extended authorized
S. Lake Worth, Fla.	Fixed plant	New jetties and pump in 1968
Twin Lakes Harbor, Santa Cruz, Calif.	Fixed plant	Operational 1972
Virginia Beach, Va. (Rudee Inlet)	Fixed plant	Revision planned; being studied

Table 14 shows the status of by-passing procedures in the United States 1970-1971. It may be observed that the flexible arrangements: dredging from traps behind depressed weirs or detached breakwaters or other traps now are in the lead compared to fixed or movable plants on jetties or trestles. Major movable plant installations are found in Durban, South Africa (stopped in 1953) and at Paradip, State of Orissa, Bay of Bengal, India. A small movable plant mounted on a trestle which may be closed or opened for passage of drift by "shutters" or "needles" is found at Nagapatam, State of Madras, India. The jetty is left open during the monsoon period.

Plate 10, Fig. 55, shows schematically various by-passing plants and arrangements. Hydraulic "lift procedures" (63) are being considered in a few places (U.S.A., India, Denmark).

DATA NEEDED FOR DESIGN

The data needed for design, needless to say, depends upon what one intends to design. If you are "scientific" you may make up a menu-card with 57 courses or so and start eating the appetizers without accomplishing any work of actual improvements. If you are "over-practical" and "over-experienced" you may wind up with a quick judgement with accompanying 50% change of failure. Table 15 summarizes what, in the opinion of the author, is necessary to secure information needed for a sound evaluation and design. With respect to artificial nourishment reference is made to the preceding section.

Table 15. Basic Data Needed for Design of Coastal Protection

Structures	Beaches
Adequate tide data incl. data on storm tides (statistically and hindcasted).	
Adequate wave data incl. data on extreme storms (statistically and hindcasted).	
Current data to the extent needed for the particular location. Longshore and transversal currents are related to tides, winds, bottom topography, discharges from rivers, tidal inlets etc.	
Profile data incl. long range and short range (seasonal) movements of profiles and shorelines normally up to at least 30 ft depth. Knowledge about undulations of shoreline and changes in bar geometry.	Profile data incl. long range and short range (seasonal) movements of profiles and shorelines normally up to at least 30 ft depth. Knowledge about undulations of shoreline and changes in bar geometry.
Grain size analyses of sand from beach and nearshore offshore bottom.	Grain size analyses of beach and offshore bottom extending to min. depth of seasonal and/or long range fluctuations. Seasonal fluctuations of grain characteristics. Detailed investigations of borrow pit materials based on core sampling. Tracing preferably by fluorescent tracers advantageous for evaluation of stability and future maintenance.

Some may claim that the experienced designer needs less data than the in-experienced. Practice, however, often tends to demonstrate the opposite because the experienced person is more aware of the difficulties and he is therefore more careful with his planning. It is also experience that advocators of patented "super cerked" devices, e.g. within the branch of permeable groins, usually need little or no data at all. The designer's "experience" is based on "faith" or on "just business". Consequently they can also be proud of having the absolute record of absolute failures.

PLANNING. ADMINISTRATIVE ASPECTS

With respect to planning of coastal protective measures even the best "philosophy" and professional (technical) approach will not work unless the administrative and political aspects are planned as well. In the United States national (federal), state, county or other public body if requested may grant specific favours in accordance with rules and laws but support by local groups is very essential and may be gained through confidence and detailed exposure of the program. The responsibility of developing plans that reconcile conflicting demands is the responsibility of all levels of government. Although planning criteria must be orientated towards the multiple-use concept based on local desires, it must also consider State and National needs. The State (province or county) should have - and some actually have - a single agency with the administrative and technical ability, financial resources, and enforcement authority to regulate coastal protective measures and provide cooperative support to help blend local interests with the State interests. It can also supply some expertise and files of basic data. And not least it should support with funds.

With respect to public administrative control with coastal protective structures it should al ys be an absolute requirement that what do ensure a positive result

for yourself should not be allowed to have any adverse effect upon property belonging to others. It is generally accepted that one should not deprive anyone of the water which flows to his property by which he upholds his living e.g. by farming or by industry. Neither should anyone cut roads or accesses over somebody else's property. Why should it then ever be permitted to rob your neighbour of his shore property by increasing nature's forces on his shore or by decreasing the flow of beach materials to his property? These are essential for maintaining it - just as important as water is for farming the land. Neither should anybody be permitted to extend the public access road constituted by the public ocean on the cost of any neighbour or shore property owner. It is peculiar that for a long period of time the most simple analogies in public administration were not recognized. The reason was probably twofold: first the mechanics of the matter were not understood and, next, public agencies were often responsible for the errors made either by direct sponsorship of the ill-advised structure or by permits granted to erect the structures. The consequence was that "face savings" and other "bureaucratic reasons" often became more important than recognition of facts. The peculiar situation therefore developed that it was the courts who were forced to look into such problems because a few individuals were hardheaded and wealthy enough not to bend their neck for certain shortcomings of public administration.
One of the major problems for progress in coastal protection, however, is that funds for planning have been difficult to obtain - or very late in coming. Most coastal states in the U.S.A. still have not provided adequate agencies and funding and the lack of coordination and planning of coastal protective measures has gradually become a national problem. The Federal Government has taken an active - but still insufficiently funded - interest. The most difficult task, however, seems to lie on the local government and group level because the democratic system does not advocate the kind of discipline which is a necessity in all warfare including the tough fight against the sea. Practices in Europe and in the United States differ to some extent due to differences in the political and administrative pattern. In Europe, particularly in the low countries and in the countries rimming the North Sea, problems are often fully national. Contributions by local governments or groups are minor or non existing but they may handle the problems partly or wholy on less exposed shores.

Ten demands in Coastal Protection are listed in Table 14. They are as strict and demanding as those listed in Deuteronomy. But if we do not obey them we may soon need a number of "Adams Bridges" for escape - because nothing can stop the forces by tides, waves and currents. King Canute of Denmark and England placed his royal throne on the beach but the waves washed over his feet. He withdrew. Flexible defence costs less and mostly it is the most successful and it does not prohibit stand-by positions when needed.

Table 14. The Ten Demands for Coastal Protection

1) Thou shalt love thy shore and beach
2) Thou shalt protect it gainst the evils of erosion
3) Thou shalt protect it wisely yea, verily and work with nature
4) Thou shalt avoid that nature turns its full forces gainst ye
5) Thou shalt plan carefully in thy own interest and in the interest of thine neighbour
6) Thou shalt love thy neighbour's beach as thou lovest thy own beach
7) Thou shalt not steal thy neighbour's property, neither shalt thou cause damage to his property by thy own protection
8) Thou shalt do thy planning in cooperation with thy neighbour and he shalt do it in cooperation with his neighbour and thus forth and thus forth. So be it
9) Thou shalt maintain what thou has built up
10) Thou shalt show forgiveness for the sins of the past and cover them up in sand

Acknowledgement - The author wants to express his appreciation for the assistance offered him in preparation of the historic sections of this paper by colleagues in the Netherlands, Mr. J.F. Agema, Chief Engineer, Rijkswaterstaat, Mr. T. Edelman, Chief, Coastal Research Department, Rijkswaterstaat and Mr. J. Battjes, Research Engineer, Delft University of Technology and in England by Mr. D.L. Newman, Research Engineer, the Hydraulic Experiment Station in Wallingford.

PLATE 1

Fig. 1. "Golden Hoop" in the Netherlands. Currentbreaker of later date in front (van Veen, 1962)

Fig. 2. Cross section of ancient dike, Netherlands. ("Antiquity and Survival", [illegible])

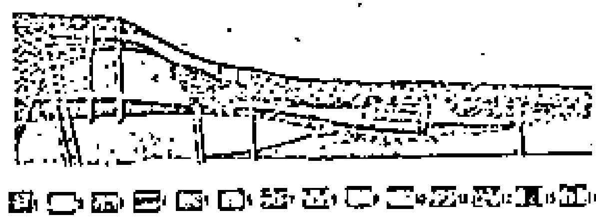


Fig. 3. Cross section of ancient dike with pile support. ("Antiquity and Survival", [illegible])

Fig. 4. Polder boys repairing dike with willow, Netherlands (van Veen, 1962)

Fig. 5. Willow mattress, Netherlands (van Veen, 1962)

PLATE 2

Fig. 6. Breakthrough at [illegible], Netherlands, Febr., 1953 (van Veen, 1962)

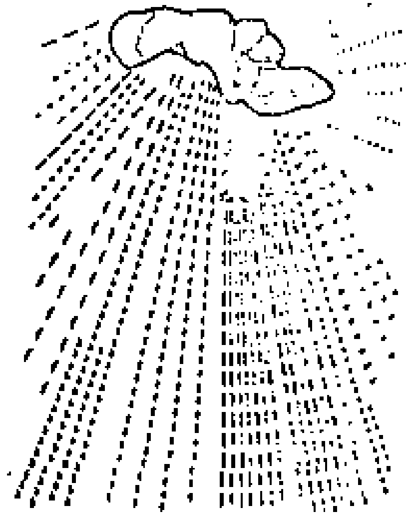


Fig. 7. [illegible] parade at closure of breaches in the dikes of [illegible] (van Veen, 1962)

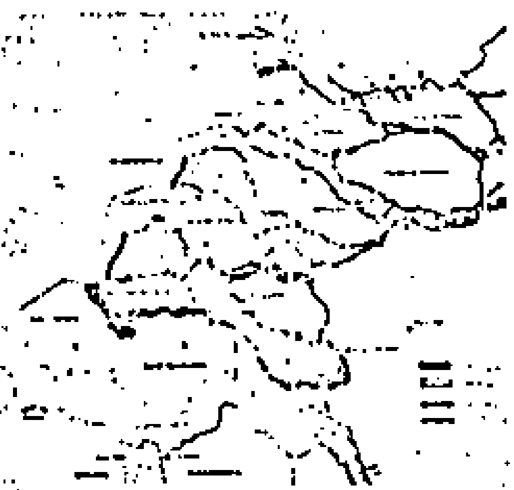


Fig. 8. The Delta Plan. (Rijkswaterstaat, 1976)

Fig. 10. Placement of [illegible] of synthetic materials. (Deltawerken no. [illegible], 197[illegible])

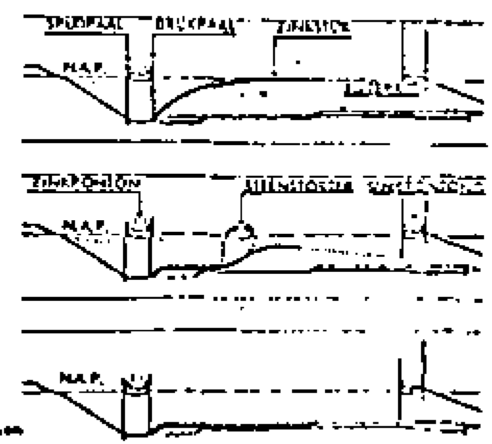


Fig. 9. Towing of willow mattress for bottom protection (van Veen, 1962)

PLATE 3

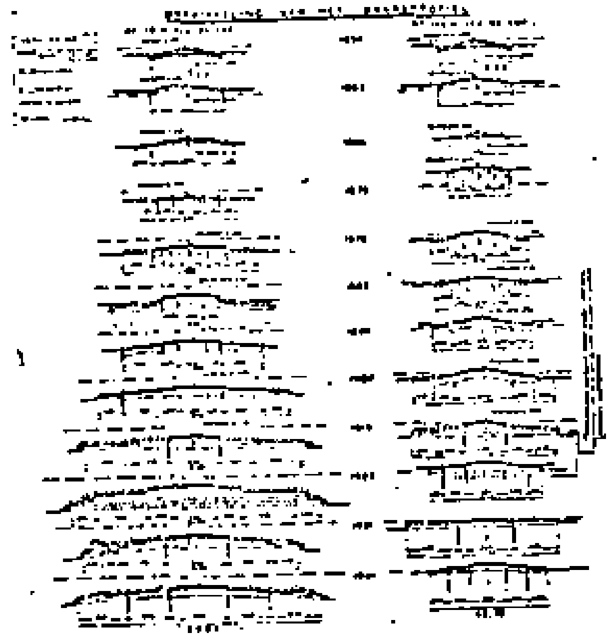


Fig. 11. Development of groins in Holland [illegible]. (Visser, [illegible])

Fig. 12. [illegible]

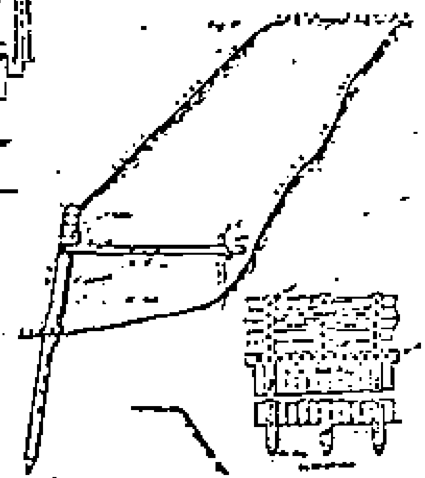


Fig. 13. [illegible] sea wall. (Mins. Inst. Civ. Engrs., 1877-78)

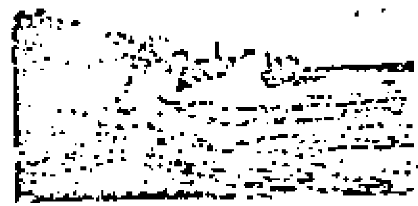


Fig. 14 b. [illegible] Sea Wall, Bridlington, under wave action

Fig. 14 a. [illegible] Sea Wall, Bridlington ([illegible], 193[illegible])

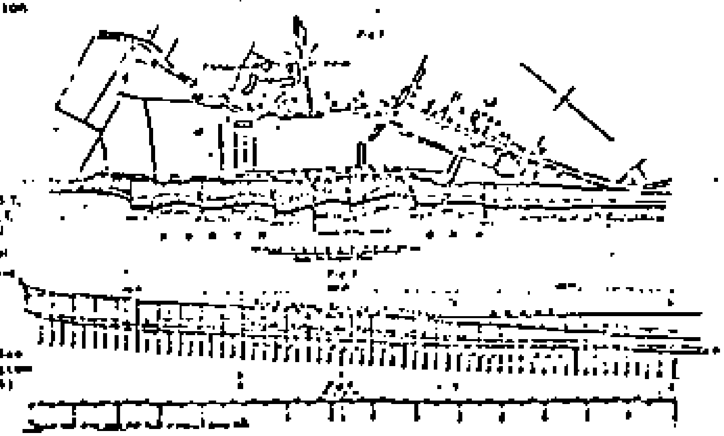


Fig. 15. [illegible] Groins (Mins. Inst. Civ. Engrs. 1877-78)

PLATE 4

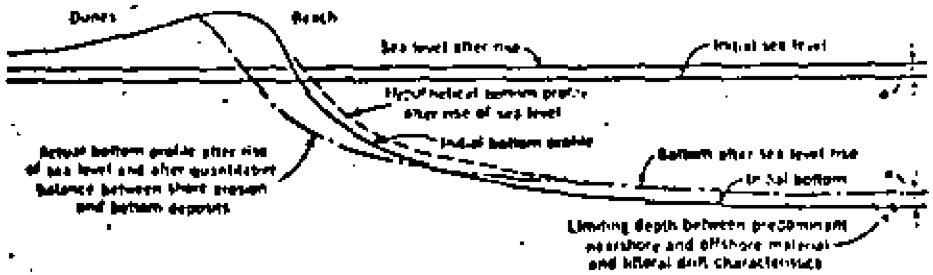


Fig. 16. The influence of sea level rise on erosion (Bruun, 1962)

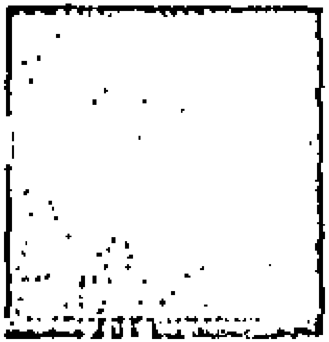


Fig. 17. Dyrhólaey, headland and natural breakwater on the Icelandic south coast

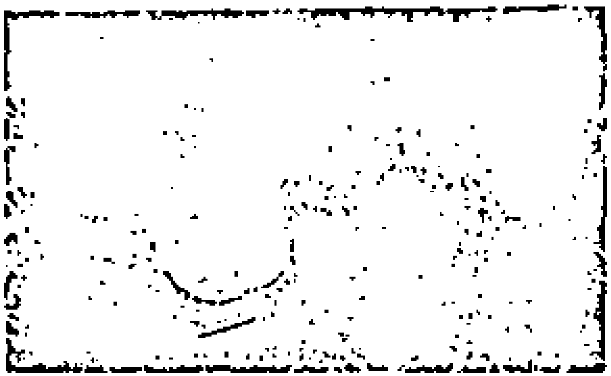


Fig. 18. Rock outcroppings make natural sea walls and form pocket beach in Puerto Rico

Fig. 19. [illegible]

Fig. 20. River pouring material into the sea nourishing beach and offshore profiles on the Icelandic south coast

PLATE 5

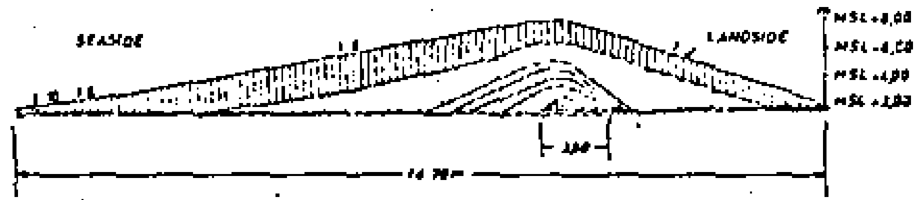


Fig. 21. Comparison of older with modern dike profiles (Kramer, 1971)

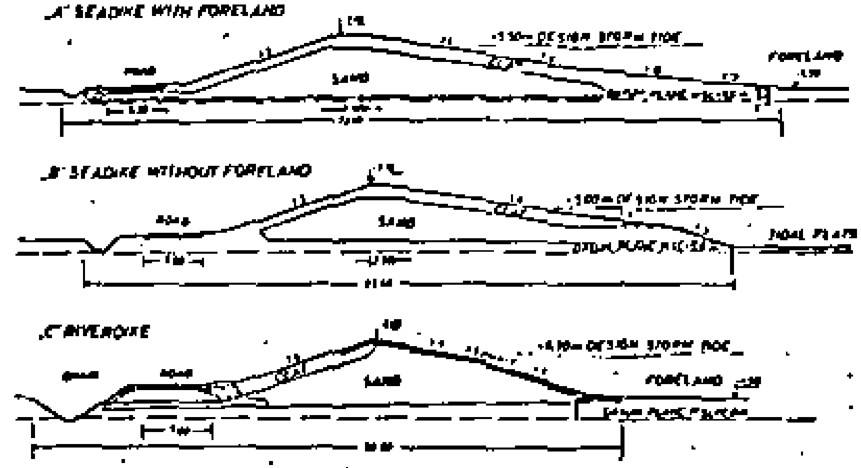


Fig. 22. Cross sections of dikes with sand core and clay or asphalt cover (Kramer, 1971)

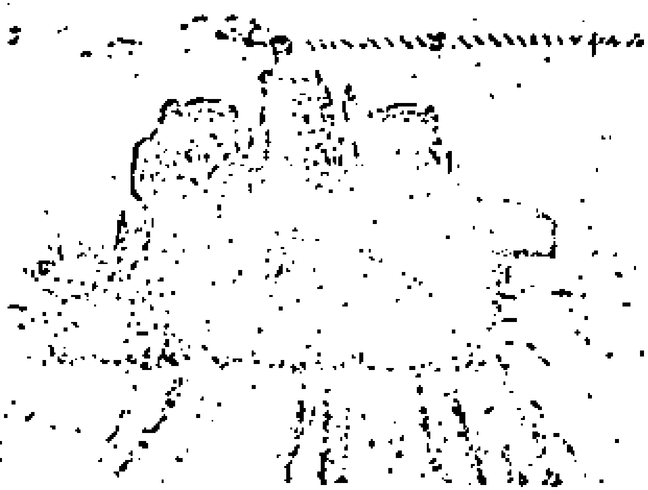


Fig. 24. Dune building by sand fences (UPS, USA)

PLATE 6

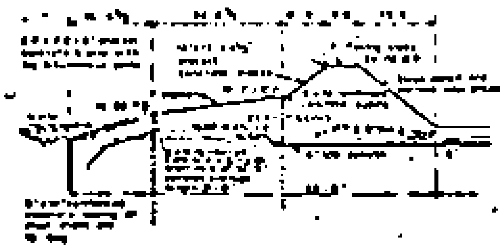


Fig. 25. [illegible] Level Sea wall (Thorn, 1960)

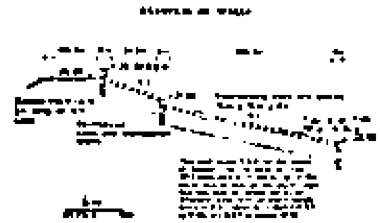


Fig. 26. Holland Sea wall (Thorn, 1960)

Fig. 27. Stone pitching revetment and stone pitching piers back groin (Rijkswaterstaat)

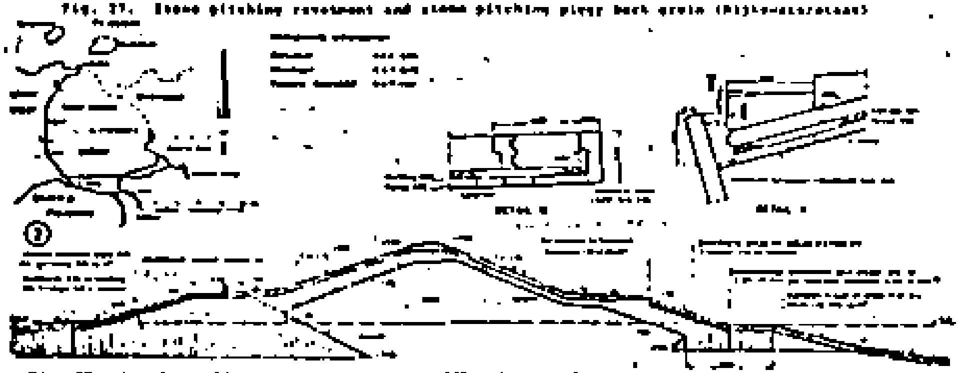


Fig. 28. [illegible] revetment on dike (Rijkswaterstaat)
Legend: [illegible] (basalt prisms), [illegible] (gravel layer), [illegible] (layer of brick), [illegible] (straw mat), [illegible] (willow mattress), [illegible] (rock), beton (concrete), klei (clay dug from canals and ditches)

Fig. 29. Revetment at [illegible] (Rijkswaterstaat)

Fig. 30. Sea Carpet at [illegible], Netherlands ([illegible])

Fig. 31 a and b. Concrete blocks with friction arrangements (Rijkswaterstaat)

PLATE 7

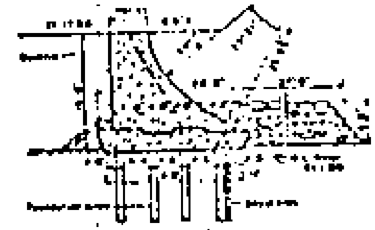


Fig. 32. The Galveston Wall, Texas (USCE = U.S. Army Corps of Engineers)

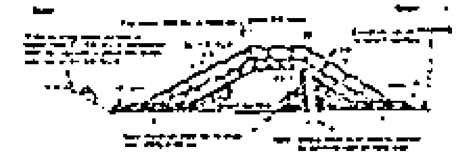


Fig. 33. The Fernandina Wall, Florida (USCE)

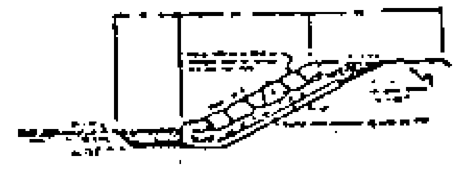
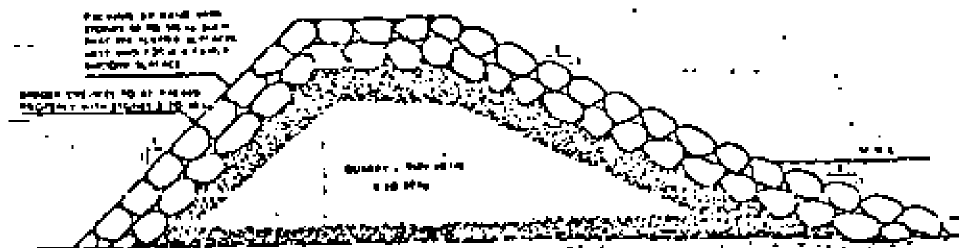


Fig. 34. The [illegible] Wall, Va. (USCE)

Fig. 35. Indian Sea Wall (Kerala State)



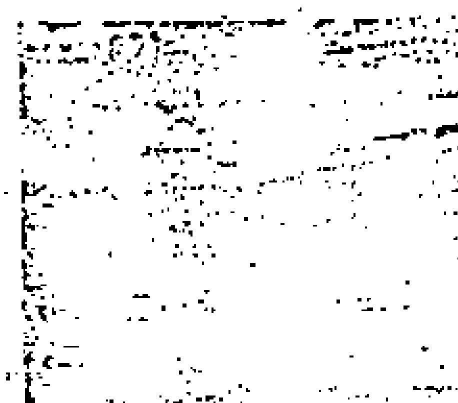


Fig. 36. Vertical wall on Jupiter Island under heavy wave attack, 1962 (Bruun and Manohar, 1963)

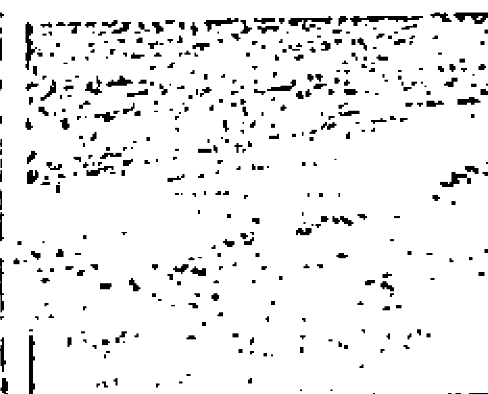


Fig. 37. Interlocking block revetment on Jupiter Island under heavy wave attack (Bruun and Manohar, 1963)

PLATE 8

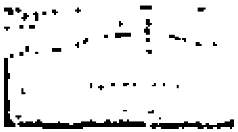


Fig. 38. Dutch [illegible] pitching groin at [illegible]

Fig. 39. Dutch group of groins, [illegible]

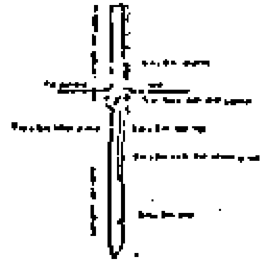


Fig. 40 a and b. British adjustable king pile groin [illegible]

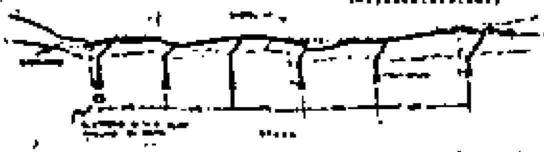
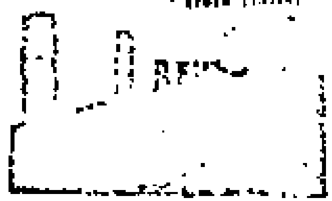


Fig. 41. Group of groins with slanting land ends on the [illegible]



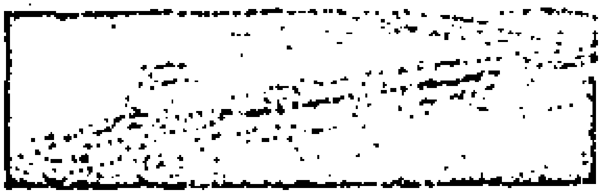


Fig. 42. Groin built of [illegible] concrete blocks in the crown and [illegible] granite in the slopes. [illegible]

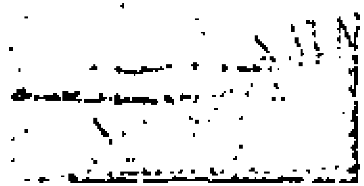


Fig. 43. Adjustable groin at [illegible] Palm Beach [illegible]

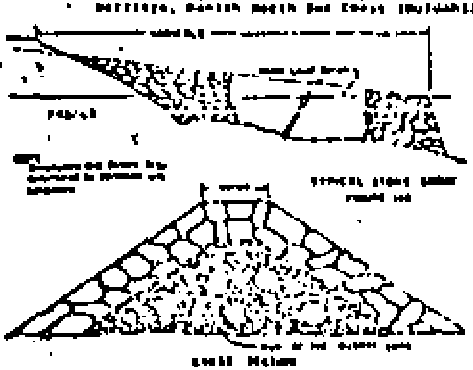


Fig. 44. American rock groin [illegible]

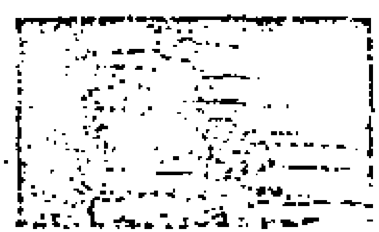


Fig. 45. Groins, mostly steel sheet piling, at Miami Beach, Florida

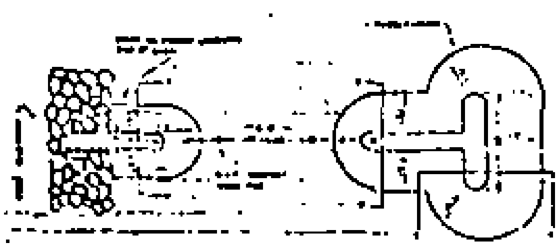


Fig. 46. Concrete design for Deerfield Beach, Fla. Adjustable king pile stem, [illegible]

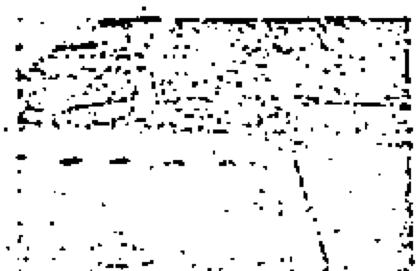


Fig. 47. Training at Deerfield Beach, Fla. [illegible]

PLATE 9

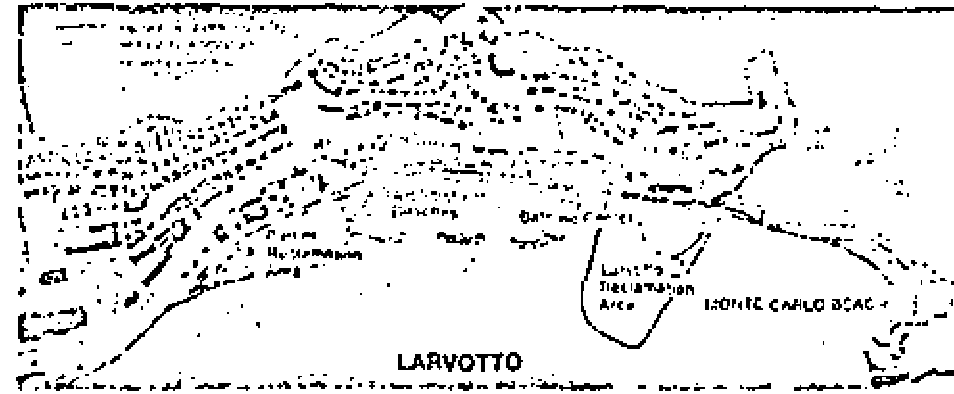


Fig. 48. Offshore breakwaters form pocket beaches, at Monte Carlo (Terra, Holland, No. 1, 1973)

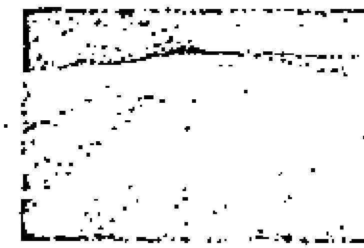
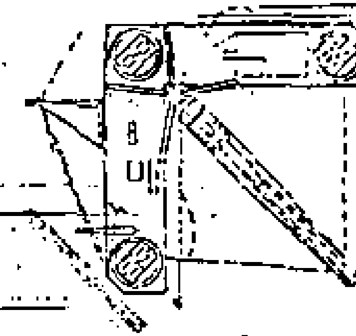


Fig. 49 a and b. Artificial nourishment from land sources. [illegible] on 14,000 ft of shore, [illegible] Head Island, S.C.



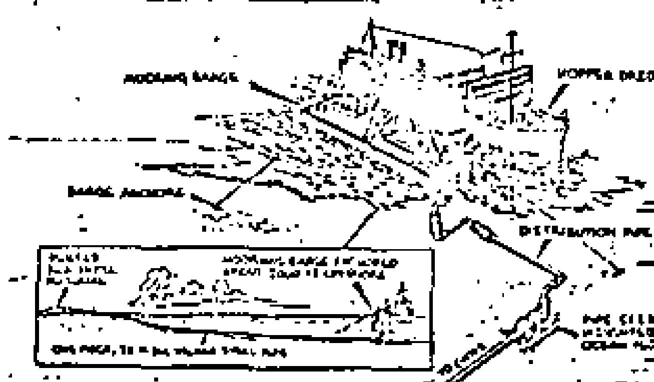


Fig. 50. Artificial nourishment from offshore by hopper dredge. [illegible]

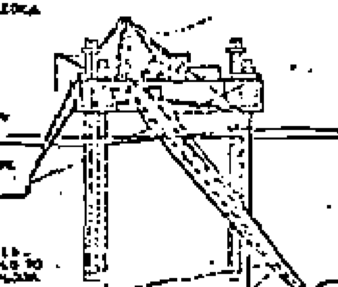


Fig. 51. Dutch platform dredge [illegible] Dredging and Construction, Jan. 1972)

PLATE 10

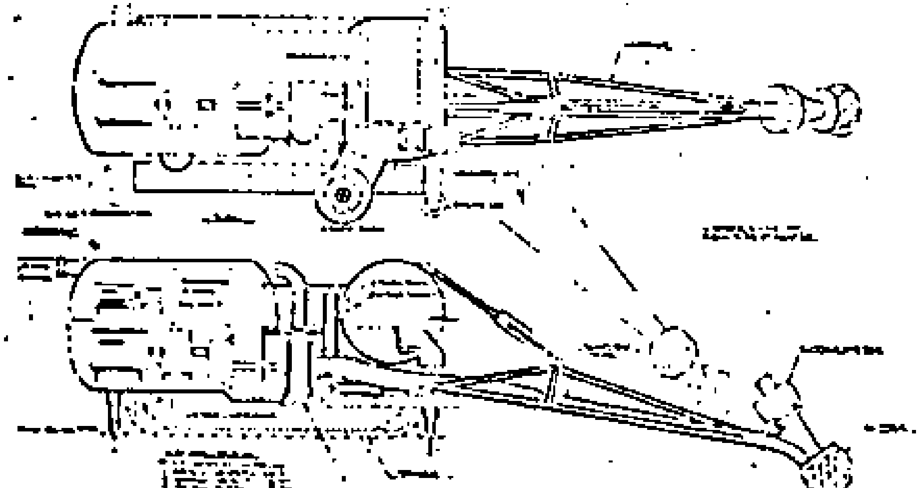


Fig. 51. [illegible] Dredge ([illegible], 1973)

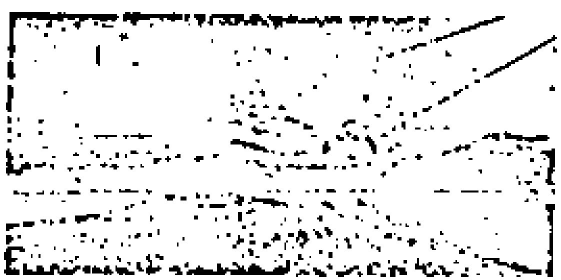


Fig. 52. Artificial nourishment by dragscraper on Jupiter Island, Fla. (Brown, 1967)

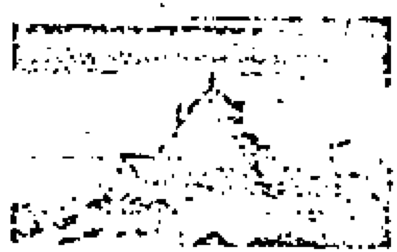


Fig. 54. Bulldozer operation at [illegible] Beach, Fla., following the [illegible] 1962 storm (Brown and [illegible] 1963)

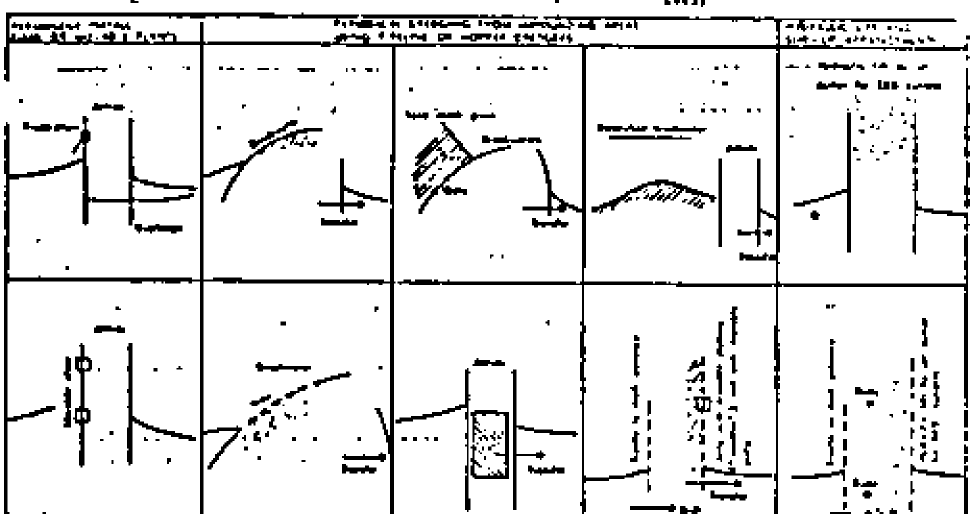


Fig. 53. Various bypassing arrangements (schematic)

NOTES ON DUNES AND DIKES

1 The dune is a natural dike which was created by wind-blown sand and was possibly vegetated by nature. Man's counterpart to the natural dune is the artificial dune or dike. The difference between dune and dike lies solely in the fact that the dune usually has land higher than sea level behind it, while the dike may protect low lying land which would otherwise be flooded.

Dunes are always built of sand, using mechanical equipment like draglines, scrapers, hydraulic pumps etc. The experience on the North Sea Coast is that an outer slope of 1 in 7 and an inner slope of 1 in 3 is practical (25). The width and elevation of the crown depends upon the actual exposure and upon the expected combined tide and wave elevations. Dunes, however, may also be erected with the assistance of wind and sand fences (28, 30). See Plate 5, Fig. 24.

They may be protected by dune vegetaion of various kinds (26, 27, 31, 32, 33). Most common are species of Armophila - (arenaria or breviligulata, the latter called American Beach Grass in the U.S.A.). Planting machines have been developed. See Plate 5, Fig. 23 (29) and considerable research has been undertaken on fertilization etc. (32).

2 The dike is by its nature a more solid design because its tasks and obligations are more severe. The modern dike is a result of almost 1.000 years development. Plate 5, Fig. 21 (42) is a comparison of older and modern dike profiles on the German part of Nord Friesland. The oldest dike built about 1.000 A.D. had a width of 3.80 m and steep slopes. The 1962 dike was higher, had gentler slopes and a width of about 70 m. In some vulnerable areas "sea dykes" and "withdrawn dykes" have been built parallel to each other to provide safety against breakthroughs of the sea dike (25).

Dike design and construction has developed in various directions in accordance with actual needs. Plate 5, Fig. 22 shows various cross sections all fulfilling specific requirements. Plate 6 shows examples of hard pavements described in a great number of publications (11, 34, 38, 40, 42).

3 A new type of dike protection consists of various kinds of synthetic materials incl. polypropylene and nylon fabrics used as replacement for willow and other types of mattresses which are much heavier and difficult to handle. One of the best known is the Dutch-German "Sea Carpet". Plate 6, Fig. 30 demonstrates an application in reclamation work at Europoort, Netherlands. This carpet is a combination of natural reed and woven polypropylene fabric which has a tensile strength of 2 to 10 metric tons per meter. It is claimed that it is unaffected by wet conditions and is very durable, remaining stable against chemical and bacteriological influences. An additive incorporated into the polypropylene gives it high resistance to ultra-violet rays. Its specific gravity of 0.9 enables it to float, while its lasting filtering properties permit the water, but not the sand, to pass through. The reeds assist in the filtering action of the fabric and increase bouyancy enabling it to be towed to the site and protect the fabric against damage from stone used to sink the carpet and keep it in position when it is used as a mattress. Not all synthetic fabrics or impermeable sheets offer adequate protection against ultra-violet light and generally speaking experiences are best when the sheets are not subjected to light but covered with other materials.

For further information on various commercially available brands in the United States and in Europe the reader is referred to articles in Proc. of the recent Coastal Engineering Conferences (Japan, 1966, London, 1968, Washington, 1970) and to commercially available catalogues.

NOTES ON SEA WALLS

1 Wave run-up - An important design parameter for sea walls and revetment is the wave run-up which depends upon spectral characteristics. According to Batties(35) explicit expressions for the run-up on a smooth slope are obtained for waves of which the heights and periods squared have a bivariate Raleigh distribution. For further information the reader is referred to refs. (11, 35 and 46) and to several papers on this topic in the Proc. from the 11th Conf. on Coastal Engineering, London, 1968, part 2. Furthermore the Proc. from the 10th, 11th and the 12th Conf. on Coastal Engineering include a great number of papers on wave forces on all kinds of coastal structures, fixed as well as floating.

2 The scour problem in front of sea walls is important and has to be considered. Wave height is the most significant variable to the depth of scour (39 and 41). Other important parameters are the position of the wall in the profile, the beach slope, the wave length, characteristics of beach material (41) and longshore current velocities. As the beach slope flattens, scour decreases. Scour decreases as the angle of inclination of the wall decreases, which indicates that scour decreases with decrease of reflection. Each case has to be considered separately. Model studies will be able to give qualitative information on the relative magnitude of scour. A 10 to 20 ft scour (toe) protection is usually necessary.

3 Plate 6 and 7 - Ordinary design criteria for a revetment call for sufficient weight of blocks including interlocking effect to withstand the combined effect of hydrodynamic uplift pressures due to wave breaking, downrush and hydrostatic pressure which both cause uplift. Normally a filter layer is placed partly to make an even slope and partly to drain water, which will inevitably penetrate through the joints between the blocks. This needless to say, requires a proper "filter ratio" between block layer and filter material. If the filter material is too small it may disappear out through the joints of the blocks and if it is too large this may increase hydrodynamic and hydrostatic pressures with adverse effects on stability. Drain holes may then replace the space between blocks.

Slope should not be steeper than the core material has a stable slope in fully saturated condition apart perhaps from the uppermost less exposed section of the revetment when - as proved by experiments - the weight of blocks is useful for squeezing blocks in the lower part of the slope together. Fill material must be well compacted to minimize settlings. Examples of failures e.g. in Holland and Florida can often be traced back to inadequate compaction e.g. caused by negligence during construction. Revetments of blocks are not used in Holland where wave action exceeds 10 ft. In Florida the limit may be set a little lower due to the predominance of sand fill and less experience in building such walls, which require good workmanship and in addition on exposed shores a protective apron and/or beach in front (38, 40).

Research on revetments for reservoirs carried out in the U.S.S.R. (43), has proven that the stability of a revetment may be improved by reducing thickness of or eliminating the filter layer entirely. The flexibility of the armor layer and the porosity of the underlying soil are important parameters.

Regarding design principles for rubble mound revetments, reference is made to a paper by Johannesson and Bruun (51).

4 Plate 7, Fig. 35 - The "developing countries" have sometimes been wiser than the "developed countries". As an example the Indian standard stone-pitched rock mound used extensively for sea walls particularly at many places along the SW coast (State of Kerala) is an excellent example of long time experience adopted to "what we have and what we can do with available tools". It is startling to see the similarity in several respects between old Dutch and old Indian experience.

5 Plate 5 and 6: Asphalt and bitumous products have been used considerably for breakwaters and revetments under small to moderate wave action (34). In several cases maintenance has presented some problems and in some respects the application of asphalt is still in an experimental stage.

NOTES ON GROINS

1 Layout and geometry - The general experience is that groins should be built perpendicular to shore although some laboratory experiments may have revealed that efficiency increases a little by turning them downdrift e.g. to 70 degrees in case waves approach the beach under 70 degrees. Length/space ratio may vary according to littoral drift capacity and exposure from 1:1 to 1:4. Streamlining downdrift is often advantageous (6 to 10 degrees tapering off).

2 The design of a groin protection also depends upon the beach material as well as the material available for construction. Unless beach material is coarse (pebble up to shingle) a streamlined design is preferable and the optimal design undoubtedly is the one which is streamlined and energy-absorbing at the same time. The Dutch groins (Plate 3 and 8) with their wide stone pitching fulfil such requirements. The vertical face sheet pile or king pile groins do not, but their function may be greatly improved by adding roughness on their sides, e.g. in the form of rubble mounds.

One rule should always be obeyed: If groins are nonadjustable they should be low or the beach will be nourished continuously. If groins are high they should be of the adjustable type and be kept adjusted to the actual beach profile, that means they largely should follow its movement. Their function is to decrease beach fluctuations, not to hinder them.

3 The efficiency of and distance between groins may be increased by adding a shore-parallel breakwater at the extreme making a T or L-shaped geometry (Plate 8, Figs. 46 and 47). This appears to be a definite advantage on steep shores but costs increase. Scour may develop at the breakwater ends if groins extend to depth when wave action is more violent.

4 Sand filled tubes of synthetic materials have been tested in Denmark, Holland and Germany. Diameter may be from 2' to 6'. Generally speaking, experience has been rather satisfactory but the tubes are exposed to sabotage and have to be protected against ultra-violet light by special treatment.

5 Please do not use groins unless they are naturally or artificially supplied with adequate quantities of sand fill.

NOTES ON OFFSHORE BREAKWATERS

1 Some offshore breakwaters are shore-connected and some are not. Plate 9, Fig. 48 from Monaco is an example of the former. Many Southern European (Italian and Spanish) offshore breakwaters are not shore-connected. Refs. 49, 50, 53 and 54 advise on design principles and practical design and construction. Ref. 51 is a review of the reasons for failures of rubble mound breakwaters.

2 Submerged breakwaters - submerged sills and training walls are mentioned in the main paper. Ref. 52 is a comprehensive paper on wave mechanic aspects of the function of submerged breakwaters. Some patented devices are also on hand. They all have in common the fact that they suffer from scour problems, and they should never be placed in the breaking zone or where longshore currents are strong.

3 The submerged sand breakwater built at Durban, South Africa, mentioned in the main paper, is an interesting invention which so far has been successful. One of its main advantages is that should it fail as a breakwater its material will function as artificial nourishment. This experiment therefore advocates offshore dumping of sand material in certain cases.

4 Various laboratory and field tests have been run with a special type of shoreparallel protection, namely, artificial seaweed; but the results are inconclusive, although it appears that it may have an application mainly in unidirectional flow (C.T. Wicker, Shore and Beach, Oct. 1966).

NOTES ON ARTIFICIAL NOURISHMENT

1 Suitable sand - It should always be remembered that not all material for artificial nourishment needs to be suitable as beach material. In cases when heavy erosion has taken place, requiring large quantities for replacement, the lower layers may be "fill" which may be separated from the upper layer by a sheet of synthetic material.

2 Heavy sand - Tests have been run with heavy sand. Laboratory tests confirm the suitability but the quantities needed makes practical application highly questionable apart from enclosed areas of limited size.

GROINS

46 Reference is made to "An Annotated Bibliography" by J.M. Balsillie and R.O. Bruno, Miscellaneous Paper No. 1-72, April 1972, by the U.S. Army Corps of Engineers, Coastal Engineering Research Center with comprehensive review of all aspects pertaining to groins, their function design and the experience gained up to date. This publication was distributed at the Vancouver Conference. References are therefore left out in this paper due to space limitations.

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PRACTICAL VIEWS ON THE DESIGN AND CONSTRUCTION OF MOUND BREAKWATERS

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ABSTRACT

Bruun, P. and Kjelstrup, Sv., 1981. Practical views on the design and construction of mound breakwaters. Coastal Eng., 5: 171—192.

The design section of this paper is an abstract of Report No. 7, 1979 by the Institute of Port and Ocean Engineering, The Norwegian Institute of Technology (IPOE). It is written in continuation of an article on "Common Reasons for Breakdown of Rubble Mound Breakwaters" published in this Journal. It discusses details of breakdown, block and blocklayer hydrodynamics, how to improve resistance to damage, the development of damages and how to describe damages in practical terms. Practical design criteria, laboratory procedures and data needed for design are mentioned and finally practical construction procedures and equipment to be used to build a mound with structural characteristics as close as possible to the one tested in the laboratory.

The construction section summarizes a number of practical experiences on construction of rubble mound breakwaters with special reference to conditions in Norway, particularly its northern "top".

BREAKDOWN MECHANICS AND DEVELOPMENT

Common reasons for breakdown

The variety of reasons for breakdown of rubble mound breakwaters mentioned in the article on "Common reasons for damage or breakdown of mound breakwaters" published in this Journal (Bruun, 1979a) with reference to Fig. 1, seem to be recognized by practicians as well as researchers. It is obvious that most breakdowns have mixed — or combined — step-by-step reasons. As an example "knock-outs" and "lift-outs" may follow each other within fractions of a second or within seconds, both being active in the destruction. The first "knock" shakes the block loose, the second carries it up. It may also happen — and this would probably be the most common case — that there are many waves and/or considerable time between the "knock" which shook the block loose and the fatal "lift-out".

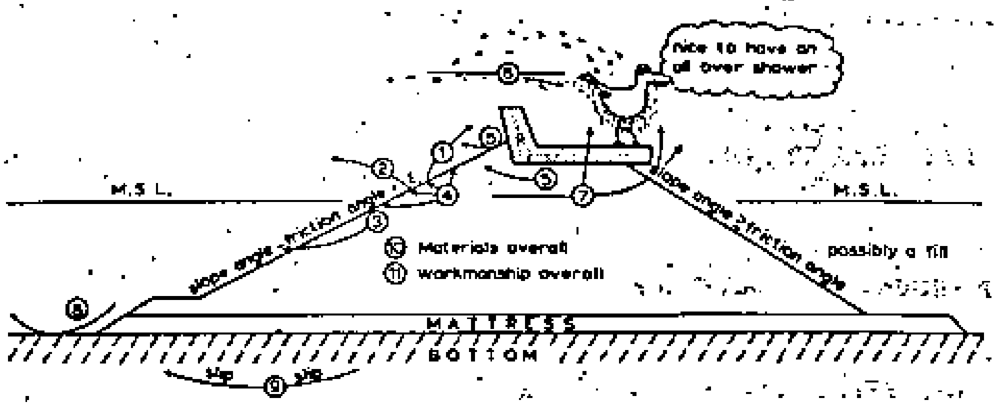

Fig. 1. Various reasons for damages to rubble mound breakwaters. Numbers refer to locations of failure.

The *development and rate of damage* depends upon structural characteristics. In any case it should be remembered that a damaged breakwater is not identical with the initial design. Most likely it has become more prone to continuation of damage even by waves of less adverse character than those which caused the initial damage, usually the loss of some upper layer armour blocks which jumped out of place exposing the second layer armour — if any. This second layer usually has blocks which are somewhat smaller than the upper armour layer. On slopes which are not too steep the displaced blocks may come to rest on the part of the slope which is below M.L.W., perhaps down to approximately one wave height. The slope may then acquire an S-form well known from old breakwaters (Bruun and Johanneson, 1976) which were "fed" with blocks mainly in the upper section (above and at M.S.L.). Example of this are the breakwaters at Plymouth and Le Havre (Fig. 2) (Priest et al., 1964). If damage to the upper section continued during the storm, or due to lack of repairs before the next storm, the breakwater may ultimately be de-

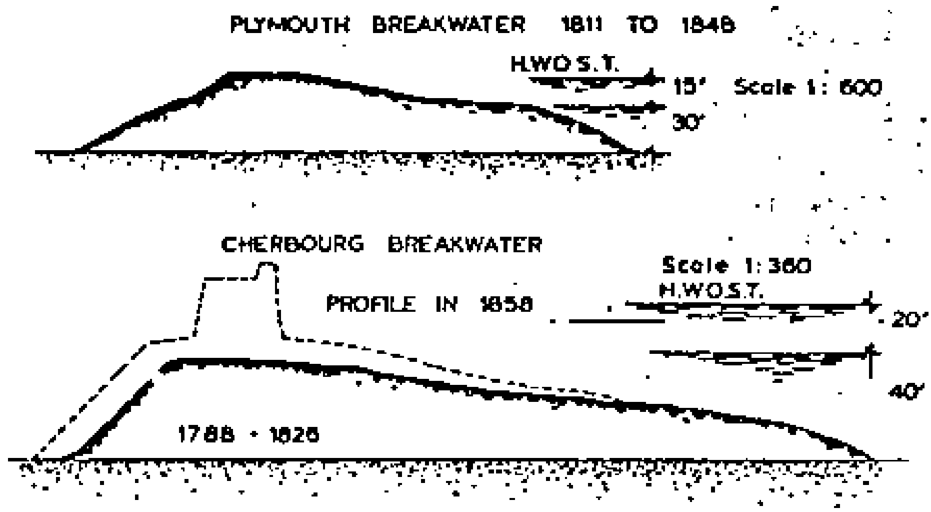

Fig. 2. Ancient breakwaters at Le Havre and Plymouth, where S-shape developed.

stroyed, leaving a mound close to M.S.L., see Figs. 2a,b. Short-term accumulation of damage may take place during one storm from hour to hour until late in the attenuation phase or until the storm has subsided entirely. Before this happens damage may wind up in fatality (Bruun, 1979a). Another situation is that damage takes place and continues from one storm until the next without any dramatic conclusion or disaster. The case of the former particularly refers to extreme events resulting in major damages which call for immediate action on repairs to prohibit a rapid and almost complete breakdown of the breakwater as a whole. This may happen anyhow, particularly with artificial "steep slope" blocks mounds. The latter case is normal when a rubble mound breakwater gradually wears out by the loss of some blocks by the departure of blocks, by settling or sinking, toe erosion or by wear of softer materials. Fortunately, this is the normal case. And it can be handled in time by proper maintenance works.

The *stability of an armour block* depends partly upon "the block itself" and partly upon "its relation" to or association with its neighbours (Bruun and Johanneson, 1976). Physically the "block itself" means its weight, volume and shape. "Its relation to its neighbours" comprises the friction between the blocks, which may consist of "skin friction" (surface to surface), interlocking or intertangling which determine the mutual capacity for sticking together and the block's "anchoring" to the sublayer as well as its "mooring" to blocks on the sides.

It is these properties which distinguish natural rocks from artificial blocks — usually mass concrete, possibly with some reinforcement. Natural rock's merit lies first of all in its weight. Its geometry is of relatively less importance.

With respect to artificial blocks the situation is different because it is possible to design a block which compensates for weight (volume) of concrete, which should be minimum to increase economy, by intertangling and interknitting (interior friction). Minimum volume of such block mounds is obtained by increase of voids (permeability). Another way of decreasing the volume of expensive materials in a mound is by increasing the slope angle. Many different kinds of artificial blocks exist having in common a high volume of voids and interknitting capabilities. But it is unfortunately true for all of them that their contact with their sublayer is inadequate and for this reason a slope of such blocks tends to break down by sliding. When slides first start, breakdown unfortunately develops fast in a "quick collapse". This focuses interest on the modes and pattern of breakdowns.

We are here at a crucial point in breakwater technology because the huge number of possibilities of placement of a certain number of blocks in fact invalidate the so-called "zero damage criterion" which obviously now becomes absurd in practical sense. The situation associated with this criterion should therefore never be understood in any other way than the situation which develops after a certain maturing of the mound by storms which occur introductorily, e.g. during the first couple of years. The inevitable damage then decreases gradually until "about stability" is established — still allowing some minor "rocking" without further or severe consequences with respect to over-

all or unit stability. This is the actual "0" damage starting situation.

Details of the hydrodynamics of wave uprush/downrush are mentioned in Report No. 7 by the IPOE (Bruun, 1979b). How drag and inertia forces compare to each other relatively in such cases depends upon block geometry and roughness in relation to flow as well as in relation to other blocks in the first and perhaps also the second armour layer (Bruun and Johanneson, 1976). In this respect one must probably distinguish between the already mentioned "skin friction" that means surface to surface friction and "surface geometry friction" which includes "friction" due to a momentum which has to be overcome (Fig.3). Hydrodynamic "lift forces" may be amplified by semi-static water pressures caused by the super-elevation of the water in the core.

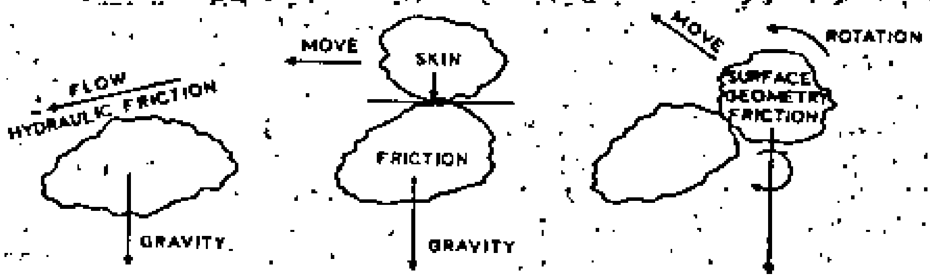

Fig.3. Various types of friction occurring in a rock mound.

The entire force development and distribution consequently becomes very complex and it is very difficult to establish general rules for their combinations. With respect to the usually patented multilegged blocks, the situation becomes even more complicated because of their relatively large surface area compared to their weight. They cause higher total drag as well as inertial forces which are both adverse to stability. Their higher permeability compared to natural blocks has the same adverse effects, as these blocks will be submerged in water which carries fast flows upbound as well as downbound. The upbound flow, which has a component downward in the mound, increases stability, the downbound flow works the opposite way. As velocities are highest in the upper part of the armour layer, lift forces together with buoyancy may "fluidize" the block layer, making the blocks float around, bumping and damaging each other. This happens most easily with long-period waves (U.S.A.C.E., 1976). In a practical sense there is therefore a limit for the permeability of a certain volume of block in relation to its weight. As forces are unequally distributed along a slope, the said limit is not the same everywhere which in turn means that blocks obviously should vary in weight or in geometry or in both along the slope as discussed later.

With respect to the multilegged patented concrete blocks, their interior friction by interknitting is by far superior to natural rock blocks which provide little or no interknitting at all. The friction between the multilegged blocks and their sublayer, however, is never satisfactory because these blocks are sitting more or less loosely on the top of the first sublayer with only relatively few "contact-points", contrary to the situation with natural rock.

And it is undoubtedly very important to have a good contact between armour and sublayer for these artificial blocks because they are always placed on as steep a slope as possible (the relatively more costly material).

When rubble mounds, including artificial blocks, are placed "at random" (pell-mell), no actual attempt is made to make them cooperate in resisting the forces to which they are subjected. They are supposed to support each other in all directions at random or incidentally by the geometry they have. In this respect the interknitting concrete blocks have an advantage compared to all other blocks even if they are only placed at random. In regard to mounds of natural rock it is very obvious that the stability of such mounds can be improved greatly — and as demonstrated repeatedly (Kidby et al., 1964; Bruun and Johanneson, 1976) — by placing some of the largest blocks as binders perpendicular to slope (Fig.4a) or by always placing the longest side ($a>b>c$) perpendicular to the slope (Fig.4b). Some of them may stick somewhat irregularly out of the "theoretical surface" but this will reduce uprush. Such placement reduces uplift as well as drag and inertia forces and friction against the sublayer is increased simultaneously due to a better "rooting". Although it costs more to place blocks in that way, practice shows that savings in volume balances increase of costs in placement.

"Rooting" of multilegged concrete armour down in the first sublayer is not done but it is common to place the first layer of Dolos blocks with one anchor stock parallel to, the other perpendicular to the slope. This may e.g. be seen at the St. Cyprian breakwater in NW Spain. The first layer of tetrapods is naturally placed with three legs resting on the slope. Placement of the next (uppermost) layer of these blocks is not uniform but attempts may often be seen in placing blocks with some mutual support or interknitting simply because the supervisor and his crane operator instinctly try to do that. There seems, however, to be no quantitative data available on the influence of such a "part-placement". On the other hand, there are many cases

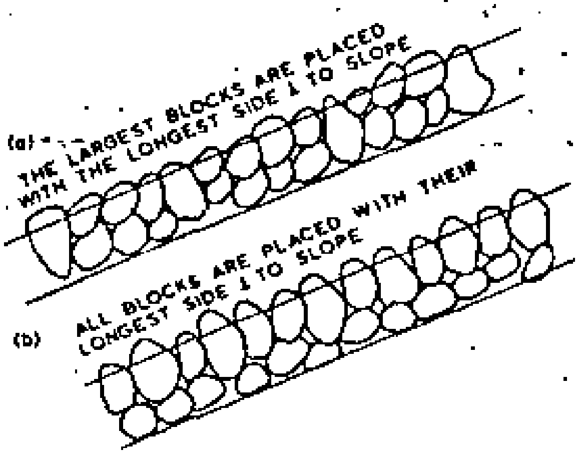

Fig. 4. Placement of blocks with the long side perpendicular to the slope to increase stability.

available of "poor-placement" seen from a hydraulic, wave mechanics, soil mechanics as well as structural mechanics point of view. This is mainly true for the first structures built of multilegged blocks. Structural drawbacks (Plough, 1979) may be solved by structural means such as reinforcements by armour steel and by the omission of elements which are too slim. The problem of a proper binder between concrete blocks, armour and first sublayer, however, still exists. How may it be solved? Theoretically the answer is simple (provide more friction or rooting). In practice the question is *how*?

In the case of a rubble mound the problem is not too difficult to handle. Binders (Fig.5) could be used. One only has to make the surface of the sublayer rougher, e.g. by letting some of the larger elements stick out before placement of the armour. But these elements must still be well rooted in the next sublayer or in the core. The importance of this is, needless to say, most evident in the case of the steep slopes — that means with artificial blocks.

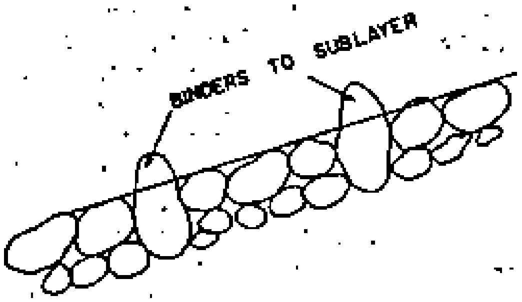

Fig.5. Binder-connection between armour and sublayer.

Another way of solving the problem could be to use sublayers of the same kind of blocks as the armour but scaled down according to similar rules as normally used for sublayers. There is, however, no guarantee that this presents the optimal solution. A block of a different, that means better, sublayer-interlocking geometry may prove to be better. Research will have to be done on this subject before the advantages of using such a system can be clarified. Summarizing the above-mentioned, one arrives at the following general rules for block stability:

(1) Blocks should provide high permeability, be rather streamlined in geometry, thereby inviting forces as small as possible, but at the same time providing substantial hydraulic friction to decrease uprush. Blocks should have a good mutual friction as well as considerable friction in relation to the first sublayer.

(2) From a hydraulic point of view, blocks should therefore be round. From a mechanics point of view they should be provided with anchor pins in all directions. The best blocks are those which "stick together" or — from research on mounds of quarried rock — those which are best "anchored down" in the next layer(s) of the mound. As explained in Bruun and Johanneson (1976), steep mounds also cause some squeezing of the single

members. This, however, may become very dangerous and may finally result in collapse if one block jumps out, as has been experienced with artificial blocks. A streamlined block-geometry cuts down breakdown forces and friction (rough irregular surfaces) and irregular (pell-mell) placement increases stability forces.

(3) With respect to the sublayer, it should consist of blocks large enough to arrest them below the armour layer. An effective "anchor system" as mentioned above, may pose problems on placement. Grouting of the upper rock layer consisting of blocks which are able to pass down through the armour layer, however, could solve the problem if such grouting can be done effectively.

With respect to damage the question is whether — apart from minor introductory "adjustment-damages" — it is wise to tolerate any damage at all. The so-called "no damage—no overtopping criterion" leaves the designer with a design which he knows only on his own dictated terms and not with a structure which may start coming apart randomly and most likely will continue doing so if the storm continues. Breakwaters which stabilized after a considerable amount of wave attacks are rare, however, but they do exist in cases where, intermittently between the storms, feeding with new blocks was undertaken. This is a common feature with Norwegian breakwaters. It is, however, not a satisfactory technical or economical procedure to let nature take over the full responsibility for the design. Prevention rather than cure is not only better in medical but also in engineering practice.

The stability of the armour layer as a whole

The duration of a severe storm is very important for the stability. Figure 6 is a schematic showing breakdown in relation to the duration of storms and

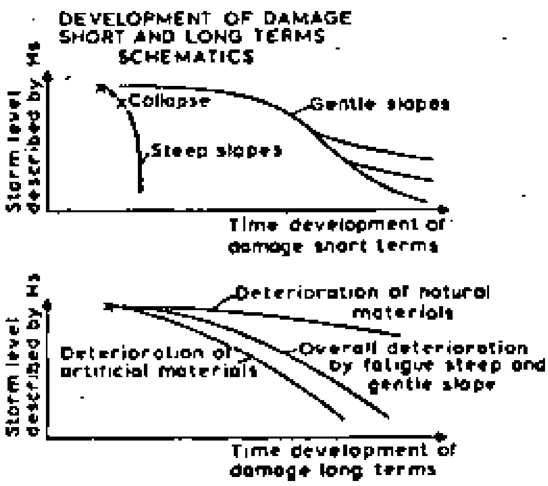

Fig. 6. Schematic description of breakdown of rubble mound in relation to duration of storms.

modes of development of damage, the ultimate result being the same. The schematics assumes that initial damage starts at the same intensity of the storm. It is obvious that the "toughest design" under otherwise equal conditions is much to be preferred to the "brittle design". Speaking about the long-term fatigue development, the development of gradual damage could also be different for the two kinds of structures and be related to very different design characteristics. Steep slopes are common with artificial (concrete) blocks where structural stability, including soundness of material, also plays an important role. Concrete elements, particularly if they are slim, may as mentioned above break relatively easily if they somehow get jammed and subjected to large moments by breaking or uprushing waves (Plough, 1979). This will hardly ever (cannot) happen with mounds of natural rock but has been experienced with some concrete blocks (Plough, 1979).

Artificial multilegged blocks unfortunately breakdown fast when damage first starts. The apparent "brittleness" of artificial blocks in wave mechanics as well as in structural respects, as visualized by Fig. 7, makes the advantages of using them smaller compared to natural blocks because they have to be designed with a larger safety factor. The N_{ZD} factors for 0 or for small initial damages therefore are not reliable — or objective — factors for comparison. The consequence of this situation is that it may be economically well justified to use rocks compared to artificial blocks up to a higher weight than justified by "commercial K_D values" and the more objective N_{ZD} values. This situation becomes even more pronounced when wave attacks are not perpendicular to the breakwater alignment but have an essential component parallel to the breakwater. This weakens the stability of artificial blocks further, as proven by experiments in several laboratories (Florida, Denmark, Norway etc.).

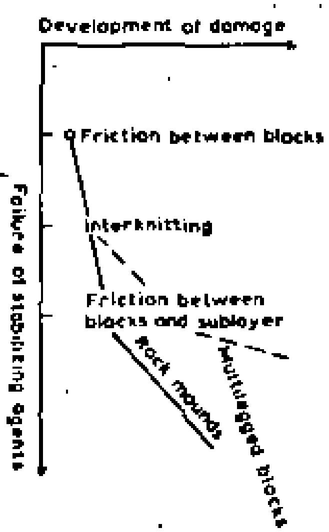

Fig. 7. Damage patterns for rock and multilegged concrete blocks.

The conclusion obviously is that natural blocks of substantial character should be used whenever available in adequate quantity, size and soundness of material. The long-term economy which also includes consideration to future maintenance is a major item in planning and optimization of any particular design of a breakwater in the future. And the older a breakwater becomes the more obvious is the importance of the materials used. Breakwaters are built to last. In some countries of low "maintenance discipline", they are allowed to collapse gradually. The uses of artificial materials, first used at Tyre in Phoenicia (present-day Lebanon), later at Ostia Rome's seaport on the Tiber and for about 100 year-old breakwaters in Europe, do not provide evidence for advantage of the use of artificial blocks if natural blocks can be found.

At Tyre, Ostia, in England, Holland, France, and the Scandinavian countries concrete structures sometimes deteriorated too quickly. Stonehenge and old marine structures in India, China, the low countries of Europe, France, England and Scandinavia are still there. They were built to *last*.

Practical classification of damages

Report No. 7 (Bruun, 1979b) discusses damage modes and damage mechanics based on practical experiments. When damage occurs in the prototype it most often starts around the M.S.L. It could, however, also start in the uppermost part of the mound in the crown due to heavy overwashing. This happens when breakwaters are too low. The normal case mentioned above is that the front slope starts "sacking" (making an S-shape). This may happen in a more or less graceful way. If some blocks in the uppermost armour move down to an elevation below or at the M.L.W. line while the second armour layer is not damaged, the mound may escape from more serious damage and repairs may be effected in time (see Bruun, 1979a, p.1, and fig.2). Removal of part of the upper armour layer does not necessarily mean a fatality. The more severe damage, which could develop to become a total destruction if allowed to continue, is the removal of the second of a two-block armour layer exposing the first sublayer which was not designed to stand up directly to wave forces. In such cases breakdown may proceed fast until complete "destruction". Then the time parameter obviously becomes very important for the fate of the mound.

A similar practical description of the stage and sequences of damage should probably be adopted in laboratory practice.

Damage levels

The problem of "damage level" raises two questions. One is concerned with the definition of "damage level" for similar designs (structures) which can be compared directly. The other refers to attempts to compare various structures.

Comparison between damages to various structures is very difficult as damage must be defined in relation to certain levels. As various types of structures demonstrate very different damage modes and breakdown pattern, a "straight comparison" is not possible. The only sensible way in which damage to structures of various design can be compared is by considering the consequences of the damage. In other words: What would be the next stage of damage to be expected to develop and what would be the further consequences?

RATIONAL DESIGN

Design formulas

The development of the numerous "design-formulas" mentioned and cautioned severely against by the International Waves Committee of the PIANC in its 1976—1977 report (PIANC, 1976), is, in fact, a somewhat sad result of a by far too long-lasting period of just empirically based laboratory exercises which, in turn, resulted in little or no progress in physical understanding of the problem. This is clearly evidenced by the "design-formulas" major disagreements associated with an endless collection of "constants" which were not constants at all as described by the International Committee in PIANC (1976). Characteristic in this respect are the attempts to distinguish between "non-breaking" and "breaking" conditions without defining the type of breaking which is all-important for the hydrodynamics and forces involved. It is not even stated whether breaking takes place on, at, or in front of the breakwater! The practical engineer or supervisor — in private — therefore mostly had views which often deviated considerably from conventional laboratory-thinking.

Based on his experience, the Dutch field engineer knew that uprush on the dykes penetrated to its highest elevation when waves were long and that this happened mainly at the end of the storm in the first phases of attenuation. The Danish dune engineer observed that his sand dunes or "sea dyke" slid down (slumped) during the attenuation phase when waves ran up highest and eventually overwashed the crown, while during the peak of the storm uprush was less and damage mainly concentrated in the development of erosion scarps on the front side of the dune. The Norwegian field supervisor was sometimes able to follow the destruction pattern at closer hand. He may not have noted the introductory damage during the peak of the storm, but he saw how the major breakdowns developed due to large and long waves overrunning the entire structure during the latter phases of the storm (Bruun and Günbak, 1978). During recent years a better understanding of the basic hydrodynamic aspects of the problem has been gained allowing a more rational, not just a formulae — and wave generator — approach. Results are published in Kidby et al. (1964), Bruun and Günbak (1976, 1977, 1978), Bruun and Johanneson (1976), PIANC (1976), Bruun (1979) and Plough (1979).

Short- and long-term test criteria

One must distinguish between short-term and long-term, corresponding to nature's own way of testing a particular structure.

Short-term extreme events usually present the actual design criteria. In most cases stability is the most important and adamant criterion while a certain amount of overwash may be allowed. In other cases, the requirement for a minimum of overwash is mandatory due to the use of the area just behind the mound (Rottinghaus, 1971).

It is necessary to test extreme storms as they are expected to develop in all phases. It is also necessary to ensure that scale effects do not occur, either in the hydrodynamics or in time (duration) aspects.

Long-term damage accumulates from small damages over many years. Initially it is almost identical with 0-damage conditions. The character and extent of such damages depend not only on structural and wave mechanics characteristics but also on the actual "workmanship" during construction, at the same time disregarding the fact that some damage always happens in the introductory phases. In addition, the mound itself will undoubtedly settle somewhat — and the underlying soil may also give way by compression and consolidation. Considering the fact that placement of 25 blocks in 5 rows horizontal and 5 rows vertical (along the slope) includes a total of 4.84×10^{13} variants, disregarding those combinations which are symmetrical around a vertical in the middle, it is clear that two sections of the mound can hardly be expected to behave identically. Due to the huge number of possibilities, those which obviously are going to demonstrate the highest degree of stability, however, will narrow down the number of practical possibilities. If all blocks demonstrate the same degree of stability in a certain equally exposed zone, they should have equal weight and an allover symmetrical geometry. This can only be achieved by regular block geometry, that means by using concrete blocks — or gabions! There should, however, always be consistency between action and reaction. The above also shows that it is risky to use blocks of "involved design", increasing the possibilities for failures. Practice has established this.

The local strength must be determined as an envelope covering all possible maxima. Figure 8 shows such an envelope schematically. It would be practical if laboratories provided the sponsor of tests with such an envelope to be used

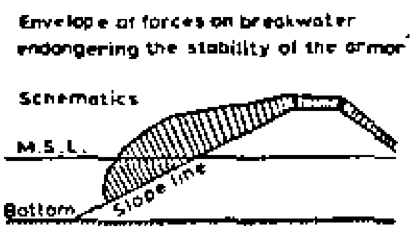

Fig. 8. Envelope of maximum forces on rubble mound breakwater.

for detailed design in cooperation with the laboratory, by placement of the heaviest blocks where they are most needed. In practical terms this means that:

(a) The *risk of damage* should be evaluated all along the slope.

(b) The *consequence of damage* should be evaluated. Consequence may, e.g., be measured in costs of repairs.

(c) *Risk times consequence* will then be the practical basis for design. This should not mean any kind of impractical "over-sophistication" of the construction work but just that placement is "as intelligent as possible".

Report No. 7 mentions the selection of short-term criteria for design with particular reference to "Final Report by the Waves Committee" (PIANC, 1976). Probably the most radical change in recommended practice in selection of design criteria for practical and safe design of a rubble mound structure was the introduction of the $\xi = tg\alpha/\sqrt{H/L_0}$ factor which created the basis for a more rational and safe design procedure. The new design criterion is the result of meticulous studies of wave records from the site in question, including spectral characteristics and runs or sequences of waves which are particularly dangerous to stability. This involves investigations of groups of wave heights as well as periods (usually this goes together) mainly for extreme events, considering all phases of the storm and ranging from the generation phase over the peak until the attenuation phases (Ewing, 1973; Rye, 1974; Bruun and Günbak, 1976, 1977, 1978; Goda, 1976; Chakrabarti and Cooley, 1977; Houmb and Vik, 1977; Günbak, 1978; Houmb et al., 1978). Report No. 7 mentions recording of damages by rational methods (Bruun and Günbak, 1978; Kristinson and Eliasson, 1978).

A high degree of "intertangling" and friction is always an advantage. Permeability has proven to be a questionable advantage for stability. The artificial block mounds of higher initial stability are not "good loosers". When damage first starts it tends to progress fast. Mounds of these blocks do not fail nearly as "gracefully" as mounds built up of natural rock do. This is due to their steep slope and to their high permeability which makes them "run-full" or "fluidize" and slide down. Long waves are particularly dangerous in this respect. While the so-called "no-damage not overtopping criteria" may be argued for normal rock mounds, which usually disintegrate rather slowly in "steps" and not in "slides", it seems much more logical to use these criteria for mounds of artificial blocks because initial damage may be followed so quickly by fatality (Bruun, 1979). However, we have to face the reality that it is very difficult or impossible to get the large-size natural rock, meaning blocks above 20 tonnes. For heavily exposed breakwaters, artificial blocks have therefore now largely taken over the block market, even in a mountainous country like Norway (Kjelstrup, 1977). This, in turn, has resulted in a large number of practical problems including:

(a) Manufacturing of concrete with as much resistance to weathering as possible.

(b) Proper storage of these blocks for maturing.

(c) Placement of blocks without damaging them.

(d) Proper equipment for safe placement of the blocks.

Although this paper is not intended to deal with all these practical subjects, which are handled well in many papers by highly experienced authors, it is emphasized very strongly that placement of blocks without damaging them is very essential. Failures have happened which could be a result of breakages to blocks with slim geometry placed in the most dangerous area at the M.S.L. line (Plough, 1979). For this reason, it is also best to use cranes placed on the breakwater as it is built forward or on a trestle as it is sometimes done (although we know from past experience that this is expensive), because the use of floating equipment causes more breakage due to wave motion making the blocks bump against each other. As breakdown quite often occurs as mass slides, particularly with the steep slopes, stability against sliding by friction between armour and the first sublayer is a necessity. One way of achieving this is to prepare the sublayer, usually built of quarried rock with a surface as rough as possible by simply turning some of the blocks so that the longest side is perpendicular to the slope. These blocks should, as mentioned earlier with reference to Fig. 5, have their "root" down in the sublayer. And it is important that all layers are thick enough to absorb variances in thickness without risks of weakening the structure. The strength of a chain is well-known to be the strength of its weakest link. Very often there is a marked difference between the draftman's design and the way it "came out".

DATA NEEDED FOR DESIGN

Wave data

In order to design realistically it is is important:

(1) To know short-term as well as long-term wave statistics in detail. This includes statistics on wave steepness and grouping of waves as well as information on the frequency and duration of storms of a certain magnitude, defined in wave height and period ranges (Hudson, 1961; Brettschneider, 1963; Nolte and Hsu, 1972; Rye, 1974; Longuet-Higgins, 1975; Bruun and Günbak, 1976, 1977, 1978; Houmb and Vik, 1977; Houmb et al., 1978; PIANC, 1978). To evaluate waves in shallow water, detailed knowledge on waves and water table must be obtained (Jonsson and Jacobson, 1973).

(2) To undertake analysis of the available wave data in order to produce realistic hydrodynamic and practical criteria for model experiments. The importance of the factors recommended by the Waves Committee of the Permanent International Association of Navigation Congresses (PIANC, 1978) should be considered. This includes probability and risk analyses based on the ξ-factor (Bruun and Günbak, 1976, 1977, 1978) as well as grouping of high waves (Nolte and Hsu, 1972; Ewing, 1973; Rye, 1974; Longuet-Higgins, 1975; Goda, 1976; Chakrabarti and Cooley, 1977; Overvik and Houmb, 1977).

(3) To predict wave-climate conditions during the construction period (Bruun and Günbak, 1977; Houmb and Vik, 1977; Houmb et al., 1978),

enabling the contractor to proceed in the safest and most economical way so that the designer and his client can take over a through and well-built ("to-specification") structure and not a product which is scarred by irregularities and wounds, some of which are not (and could not be) properly healed. Consequently, damages may soon happen and may wind up in complete destruction.

Other information needed for design includes:

(4) Data on the stability of the bottom surface and lower layers should be well known. This requires soil mechanics tests and analysis.

(5) Materials to be used must be sound and must not submit too easily to weathering, possibly including ice exposures.

(6) The contractor should have the proper equipment which allows safe placement of elements in the structure (Kjelstrup, 1977).

General requirements to design construction and maintenance include:

(7) The design as a whole as well as in detail should be practical. Fine drawings and laboratory tests are not enough. It may not be possible to follow these in practice (Fig.9).

(8) Supervision must be strict. By far too often supervision has been sloppy. Good supervision requires a full understanding by the supervisor of all aspects of design and construction — and therefore also some good old field engineers or foremen who have good eyes and a lot of experiences with waves, materials and equipment, making it possible for them to improvise and not leave this aspect solely to the contractors discretion. An engineer may be good on roads, sewers and bridges, but this does not necessarily mean that he is also good in marine structures. The cheapest bid may sound attractive but it is not necessarily the best choice. Earlier experience is — or should always be — a main factor. Better and more practical equipment has been put to use during recent years, e.g. backhoe and splithull barges.

(9) The situation and the appearance of the breakwater upon its completion is carefully recorded by adequate surveys.

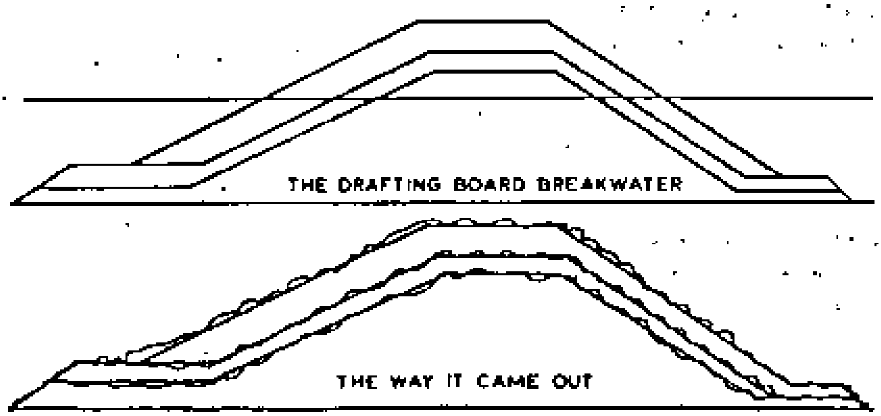

Fig.9. Theory and practice.

(10) The development of breakwater stability is followed closely by surveys including photography. The best, needless to say, would be to install a wave buoy in front of the structure, e.g. at 20—40 m depth and correlate any damage with the observed wave action.

(11) All maintenance should be recorded not only "in total" but in detail which permits judgement as to why maintenance was particularly heavy in certain sections of the breakwater. The wave situation which caused the damage should be thoroughly recorded in all details which are necessary for analysis of the wave-structure interaction.

In its final section, Report No. 7 (Bruun, 1979b) mentions laboratory procedures and suggests quantitative analyses replacing the earlier practice of counting of blocks which "left the slope" and/or "rolled down" a most unsatisfactory and superficial way of classifying and measuring damages. The OBDS (optical breakdown sensor) is mentioned in detail in Kristinson and Elisson (1978). It is useful for objective recording of damages up to a certain level.

PRACTICAL ASPECTS OF DESIGN AND CONSTRUCTION OF BREAKWATERS WITH SPECIAL REFERENCE TO NORWAY

Introduction

In planning and building breakwaters exposed to the wave action of more than a normal fjord sea, close cooperation is required between those who design the breakwaters and personnel who have practical experience from construction work. It all too often happens that constructions, which from purely theoretical or laboratory considerations seem reliable, may prove to be very difficult or unnecessarily expensive to build.

Unfortunately, it has become usual in laboratories around the world that personnel have only laboratory experience and little or no practical experience. At the same time, it is unfortunate when central authorities are fully occupied with administrative activities and only have been able to follow developments in the field through sporadic inspections of building sites. In order to find a solution that is favourable from an economic as well as a technical point of view, close cooperation between theorists and practitioners is needed. Only a few have experience of both.

The writers, together with colleagues elsewhere, have found that this situation is common everywhere and not confined to any particular nation. In some countries, breakwaters are planned by consultants and built by contractors, so that the government or the builders only carry out the inspection. In other countries, among them Norway, planning and construction is carried out by the government. The government undertakes investigations, does the planning, handles building policies and constructs the harbours. In this case not enough emphasis may be put on supervisory activity, and one does not have the guarantee of economically appropriate and technically sound work which results from competition.

The following is a brief summary of the conditions which must be clarified and also the mistakes that may arise if the coordination between the various authorities is inadequate, or if individuals or groups proceed with insufficient contact with other professionals. If more complicated conditions with several possible alternatives are involved, three-dimensional model tests will usually be carried out in order to find the technically best solution. Now almost all such projects seem to have a political edge. Since it is the politicians who finally make the decisions in our democratic society, it often happens that the technical viewpoints must give way to political. In other words, the harbour may be built on the wrong place seen professionally. It is not unusual that non-technical consulting companies design breakwaters for 30 and 40 m depths. Laboratories too, may propose solutions that are prohibitive due to the costs.

Design criteria

If sufficient time is available for pre-studies, one gets the most reliable information on waves by recording them over a period of at least one year, preferably longer. In many places visual observations can be most valuable. It is always advisable to listen to what the local population has to say.

Regarding existing conditions one can usually get objective information. On the other hand, information is often given in order to acquire a particular site or harbour geometry, one must be careful as such information more or less unconsciously may tend to pursue one particular solution.

Choice of breakwater profile

The choice of construction materials is important. The building materials locally available have great influence on the breakwater's cost. If there is good local rock, one must seek a breakwater construction with the greatest possible use of quarried rock. Concrete will always be more expensive, and if there is no concrete sand at the site, concrete blocks of various kinds (tetrapods, dolos, tribars, etc.) may become very expensive. Choice of concrete may be an issue at sites where one has poor rock, but a good supply of sand for construction.

Choice of equipment for construction

In order to build a particular profile, mechanical equipment with particular qualities is required, especially for lifting (ton-meter capacity). Mechanical equipment has been under rapid development in recent times. Machines that were usual just 10 years ago can no longer be used today. In Norway, however, they may not be allowed by the Norwegian Labour Commission! Cranes on tracks have now disappeared from construction sites.

Transportation equipment too looks different now from the way it used to be. It is especially in this field that consultants and supervisors, who make the decisions on purchasing of equipment, have not always followed the development closely enough. In this field, close cooperation is urgently required between the field supervisor who has the practical experience, the person who is to design the profile, and those who will approve the project. One has to have local site experience in order to know and understand completely all the problems that will arise during the construction of a breakwater. Short inspections at the building sites may only give a superficial impression. It is therefore the construction engineer (and not the "desk man") who must have the final say with respect to the choice of a breakwater profile.

Choice of profile

Practical considerations are often decisive in this aspect. Details that may seem reliable, but which in reality are of minor importance to stability, can be expensive to carry out in practice. Also, sometimes the structures designed may be impossible to build! The one who designs the structure must clearly understand under what conditions the construction engineer has to work. In tropical waters and in areas where one can count on long periods of good weather, certain "theoretical" profiles may be built as they were designed, as long as they are not all too involved. However, it is unavoidable, at any rate on the coast of North Norway, that one has often great difficulty in following prescribed profiles, even with relatively moderate wave action. It may happen that one must make provisional protection to save the material already in place. This of course means that too complicated profiles cannot be carried out. The result can easily be that builders put less emphasis on the blueprints, but build the breakwater according to usual practice and as they think is best. This can be a problem if changes in the profile become necessary for reasons of stability.

Length of construction period

It makes all the difference in the world for someone designing a breakwater to know how much construction time one can expect to have, in order to complete the work. Hasty work should be avoided but may sometimes become necessary. Considerations of rapid execution can weigh so heavily that one is forced to compromise the stability to a certain extent. There will always be some movement in a breakwater during its first years. Settling is most usual, but washouts can also arise as a result of:

(a) Settling of subsoil.
(b) Settling of the rock mass, see Figs. 10 and 11.
(c) Washing out of fines and possibly snow in the core material.
(d) Storm damage.

Therefore, it is usually best to build and leave a breakwater one or preferably two years before one completes the permanent construction. This increases the stability. However, the breakwater can be so exposed that it is not advis-

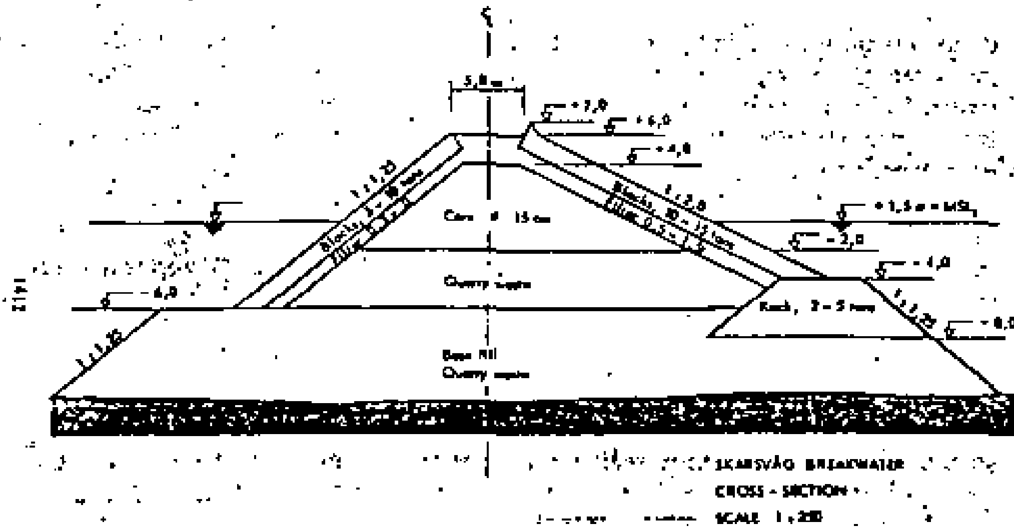

Fig.10. Breakwater profile for a medium exposed construction on the Norwegian arctic coast.

able to leave an open structure unfinished or unprotected. This may cause considerable extra work.

Quality control

In this respect countries where breakwaters are designed by consultants, built by contractors and inspected by the consultant have certain advantages. On the other hand, if the construction supervisor keeps an eye open for the usual weaknesses in a breakwater construction, the worst calamities should be avoidable. It is, however, often the young and less experienced engineers who are sent out to the sites. In such cases good contact between him and the construction supervisor is essential. Most important is that frequent inspections of the submerged part are undertaken. The necessary equipment for such work is a plumb line, a water telescope, and at greater depths also the use of divers. The most common mistake is that because of lack of underwater inspection the superstructure is built on a foundation with a steep slope below water. If the blocks under water cannot be placed with a crane barge or a crane on the breakwater crown with a long enough boom, there is no other mean of moving blocks below the water level than by blasting with small charges, in order to place them in the (Fig.12) prescribed slope. When a solid foundation is created in this way, the superstructure can be built up with clamshell or backhoe equipment, or with the help of a crane. If progress is to be made in several stages, an exact work description is required. With especially exposed wave breaker constructions where both floating and land-based equipment is to be used, the planner must have a complete knowledge of the functional operation.

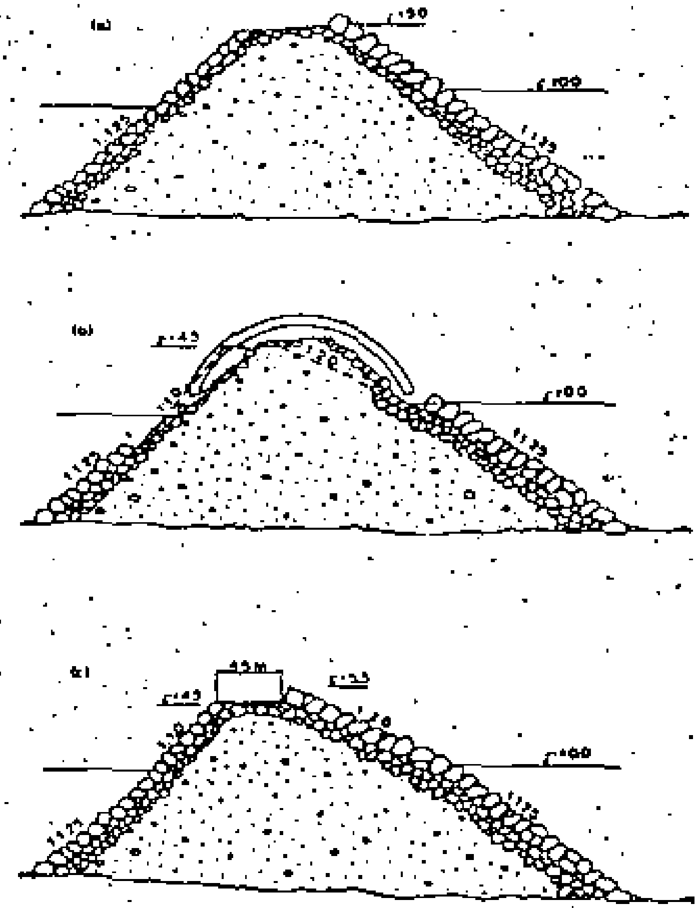

Fig.11. A. Breakwater after first season of construction. Both slopes with natural angle (1:1.25). The jetty has an extra height of 0.5 m (Gryllefjord, Troms).
B. After one or two years the blocks are removed from the slopes, and some of the core is moved from the outside to the inside to achieve a different slope of 1:2.0. All done in one operation with a medium-sized (48 metric ton) backhoe bagger.
C. After two more years the settlings have come to an end and the breakwater is finished with concrete cover and parapet connected to the armour blocks. Scale 1:250.

In many cases some time must pass between each step of the operation. One must then have to protect the unfinished work so that it is not lost. Transportation of heavy building equipment by sea or by land, and assembly is today so expensive that attempts must be made to hold such costs down. Another consideration is that the labour force must be adjusted according to the work that is to be done. All these factors must be subjected to a joint evaluation, and even if the designer knows the main features of the planned structure, he must not fail to listen to the supervisory engineer, who is and will continue to be the key man in the construction work.

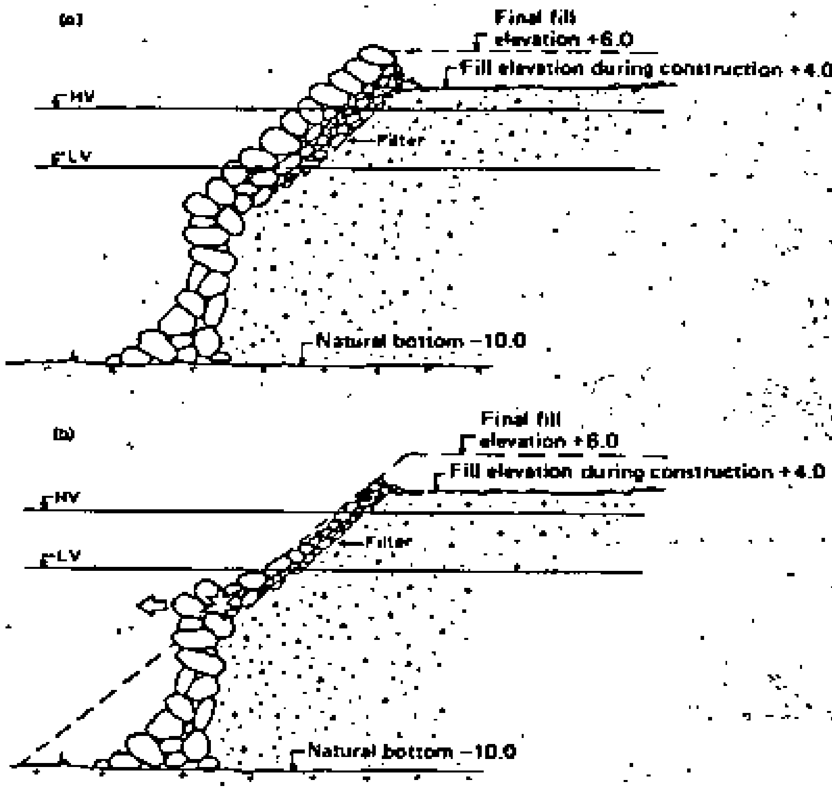

Fig. 12. A. An unstable breakwater head, built up on a steep slope below water. B. Down-blasting of the underwater armour blocks with small explosives to obtain the prescribed slope.

CONCLUSION

Evaluation of the stability of a rubble-mound structure must of necessity consider a number of factors, including some which so far have not been paid proper attention. Thorough stability analyses must include consideration of the overall stability of the armour and its sublayer(s), the stability of the single armour unit and the structural stability of the single unit. Factors like "intertangling" or "interknitting" of blocks are important and should be looked into. Friction between the armour layer and its first sublayer is a very important factor. Data needed for the design include adequate wave data allowing short-term and long-term analyses useful for extrapolations. Particularly important is the evaluation of steepness distributions. Reliable laboratory procedures must be secured in future model tests and scale effects must be included in scaling.

A number of practical aspects must be considered in design to avoid that the final structure deviates too much from the design. It is therefore necessary that designers, builders and construction supervisors get together before the design is finalized.

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REASONS FOR DAMAGES TO THE ARZEW EL DJEDID BREAKWATER

DECEMBER 28-29, 1980

Abstract - The main topic of this report is an evaluatio of the reasons for the failure or heavy damages to the breakwater. Basic information and material include: Site-inspection in January, 1981, Progress Report by DHI June, 1981, Final Report by DHI, received on August 21st (DAREP) 1981, and report by Christiani and Nielsen, whic is an integrated part of the DAREP.
The report also mentions briefly the suggested repair works and make a couple of overall suggestions for the final design.

To avoid a voluminous and less over-sightly report refer ences are - to a large extent - made to existing reports or publications. The most important of these are enclose as a total of 8 Appendices.

INTRODUCTION

To make such evaluation as objective as possible it was most preferabl to review the reasons for damages to Mound Breakwaters in general and next to compare the damages to the Arzew el Djedid breakwater to the general experiences on breakwater stabilities and damages.

As an integrated part of the report I have therefore included artic on "Common Reasons for Damages to Rubble Mound Breakwaters", publishe by Coastal Engineering International" in 1979 as Appendix 1 (ref. 3). Since then (two years) some further progress has been made particular in the wave hydrodynamics versus structures' field. In reviewing the 1979-article, as done in the report, the most recent experience has been added as described in Appendix 5 and 8 on Waves, in Appendix 3, 4, and 7 (Chapter 5, pp 195-223) on Waves versus structures, and in refs. 4 and 21 - (Appendix 3 and 4) on constructions.

GENERAL ABOUT THE STABILITY OF MOUND BREAKWATERS

Mound breakwaters, by nature, are rather "fragile" and no wonder. As explained in Appendix 1, "Common Reasons for Damages to Mound Breakwaters", 1979, some damages therefore, can always be expected, but if the design has been well done and the construction equally well under taken the unavoidable initial superficial damages to armour layers will fade out, following a few storms.

Under equal exposure along the breakwater damages, therefore, will be randomly distributed. If a breakwater fails along a longer section, leaving other sections intact without damages, this is either caused by unequal exposure to wave action, unequal soils condition, unequal materials, incl. materials handling, or unequal construction procedu incl. supervision.

Assuming introductorily that all conditions were equal along the Arzew el Djedid breakwater reference is made to Fig. 1 (Fig. 1 of Appendix 1).

DAMAGES BY KNOCK-OUTS

General - Damages by knock-outs are caused by concentrated hydrodynamic pressures most often associated with plunging waves.

Damages by knock-outs at Arzew el Djedid - There is no direct evidence that this wave damage has happened, but it is on the other hand certain that the socalled ξ-factor, $\xi = \frac{\tan\alpha \cdot 1,25\ T}{\sqrt{H}}$,

α = slope angle, T = wave period and H = wave height at the toe of the slope, has been located in the surging breaker area, where $\xi_b >$ about 2.o, referring to breaker conditions.
Example, using data from the DAREP report (1981):

$$H \sim H_b \sim 6{,}25\ \text{m},\ T = 1o\ \ (13)\ \ \text{sec.}$$

$$\xi = \frac{3 \cdot 1{,}25 \cdot 1o}{4 \cdot \sqrt{6{,}25}} = 3{,}75\ (4.9)$$

If H_b is 9 meters, ξ_b is 3,1, still in the surging area.

As breakers, therefore, during the severe storm in Dec. 1980 largely must have been of the surging type, it is not likely that plungers occurred. This discloses the shock-pressures, which usually accompan[y] plunging waves, creating high local pressures, which may easily damage blocks in exposed positions and particularly multilegged blocks, which will tend to "bridge" or "cantilever".

DAMAGES BY LIFTS

General - During the storm of Dec. 28-29, 198o, waves of periods exceeding 1o sec. occurred , particularly during the later phases of the storm. According to earlier tests with Tetrapodes, as e.g. reported on in refs. (15,17) "resonance", that means uprush/downrush period is equal to wave period (refs. 2,3,4,2o) may occur for ξ-values of about 4 to 6 for a Tetrapod slope of 4/3. As explained thoroughly in refs. 2,3,4, and 8 the condition termed "resonance" is characteristic of the onset of very deep downrushes and these downrushes stay very low, as long as the wave period exceedes the resonance period. Simultaneously with the onset of resonance the stability of the mound enters in a critical condition due to the following circumstances (refs. 2,3,4, and 8)

(a) downrush velocities are maximum.
(b) the velocities in the arriving "next wave" attain maximum values along the slope.
(c) hydrostatic pressures from inside the mound obtain maximum values.

With waves of $H_b \sim 6{,}25$ m and T = 13 sec., ξ becomes about 4.9 for the 4/3 slope. Tetrapode mound, or corresponding or close to reso[n]ance. For higher periods or lower waves or both, conditions are still close to resonance and on the particular dangerous side of the resonance period.

The question is now, how a 4/3 slope of Tetrapodes behave under such conditions.

In order to discuss this subject let us, introductorily, look at the relative stability of mounds of multilegged blocks as compared to mounds of other blocks and at the same time investigate what may be termed "the optimum slope stability".

The reason for the invention of the multilegged blocks was that they were able to stay stable for steeper slopes than quarry or rectangular blocks, thereby saving materials. This, however, raises the question of risks involved in the use of such blocks on a steep slope.

Referring to elementary principles of soil mechanics a steep slope of uniform materials of course is less stable than a slope of less steepness. This, however, still refers to a homogenous slope and not to a slope, which is built up of layers of entirely different materials.

Looking at the characteristics of mounds of multilegged blocks of different shapes all have larger permeability, 45 to 55% than mounds of rectangular or cubic blocks or rock mounds (refs. 8 and 21). At the first look such high permeability appears to be an advantage, as energy absorbtion increases (reflection decreases), and the consumption of materials decreases by about 15-20% compared to rock-mounds and by about 10% compared to rectangular or cubic blocks left in pell-mell. On the other hand, due to less volume of material gravity forces, thereby friction between blocks, become less, at the same time as forces by the flowing water in the mound increases. This in turn means that hydrostatic, as well as hydrodynamic forces, which both try to dislocate blocks during a downrush, increase. The advantages of using multilegged blocks, therefore, may be outweighted by disadvantages in other respects. This depends not least upon the slope angle. Bruun and Johannesson (ref. 1 and Appendix 2) analyse in great detail the forces on a single unit in a mound, (ref. 1). Using a ball as a theoretical example it is shown (ref. 1), how it is possible to calculate the number of blocks, which are necessary, when piled up on the top of each other, to produce enough pressure-force to keep a particular ball in place, assuming a certain lift force, friction between the balls and friction between the balls and the 1 st. sublayer. It is obvious that any reduction of the squeezing force on the sides will make it easier for the ball to escape. Such reduction may be a result of high friction forces between the ball and its sublayer. On the other hand a large friction force between armour and sublayer will decrease the possibility for slides. While single units, therefore, may stay in place, if adequately squeezed, layers of block-units need adequate friction between the blocks and the sublayer to avoid a mass sliding (ref. 1, Appendix 2 and ref. 8, Appendix 7, Chapter 3).

As armour blocks, ranging from quarried rock over rectangular blocks to multilegged blocks, have very different friction capability, it is easily understandable, why the optimum stability slope for blocks of various geometries varies greatly. Table 1 from ref. 15 (Appendix 3) shows the optimum stability slope for 6 different types of armour units.

Table 1. Optimum stability slope for various blocks (ref. 15)

Type of Armour Unit	Cotα optimum
Rip-rap	> 5,o
Quarry stones	4,o to 5,o
Parallelopedic blocks	3,o to 4,o
Stabits	2,o to 2,5
Tetrapodes	2,o to 2,5
Dolos	1,75 to 2,o

For rip-rap and quarry stones the direct gravity force against hydrodynamic lift force apparently is the most important factor. For concrete blocks of less involved geometry the importance of side pressure increases and for the complex blocks with legs, side pressur or "interknitting-intertangling" is most important.

From Table 1 it may be seen that the optimum stability for multilegge blocks is less than the slope with which these mounds normally are built (4/3). Consequently the wisdom of using a slope steeper than the optimum may be questioned. This may be realized in a different way as explained below.

Stability of an armour layer may be written as (refs.15,16, and 17)

$$w = \gamma W \, H^3 \, R \, \Psi \qquad (1)$$

when γW = specific gravity of water

H = wave height at toe of the structure

$R = Sr/(Sr - 1)^3$

Sr= relative weight or armour units

Ψ = stability function = $f(\alpha, H/L_o)$

Flow characteristics on rough permeable slopes are well represented by the Iribarren number (ref. 13):

$$Ir = \tan\alpha / \sqrt{H/L_o} \qquad (2)$$

Ψ can be written

$$\Psi = A(Ir - Iro) \exp B(Ir - Iro)$$

$$Ir > Iro \qquad (3)$$

$$Iro = 2,654 \tan\alpha \qquad (4)$$

$$Ir = \tan\alpha / \sqrt{H/Lo} \qquad (5)$$

A and B are fit coefficients, which depend on the type of armour unit and slope angle.

For Tetrapodes one has:

$$\Psi(\alpha = 4/3) = o,o5$$

$$\Psi(\alpha = 2) = o,o1\text{-}o,o2$$

Scatter and Confidence Bands-Tests

have shown that there exists an important randomness in the structura response. The stability of the main armour layer varies with the character of the unit and with the same mound ...

Table 2. Fit-coefficients for Tetrapodes (ref. 15).

α	A	B	Iro
1,33	0,03380	-0,3141	1,99
1,50	0,02788	-0,3993	1,77
2,00	0,02050	-0,5078	1,33

Fit-coefficients A and B (Table 2) were obtained by means of linear regression using the change of variables:

$$\Psi = Ir - Iro \quad (6)$$

$$\xi = \ln (4/Ir - Iro) \quad (7)$$

through which the fit model defined in eqs. (3), (4), and (5) is transformed into the straight line:

$$\xi = mx + n \quad (8)$$

where $m = B$

$n = \ln A$

Scatter in the experimental results can be accounted for by modifying eq. (8) as:

$$\xi = mx + n + S$$

when S is a random variable of averages. S depends solely on the structure.

Assuming that S is Gaussian distribution, and estimating its variances (S^2) by means of experimental results (X_i, ξ_i) according to

$$S^2 = \frac{1}{N-1} \sum_{i=1}^{N} (\xi - mX_i - n)^2 \quad (9)$$

where N is the number of data, confidence bands may be defined that measure scatter of the data.

Detailed calculations show that the highest control curve for 95% confidence level, which may be taken as a stability function of design $\Psi_D = f(Ir)$ can be obtained by multiplying A in Table 2 by a factor 2 for Tetrapodes. For rip-rap the corresponding factor is 1,5, for quarry stones 1,5, and for parallellopedic blocks 2,5. This factor can be interpreted as a safety coefficient in respect to structural response of the mound. The random variable is highest for parallellopedic blocks (2,5), but also rather high for Tretrapodes (2,0).

Confidence band factors for initiation of damages for Tetrapodes are given in Table 3 and control curves are obtained by multiplying the best fit curves eq. (3) by these factors.

Table 3. Confidence band factors for initiation of damage for Tetrapodes (refs.15,16, and 17)

α	90%	95%	99%
1,33	1,51	1,64	1,31
	o,66	o,61	o,52
1,5o	1,99	2,27	2,93
	o,5o	o,44	o,34
2,o	1,73	1,93	2,37
	o,58	o,52	o,42

Apparently slope α = 1,5o is the least safe. The large difference in regression between α = 1,33 and 1,5 is peculiar, but confidence is a little better for α = 1,33. Yet it is apparent that one has to count on large scattering for all slopes, but that block size for α = 2 can be about half of, what it has to be for α = 1,5 and α = 1,33, with 1,33 giving the highest weight, but a little better structural confidence than α = 1,5 and α = 2,o.

What may be deduced from this is that other factors included in overall confidence, like structural aspects and concrete quality, must be as safe as possible, for the steep slope, α = 1,33.

The experience with multilegged blocks has been that this often has not been the case.

A slope of α = 4/3 therefore is only safe if:

(a) blocks are large enough to enable the mound to withstand solitary, double (or triple) mammuth waves and 3 dim. effects.

(b) blocks are structurally sound with no tendency to easy breakage.

With respect to Arzew el Djedid (a) is not known, as no specific tests have been undertaken on such waves. Wave action along the breakwater varies somewhat in details and it may happen that at one particular location, due to slight changes in bottom topography, the possibility for larger waves by a tightening of orthogonals, is larger than at other places. If such waves damage the mound, further damage may spread out from that point due to 3-dimensional effects.

The same may happen, if the mound is damaged due to structural weakness in one particular area, e.g. where 3 - 4 blocks are broken. In the α = 4/3 slope damage will spread out rapidly, because such slope is fragile and vulnerable due to its steepness.

Damage-picture for Lift-outs. General - Lift-outs will occur, where the combined lift-forces are maximum and that is, where forces by downrush, toe velocities in the plunging, collapsing or surging wave and hydrostatic forces inside the mound join hands. This happens when downrush is as low as possible, or at "Resonance". The Resonance area, therefore, is found below the still-water level and

1,5 to 2,o wave heights (Hs) or about 1o-12 meters maximum in the case of a 1o meters maximum wave height or about 6 meters for a 5 meter wave of resonance period.

In the case armour blocks are too light, this means that armour will be lifted out and washed down by the downrush and left as a deposit slope below the 2,o Hs level or - in the case of a 1o meter maximum wave - below minus 1o to 12 meters. This process builds up a platform of "debris" consisting of armour blocks usually in the lower most layers with sublayer material on the top, often leaving cavities caused by the loose settling of the materials. A characteristic S-slope builds up presenting an ideal geometry for stability as mentioned in great detail in refs.1,2and 6 with reference to actual cases. See figures 2,3, and 4, and Appendix 7, Chapter 3.

Damage-picture for Lift-outs at Arzew el Djedid - The below mentioned refers to the geometry of the damaged section, which is described in great detail in the "Damage Report of August, 1981" by DHI and Christiani and Nielsen. In the following these reports are referred to as DAREP. As a matter of ease for the reader, reference is made mainly to figures of the DAREP.

Ref.: Profile 4o, Figs. 2,8,one of the characteristic profiles. It has a platform between -1o and -12 m and a deposit slope below -12 m. Other profiles are more complex. Profile 7o B (p 25 of DAREP) has some smaller platforms created by deposits of Tetrapodes down-slope in stairs or ridges, supporting the material behind them, giving a sawteeth geometry, including "vertical" and "horizontal" faces. This in turn has also caused the formation of cavities by bri[illegible]ing of blocks. The formation of such cavities, however, may also be a secondary damage caused by the suction out of loosely deposited blocks washed out from above. Cavities are also known to be formed in steep slopes around the M.S.L. due to the lack of enough sqeez-ing force from the blocks above caused by incidental bridging. A recent axample of that is found at the rock breakwater at Akraness in Iceland, which had a 1 in 1,5 slope of 4-6 tons rock (ab. 3-4 meters storm waves). Several cavities have developed just above MSL (4 meters tidal range).

With reference to several examples of platform location and formati[illegible] the DAREP finally recommends a platform profile. Profile E of the report, for execution of the initial repairs. Its profile follows largely the results of model studies. The platform is 16 meters wide and elevation is -5 meters. With reference to the later mentio[illegible] Appendices 5 and 8, on wave criteria, including wave trains of particularly dangerous character, it is believed that the elevation of the platform, where it is possible and not hampered by deposit[illegible] slopes, could be lower, e.g. as indicated, in fact, by the form-ation of such platform in profile E, fig. 5.18. In such case the platform could raise from the outer edge at -8 (-9) meters to -45 meters in the about 16 meters wide berm to the toe of the upper slope rounded up towards this slope and supplied with larger "corne[illegible] blocks. In this way the repair-profile would become more practical and simulate the lab-profile, following wave action by 6 to 6,5 meter waves, better (ref. profile E, Fig. 5.18). See later under initial repairs.

The DAREP describes the condition of the blocks, which apparently have been lifted up (and rolled down). One may visualize that: first the armour came down and next the the sublayers, with the result that sublayer materials (of reported less block size, B4, as required) was placed on the top and in between the Tetrapodes.

Looking at the deposit slope below the platform (Profile 4o, Fig. 2,8) and the size of blocks found in said slope; it seems quite obvious that the blocks in the upper slope were lifted up and rolled down. It is possible - or even likely - that this was caused by a succession of large waves and perhaps by some "mammuth waves", as such waves will cause the highest lift forces and also "roll-down"-forces; resulting in the development of a berm. In ref.1, Appendix 2 it is shown, how such profile in fact simulates a "step-profile" in beach morphology. Fig. 5 (ref. 1) shows, how it was possible to construct a berm profile for a mound breakwater from a beach profile. The importance of this is that it, thereby, may be concluded that the mechanics of the formation of the two profiles is similar. That means the "Step" in the mound profile is actually formed by a deep downrush penetrating almost as deep as the outer edge of the berm. Transferring this to the cases in Arzew el Djedid and to Akraness, Iceland, it means that in the final profiles the outer edge of the berm shall be located in 8 to lo meters at Arzew el Djedid and at -5 meters in Akraness. It will be so in the final repair profile in the Icelandic case. Berm will be 8 meters wide in Iceland and slope 1 in lo giving elevation - 4 m at the inner edge of the berm to be rounded up towards the upper slope. This is a practical geometry.

DAMAGES BY SLIDES

General - There are two different kind of slides. One is a soil mechanic slide, which may take place in the breakwater itself or below the breakwater that means a "soils-break".

Soils-failures in mounds are usually demonstrated as a slip-circle development, by which material on the top settles almost vertically down. This, e.g., happened during the 1969-earthquake with the Gueria harbour in Eastern Venezuela. It is not very likely to occur outside potential earthquake areas, as e.g. Akraness, Iceland. Another soils failure could be in the ground itself, e.g. where a layer of soft material or low shear strength is found below the breakwater, as in the case of DOS BOCAS harbour in Mexico. In such cases measures against slides may have to be taken, e.g. by over-size toes to counterbalance a slide caused by the weight of the breakwater trunk. This will be done at DOS Bocas, but mainly to provide a solid support for a surface slide or armour and first sublayer. Such slides may take place as a result of downrush forces on an armour layer of relatively high friction due to protruding elements and a too smooth surface of the first sublayer. This could e.g. be caused by a too small block size of the first sublayer. Refs. 3, 4, 6, and 8 give a full description of this and warn against such occurrences. Ref. 6 is included in Appendix 3, ref. 8 as Appendix 7 (Chapter 3). Table 3.8 on p 215 of Appendix 7 explains the advantages of inclusion of $W/2$ (half block size) layer between the armour layer and the first (standard) layer. Refs. 6 and 8 (Appendix 3 and 7) advise on, how first sublayer may be roughened to provide adequate friction (Fig. 6). A $W/2$ layer, however, is preferable and recommended by the US Army Corps of

Damage picture for slides at ARZEW EL DJEDID - With respect to the two "soil-slides", there is no evidence that such slides have taken place at Arzew el Djedid. Reference is also made to report by Dames and Moore on soils conditions in the breakwater line. The distribution of block sizes proves that. On the other hand, it is obvious that "slides" along the slope have happened - partly as "the next move" following a "lift" (e.g. photo 14). This is another indication of, what actually happened during the storm, which caused the final or major collapse of the breakwater at Arzew el Djedid.

GRADUAL BREAKDOWN OR FAILURE DUE TO FATIGUE

General - As explained in Appendix 1, p 262, this is a result of continued rocking of the blocks - which in turn in such case have not been large enough to withstand uplift and shear up-and-downrush forces. Such rocking may finally cause a collapse, if not properly checked in time. It is possible to locate such rocking blocks by visual observations of blocks during a calm weather period. It is revealed by scars in the blocks. One can also "hear" such movements during a storm as clicking sounds from the breakwater.

Damage picture for Gradual Breakdown at Arzew el Djedid. - It is likely that such gradual breakdown had started before the big storm. During the construction period 1976 - 1978 waves exeeding 3 meters were recorded 11 times. In April 78 waves were 5.6 to 6.0 meters.

REFLECTION FROM THE FACE OF THE WAVE SCREEN CAUSING SCOUR

General - As explained in Appendix 1 (Fig. 1) waves or wave uprush striking a vertical wall, like a wave screen, may cause severe scour in front of the wall, which in turn may be undermined and thereby collapse. This has happened in numerous cases, e.g. recently at Siñes, Portugal, Bilbao, Spain, Tripoli, Tripoli, and Akraness, Iceland.

Reflection and Scour at Arzew el Djedid. - The DAREP shows a number of surveys, incl. Figs. 2.3, 2.4 and 2.5 and a number of photos 3, 4, 9, 11, and 12, which clearly demonstrate the above mentioned effect. Another associated damage is described below.

UPLIFT PRESSURES ON CROWN SLAB AND CROWN BLOCKS

General - If the upper sections of the mound is highly permeable, pressure by uprush waters will penetrate the mound and exert uplift pressures on crown slab and/or blocks - as well as pressures on the armour of the inside slope, as it did at Akraness in Iceland. The result may be "bursts" or push-outs of these slabs or blocks and a resulting serious damage. The worst situation occurs, if material below the slab or blocks is also eroded away, leaving the slab or blocks (which may bridge or cantilever) free for direct uplift. The screen may then tumble over after having suffered damages, or it may turn over as a whole and suffer damages thereby.

Uplift Pressures on Crown at Arzew el Djedid. - Numerous surveys and photos, incl. the above mentioned and photos 1, 5, and 7 bear wittness of one of the main reasons for the collapse of the crown. Heavy overruns have contributed hereto. The concrete seems to have been up to required standards.

OVERWASHES OF THE CROWN

General - Uprun or uprush on a 4/3 slope of Tetrapodes will according to experiments (ref. 17 and Appendix 3) reach up about 1·Hb on the slope.

Overwashes at Arzew el Djedid - As waves of 9-lo meters height may have, or probably have, been integrated in wave trains during the Dec. 28-29, 1980 storm, and such waves may cause a 9 meter uprush, (refs. 2, 8, 14, and 24) it is no wonder that the wall with its wave screen at +7.5 m has been overtopped in some instances considerably. Introductorily the mound in front of the screen settled, exposing the screen's vertical wall. Uprush then continued eroding in front of the screen, at the same time as extreme uprushes hit the wall very hard causing high oversplashes, which came down with great power on the slab inside the screen, probably damaging the inside slope at the same time. This situation worsened, when the wave screen finally tumbled over. Reference is made to ref. 3, Appendix 1, and refs. 12, 14, and 17.

SCOUR OF THE BOTTOM IN FRONT OF THE MOUND

General - If the mound is located in relatively shallow water, heavy wave action, including reflections and deep downrushes, may cause scour of the bottom in front of the mound, particularly if the material is very fine sand or silt with little cohesion. The scour, however, may be covered up at the end of the storm. There are few cases, where such events have caused any real damage to mounds - and in case, only in very shallow water, where a longshore current assisted in carrying the eroded material away, thereby deepening the scour hole.

Scour of the Bottom in front of the Mound at Azew el Djedid. - There is no evidence that such scour has occurred, although - most likely it has been some scour at the extreme ends of the breakwater, which may have damaged the toe structure.

DEFECTS IN THE SOUNDNESS OF THE MATERIALS USED FOR CONSTRUCTION

General - It is a definite requirement to be fulfilled for marine construction works that all materials must be able to withstand the corroding and deteriorating agents in seawater. Ref. 21 lists requirements to all materials involved in marine construction. If the materials used include substances, which may be damaged by seawater, counter measures, like puzzolan,and various admixtures must be added. The requirement to density of the concrete is absolute.

Natural rock is not always as dense as desirable, and it may also be somewhat stratified due to high pressures and/or sedimentary origin. Vulcanic rock is sometimes very dense and heavy (3,3 specific gravity) and sometimes filled with holes and less dense layers, e.g. of hardened vulcanic bombs or ash (2,o specific gravity). All

Defects in the Soundness of the Materials used for construction

at Arzew el Djedid - Reference is made to the DAREP. There are no direct complaints on concrete standards. The C N report states that concrete quality was satisfactorily. The concrete blocks and the wave screen would undoubtedly have broken or ruptured regardless of concrete quality.

There are, however, several remarks, which refer to the sublayers, incl. "Un bon nombre fut jugé comme étant plus petits que 5'" (p 26, p 28, and p 29), "les roches paraissant avoir été cassées, car il y a de nombreux fragments dont les formes correspondent" (p 32), "les roches HC, si elles existent, sont évidement ensevelies" (p 36), etc., including remarks or information, which refers to "workmanship" or "controls" rather than to health or soundness of materials. The combined DHI and C N (DAREP) report mentions fractures in the rock, which due to chemical and perhaps mainly physical development, has made it difficult to produce large rock, like p 104, "Il est évident que cette structure rend difficile de produire de très grosses roches et diminue la resistance des roches produites." As mentioned on p 105 the rock has suffered further by the various handling procedures including 1 to 2 m chutes, consequently "les rapports mensuels de Parsons mentionnent également souvent un déficit de roches HC pendent les traveaux" (p 105). This deficit may have been corrected, however.

WORKMANSHIP

General - This important subject is dealt with in detail in refs. 3, 4, 6, and 21. Reference 4 is included in Appendix 3.

No reason is seen, at this opportunity, to repeat a number of mandatory and accepted rules or standards for good workmanship. The requirement to adequate and able supervision is absolute. This is often, when things actually went wrong.

Workmanship at Arzew el Djedid - Reference is made to a number of statements in DAREP, of which a few are repeated below:

p 20: "Hormis les blocs B5 qui on dévalé le talus comme indiqué sur certaines figures; on trouve une certaine quantité de déchets provenant de la construction, ainsi des déchets de béton, des agrégats, des coffrage, des ronds d'armature, photo 25."

p 26: "Il y a bon nombre d'exemples de roches cassées dans la première sous-couche, car il y a des fragments dont les formes correspondent. Cependant ces dégâts pourraient avoir en lieu pendent la construction".

p 30: Souvent, aucune roche de butée n'est trouvée (par exemple les profils 2, 30, 57B et 63B) ou ils se trouvent complètement enterrés (par exemple les profiles 20A et 25A)."

p 36: "Les roches HC, si elles existent, sont évidement ensevelis

(p 63 - B4 of 4 to 6 ts is placed on B1 of 0.2 - 1.0 ts. 0.2 should probably have been 0.4, P. Bruun).

DAMAGE TO INSIDE SLOPES

General - When heavy overwashes occur, the water will come down on the crown and the inside with great power (refs. 3 and 14). This is particularly true, if or when the wave screen collapses.

The slopes may also be damaged, if water penetrates through the upper permeable crown and exert pressure on the armour of the inner slope, as it happened at Akraness in Iceland.

Damage to inside slopes at Arzew el Djedid - The condition of the inner slope is described in detail in the DAREP, including Figs. 2.9, 2.1o, 2.11, 2.12, 2.13, 2.14, 2.15, and 2.16 and numerous photos incl. 21, 22, 24, 25, and 26. The effect of the impact of the falling water is obvious from these figures. Rock material in the upper slope has been eroded and left in the lower slope, e.g. Figs. 2.13 and 2.14. Concrete and rock debris has been carried over the damaged crown and left in the inner slope. This has - as in the outer slope - created some cavities in the confused mixtures of B5 (slided down from the top of the slope) B4, B3, and B1 rocks. The bottom outside the slope, however, does not seem to have been eroded.

The excess materials in some profiles, e.g. No. 51, Fig. 2.13, may be traced to deficits of materials in the outer slope.

DAMAGE TO EXTREME ENDS OF BREAKWATERS

General - This topic is not dealt with specifically in ref. 3, but is mentioned in refs. 8, Chapter 3, and in ref. 25 with reference to experiments. It is well known from experiments and field experiences that the extreme end of a breakwater has to be reinforced by larger blocks (up to twice the weight of trunk blocks). The reason for that is that heavy overwashes at the extreme end will attack the armour horizontally in an exposed condition, as the armour is not backed by breakwater trunk. Severe damages to breakwater heads have been experienced in numerous cases, some of the most serious on the Hawaiian Islands, due to the great horizontal power in waves of periods 15 to 24 sec. Blocks in the extreme ends are thereby washed out for deposit inside, often in an elongated mound like a "recurved spit" in coastal geomorphology (ref. 8, Appendix 7, Chapter 7). Any advantage associated with the use of multilegged blocks thereby vanishes.

Fig. 7 (ref. 25) shows, how such case was handled at a harbour at Sørvær in the Northern part of Norway located in the Lopp Sea, North Atlantic. The very exposed head at Sørvær harbour had suffered numerous damages. Based on old Danish experiences, e.g. at the old harbours at Skagen and Frederikshavn in Northern Jutland and model tests in Norway (ref. 25) the head was rebuilt lo years ago by curving the jetty head outward and providing it with a strong toe. This outlay has the following advantages. It gives:

(1) Higher stability of the head. Blocks in "corner" are locked in. Slides are supported by the toe-berm.

(2) Better and safer navigation conditions in the entrance due to "smooth" diffraction pattern along the head. This, of course, is of major importance for smaller vessels.

(3) No concentration of wave action in front of the head

(4) No outwash of materials into the entrance endangering navigation.

No damages have happened at Sørvær following the reconstruction of the head as explained above.

Damage to extreme Ends at Arzew el Djedid. - Both ends West and East, have suffered very severe damages as described in DAREP with references to Figs. 2.2, 2.6, 2.17, 2.18, and 2.19 and Photos 28, 29, 30, 31, and 66 to 72. The damage picture conforms well with the general picture described above: armour and sublayers washed out and deposited inside in an elongated spit, 2nd sublayer exposed (Fig. 2.17) and debris of armour and first sublayer deposited down slope (Figs. 2.15 and 2.17). The geometry (layout) and structural design of the head has definitely not been adequate. The head was turned inward instead of outward (as recommended by the undersigned for Tetra Tech and Parsons in 1975). The armour blocks were too small. If the head had been turned outward as recommended, the damage, without any doubt, would have been less. If the available equipment was unable to handle heavier blocks, the blocks could, in an emergency, have been tied together by chains or cables, as used in breakwater heads in Northern Norway with good results. Other patents exist for tying blocks together.

REASONS FOR DAMAGES BY WAVE ACTION

General - Damages to breakwaters are, with the exception of such damages, which are caused by seismic action or soils-breaks or both, always caused by wave action, which is the main criteria for the design of breakwaters of any kind. It is therefore mandatory that adequate wave data are available for design.

Ref. 6 mentions "Rationel Engineering Design" and its Table is reprinted below as Table 4.

Table 4. BREAKWATERS ENGINEERING-RATIONAL DESIGN PRINCIPLES (ref. 6)

by Per Bruun

GOAL: A stable and economic design, all factors considered. Minimum adverse effects on the environment. Design shall be practical in all respects, easy to build and easy to maintain.

EARLIER AND PRESENT PRACTICES		NEW PRACTICES
Data used:	Wave spectra Extreme wave heights "Design wave"	Data to be used: Materials and equipment available for construction. Wave conditions of most dangerous character for stability of structure. Probability and risks of occurence of such conditions.
Procedure:	Empirical	Design Procedure: Optimization of

Structural design based on formulas and models considering armour layer, uprush and overrun only. Soils investigations limited to foundation conditions. Construction procedures and conditions usually neglected in models. No operational data for construction secured.	Introductory mathematical design models with inputs from all data secured and comparative experiences, if possible. Hydraulic models based on the results of physical analyses including structural evaluation of overall as well as unit stability breakdown patterns, maintenance evaluation, interior and exterior soil mechanics. Constructional and operational criteria: Period of construction, risks and overall economy.
<u>Result:</u> Contractors may suggest their own design based on their own experiences and equipment and possibility of successful competition. Unreliable time schedules during construction. Unexpected, erratic breakdowns. Excessive maintenance. Undxpected influence by structure on the environment.	<u>Result:</u> Optimized design of maximu stability and economy, all aspec considered. Reliable time schedules. All influences on the environment pr dicted, considered and accepted with the necessary corrective steps.

RESEARCH AND ENGINEERING

<u>Hydrodynamics:</u> Wave conditions criteria for stability and operatic

<u>Structural:</u> Overall stability
Unit stability
Maintenance
Economy

<u>Soil Mecanics:</u> Exterior as well as interior stability.

<u>Environment:</u> Influence of structure and construction on the environment.

<u>Construction:</u> Practical.
Adherence to design, capacity and time.
Controls before, during, and after constructions.
Coordination.
Reserve capacity available.

<u>Future expansions</u> must be considered and discussed.

It distinguises between "Earlier and Present Practices" and "New Practices". It calls to the attention, as mentioned carlier in ref. 2o by the International PIANC Committee, that "Spectra" and "Spectral similarity" is inadequate for design. Refs. 8, 13, and 22 and Appendix 7, Chapter 7 give considerable information on this topic and advise that "wave conditions of most dangerous character for stability of the structure" and "Risk Analyses" shall be used for design, (refs. 8, Appendix 7 and ref. 12).

Ref 8, App. 7, give in Table 3.9 p 219 an example on how such analyses may be carried out considering wave steepness and their probabilities. At this time (1981) several results of research on "Wave Grouping" (refs. lo and 22) and "Jumps in Wave Heights (ref.lo) are available. It has, however, been some difficulties with dissemination and publishing of such information for general use. As an example: a Dutch report from 198o on a large breakwater in Mexico does not include "sequences of waves". The planned new tests, however, will include such waves. Special wave groups were observed in the records for Siñes in Portugal by Bruun and Grünbak. Tests have been and will be run on Siñes using such groups in Canada as well as in Holland. Likewise special sequences were observed in Iceland, and tests on the Akraness damage are run with such sequences.

To give the reader a complete picture of the necessity of running such tests, Appendix 3 was produced in accordance with discussion, which took place during and after the 5th POAC (Port and Ocean, Arctic Conditions Committee) conference in Trondheim, Norway in 1979 (part of material, 198o). Reference is made to the Proceedings from the POAC conference, published by the Norwegian Institute of Technology in 198o and to Proceedings of the Breakwater Seminar at Santander, Spain, 198o. A copy of the Santander Proceedings, which is referred to in several instances in this report, is enclosed as Appendix 3.

WAVE CRITERIA FOR TESTS ON THE ARZEW EL DJEDID PROJECT

The situation is that tests on the breakwater were run by DHI before or in 1977. It was in 1976 that PIANC first published its radical new recommendation for the design of mound and other breakwaters (ref. 2o) and it took quite a while, before these new principles were known and designers started to follow PIANC's recommendations.

These new principles are fully realized by DHI, but at the time of the DAREP no analyses of data following the new design principles were available, consequently no testing accordingly. This, however, is suggested for the tests to follow.

SUMMARY ON REASONS FOR DAMAGES AT ARZEW EL DJEDID

THE COLLAPSE of a breakwater of multilegged or other blocks may have WAVE MECHANICS, STRUCTURAL or MIXED WAVE MECHANICS and STRUCTURAL REASONS;

re: WAVE MECHANICS REASONS:

(1) Although tested with waves recorded from a bouy located some distance from the breakwater the data may not be fully representative for the wave condition occurring all the way along the

breakwater right in front of it. Smaller irregularities in bottom topography may at some points in front of the breakwater cause combinations of waves, which are higher or longer than the recorded waves. Refraction analyses by DHI have shown that waves may be 5% higher at some places along the breakwater.

(2) 3-dimensional effects may also arise due to the history of the storm, which may cause very peaked, high, short-crested waves - as it e.g. happens in the North Sea. To account for that model experiment should also be carried out 3-dimensionally. Combined wave action may be determined by hindcasting of a storm's time history, and 3-dimensional tests are therefore also included in the further test-program.

(3) Extreme wave height tests, when the extreme heights and their distribution must be determined, are necessary. The 1976/77 DHI reports use a logarithmic extrapolation. The DAREP extrapolates by a Gumbel distribution, which is used for probability analyses to determine exeedances, but not for design of sequences of particular dangerous character, as detailed wave analyses needed for such design, have not been undertaken - so far.

(4) The storm which occurred, may also, as a whole, have bypassed the recorded storms, incl. the "design storm" in strength, because recording period was too short. Two or three years of recording, of course, is not enough for a fully reliable statistic. According to Fig. 4.1 of the DAREP the storm of Dec. 28-29, 1980 seems to have exeeded by approximately 1 meter other storms on record since 1976. But it corresponded to the hindcasted storm of 6,5 m (Hs) with 14% probability for 5-years-occurrence(Table 4.5, p 62), for which tests have been run. (Fig. 4.8, p 67) with very limited damage only.

(5) The occurrence of special dangerous combinations of waves including long "mountenous" waves, one or two, a phenomena, which recently have become known, must, however, be considered and included in the test program; so must "jump in waves" (ref. 10). Their effect is a lifting of the armour layer, weakening its stability for any wave, which arrives immediately after the long wave. The long waves cause a decrease of the friction force between armour and the 1st. sub-layer, so that it gives way or slides easier. Furthermore it generates a deep downrush. There is no direct proof that such waves of special and very dangerous character have occurred but they appear in wave records e.g. from Siñes, Portugal, Dos Bocas, Mexico, and Akraness, Iceland. Reference is made to recent (August 1981) papers by Bruun and Moe (ref. 9) and by Plough (ref. 22).

As already mentioned it has been decided to analyse existing wave records with respect to "special waves". The results will then be used for further testing.

STRUCTURAL REASONS FOR DAMAGES

Overall Effects - The design, which is tested, is supposed to be followed in the construction. The question always is, whether this was done in practice. As mentioned above the DAREP speaks about deficiences in qualities as well as qualities of the material found in the breakwater.

A common reason for damage is that the porosity of the 1st. sub-layer is too small compared to (or smaller than) the porosity of an armour layer. The result is that static pressures, which build up in the mound due to wave uprush, now become higher than assumed or accounted for in the design, and this has an adverse effect on the stability (ref. 1, Appendix 2). The porosity of the Tetrapode armour is about 45%. The question is, whether gradation or placement of the sublayers makes the porosity smaller than it was supposed to be. The B1 layer had often too small grain sizes, and B1 material may have penetrated into the B4 and B5 layers. The two classes of rock are found mixed.

The sublayer is also designed to be placed in an even layer with little variance in thickness and with a plane slope. So is placement of the armour layer supposed to be. The obvious question is: Was this actually done ? In the case of damages, were placement-procedures the same throughout construction, or were they changed? Certain changes in the placement of blocks were made during the construction of the Arzew el Djedid breakwater (more "regular" to "pell-mell"). This raises the question of, whether it is possible to trace any connection between change of procedures for placement and the damage, which took place. The situation is that construction procedures for the middle, heavy damaged part of the breakwater actually were changed as described above, but it is not possible to give any definite answer to the possibilities of further damage for that reason.

Another question is: Did the contractor use quarry rock to even "beautify" the surface of the 1st sublayer, thereby decreasing its porosity and making it smoother, in this way also decreasing the resistance against sliding. There is no evidence of that, however. But compatable porosities between layers are very important.

With respect to the armour, it is mandatory that it is well placed in an even layer with an even slope. If this is not done well enough, 3-dimensional effects may arise, which will influence the stability adversely. The possibility, of course, also exists that an adverse wave condition occurs at the same place, where placement of armour and/or sublayer is less adequate. If the 1st sublayer is too smooth, sliding could take place, if the mound is attacked by a long wave which, by swamping of the mound, makes friction between armour and sublayer so small that failure as one compact layer may result. Required layer thickness, of course, must be respected as well as grain size distributions. In this respect the DAREP report gives some adverse information. As mentioned earlier it is stated several times (p 26, p 28, 29) that rock size is smaller than stipulated, mainly for the B1 and B4 fractions.

The HC large size fraction was sometimes hard to find (p 35), and some of the rocks were fragmented (p 32). As the coverlayer of Tetrapodes rested on B4, which rested on B1, it may have been leakages of rock B4 through the Tetrapodes and possibly also leaks of B1 through B4. At the same time the B4 layer presented a smoother surface against the Tetrapodes than assumed. This circumstance has - at least - weakened the design. Photo 8o may also leave an impression of poor contact between armour and first sublayer.

The DAREP describes, how difficult it was to find remnants or evidences of the toe built up of armour HC.(on B2 and B1) materials (p 3o, p36). This, of course, may be a result of the general collapse, by which HC rock was covered up by other material, including Tetrapodes and B4 and B1. A single remark on p 3o "Au profil 15A, une roche HC fut trouvée cassée en 5 fragments (voir photos 57, 58 et 65)" may be descriptive of the situation, actually found. It is difficult to place a toe of the dimension recommended and therefore likely that the "theoretical profile", Fig. 4.5, has remained "theoretical". This happens quite often. It was, however, difficult to get larger rocks, particularly HC, from the quarry, as mentioned earlier.

There is no sign of slip circle failure, neither of any deep toe scour.

The DAREP mentions several times deficits in the amount of materials Strongest this is expressed on p 69 as: "L'étude des profils très endommagées d'après les reconnaissances effectuées par la SONATRAM a indiqué un grand déficit de matériaux (roches et tetrapodes) par rapport au profile théorique du projet".

"On remarque également n'ont pas trouvé durant leur inspection du côté exterieur du brise - lames de roches HC (plus de 1o tonnes)".

"Ce déficit en materiaux ne pouvant à peine s'expliquer par l'effect de la tempête, les conséquences d'un tel déficit pour la stabilité de l'ouvrage doivent être examinées au cas où le déficit existait avant la tempête."

"Partant de cette hypothèse, le profil No. 1 (Fig. 4.9) est supposé représenter la configuration de sections actuellement très endommagées avant la tempête de fin décembre 198o."

The difference in survey methods may have played a role, however.

The conclusion of the tests run on profile Fig. 4.9 (p. 7o of DAREP) reads: "Le dernier profil testé (avec deficit de materiau) est moin stable que le profile théorique" - "Il n'est pas probable qu'un deficit en materiaux soit la seule et peut-être même pas la principale cause des dommages occassionnés a l'ouvrage".

The deficits mentioned are undoubtedly in part a result of some settlings, which could be expected under all circumstances. They are, however, sometimes of such magnitude that the question arises of, whether the materials have been available in the profile as delivered upon the end of construction. This question can only be answered with reference to the profiles surveyed immediately upon completion of the work. Some settlings - of limited order - will always take place, however.

If material washed out below the armour, settlings would take place (as evidenced clearly from the surveys, incl. Photo 8). On p 15 it is described, how "après la tempête, environ 12o tétrapodes disponible ont été posé sur la partie Est de l'ouvrage".

The wave screen in itself has been responsible for the scour in front of it, (exactly as in the cases of Siñes, Portugal, Bilbao, Spain, and Akraness, Iceland) with the following collapse of the screen and heavy damages to the crown in Siñes.

As it has been impossible to investigate the condition of the core-fill, there is no other answer to the question than the supervision's decisions or approval of the core material used.

An indication of a substandard core fill may be traced by observing the place, where settlings start. Settlings will come first, where water movements caused by fluctuations in pressure, are maximum, and this is where uprush/downrush give maximum deviations in pressure. In that case this could happen partly in front of the wave screen and partly, where downrush penetrates deepest. This happens, as mentioned earlier, when uprush/downrush period is equal or close to the wave period, or at "resonance" (ref. 1, Appendix 2, refs. 2, 8 (App. 7) and 2o).

Structural Soundness of the Armour Layers of Tetrapodes

at Arzew el Djedid - The DAREP mentions the extensive damages on Tetrapodes, extending from the top of the mound (photos 8, 1o, 11, 12, 13) to the lower slope and base of the mound (photos 5o, 51, 56, 57, 58) and numerous profiles, example: Fig. 28, p 26 says: "Malgré qu'il y ait une double (et quelquefois triple) couche de Tetrapodes sour toute la longueur du profil, il y a de nombreux signes de dégâts. Le pourantage de Tetrapodes endommagés dans la couche supérieure va jusqu'a loo, et diminué toujours dans les couches inférieures (par example o% sur le profil 7oB, 5% sur le profil 69B, 3o% sur le profil 94). - Further: "Un grand nombre de ces Tetrapodes ont glissée le long du profil en laissant des cornes encestrées plus tout sur le talus. D'autres montrent des signes de dégâts évidement causées par le choc de corps solides en mouvement. - La première sous-couche sous les Tetrapodes comporte des signes de mouvements qui ont en lieu après la pose des Tetrapodes".

These findings explain the mechanics of the breakdown. Blocks were lifted out - some perhaps after having been exposed to chocks - of the mound and rolled down. During this process they were exposed to strong inertia forces and bumped against each other and against the sublayer. Some blocks apparently were damaged before the storm (normal for a Tetrapode-mound). Other blocks bear wittness of having rubbed against each other. This means that they were not large enough to avoid movements - before the storm.

Multilegged blocks, including Tetrapodes have failed in other mounds, e.g. Tripoli (Tetrapodes), Siñes, and St. Cyprian (Dolos). Wave forces have been the main reason in the two former cases, while concrete qualities seem to have played a role in the case of the St. Cyprian breakwater. There is no report on substandard concrete at Arzew el Djedid.

CONCLUSION REG. UNIT STABILITY. ARZEW EL DJEDID.

Recent failures of single structural elements in mounds of multi-

(1) The concrete in the blocks was actually too weak and below required concrete quality. - No evidence reported at this time.

(2) The blocks have been mistreated during the placement, which resulted in too many cracks and breakages. - There appears to be some evidence of that. Dumpings or chutings have been too hard in some cases.

(3) The blocks were too small and therefore were subject to movement causing bending forces, which broke the blocks. - There is ample evidence of that.

(4) The blocks were not strong enough to carry the load of overlaying blocks in situ. This is a severe, but actually occurring reason, for failures. - There is no evidence of that.

(5) The blocks, although they were strong enough to carry the load in situ, were unable to carry the load by combined unit weight, whether boyant or not, plus the load caused by downrush that exerted pressures on a large number of blocks, which possibly were lifted up high enough to disrupt any intimate connection with the sublayer, thereby increasing compression forces due to loss of frictions. - This may be a main reason for the failure at Arzew el Djedid. So it is at St. Cyprian in Spain.

(6) Blocks were, during placement, put in jammed positions due to misunderstood attempts of making the armour layer more dense. - This happens always and it is also evident in this case, but not excessively.

(7) The materials, which were used for casting of the blocks, included chemical elements with deteriorating effects on the cement, which gradually was dissolved, resulting in disintegration of the concrete. - There is no evidence of that.

Structural Soundness of Sublayers and Core Fill in the Breakwater at Arzew el Djedid - The settlings and movements of the outer slope, and to some extent also the inner slope of the breakwater, may be taken as an indication that inadequate grain sizes may have a responsibility for the collapse with respect to armour as well as sublayers. This holds true, whether defiencies in quantities of materials are correct or not.

The questionable soundness of some materials used will of course have contributed hereto (p 26, p 32, photos 39, 41, and 63). Some of the rock materials, however, as mentioned earlier, suffered breakages during placement due to too hard dumpings (chuting). This happens quite often. Core fill often includes too many fines, which gradually may wash out. Such wash-out may have happened in this case during the collapse stage, but does not appear to be predominant.

CONCLUSION ON REASON FOR DAMAGES AT ARZEW EL DJEDID

Damages happened during a storm on Dec. 28-29, 1980 described by DHI in preliminary report of June, 1981 and in the DAREP of July/Aug., 1981.

Model tests performed before the structure was built showed that the design - if built according to plans - would withstand wave action as recorded, or predicted.

Model tests performed recently were based on hindcast wave data from the Dec. 28-29 storm and extrapolated data as described in the DHI progress report and in the DAREP. - Models on particularly dangerous waves, incl. deep throughs and high long crests, as well as certain sequences of large waves, have not been undertaken at this time. Some of these waves may, however, have been "hidden" in the spectra used for testing. Based on all available information the following possibilities for damages seem to exist:

WAVE ACTION

A. Inadequate wave data. The time period of recording has simply not been, and could not be, long enough to develop a fully reliable long-term and extreme height statistics for storm wave action.
B. The place of recording may not have been fully reliable in respect to representation of data along the breakwater, when wave action, most likely, will undergo some changes. So far little deviation (ab. 5%) has been estimated by diagrams.
C. Long wave effects, and the effects of dangerous sequences, according to available analyses.
D. 3-dimensional effects, which according to experience may be very detrimental to the stability of multilegged blocks in particular. Superficial placement could also cause 3-dimensional effects of adverse character.

STRUCTURAL STABILITY. OVERALL

E. Variances in thickness of the armour layer and/or poor placement of blocks. There is some evidence of that.
F. Poor friction connection between armour and 1st. sublayer. - This is likely to have occurred, as 1st. layer as well as 2nd. sublayer were smaller than they were supposed to be.
G. 1st. sublayer has major variances in thickness, friction and/or porosity. This may be caused by changes in construction practice during the construction. The question: "Is there a dependency between damages and construction work as it "proceeded" - has not been answered at this time, but changes of construction practice did take place.
H. Is there any sign of damage to the toe ? If this is the case, the toe may have been week in the start, or the toe was damaged, when the upper part of the structure collapsed. Is it possible to see if the toe is still intact-regardless of damages to the upper structure? Is it always OK at the non-damaged section of the breakwater ? The question may be answered as follows: There is little evidence of the existence of a toe-structure, but the toe may have been buried in debris from the collapse, or it may have vanished during the construction, simply because of its (too) limited size
I. Is there any sign of settling of the structure with particular reference to the upper part of the slope, by which the wave screen was exposed on a section high enough to cause frequent reflection of uprush, thereby damage to the armour. If settlings have taken place, the reason could be: a) wash out of too small particles included in the 1st. sublayer or b) core fill has included too many fines, which washed out through the 1st. sublayer. It is

(a) initial deficits

(b) outwash of materials, mainly sublayer and core fill.

The surveys made demonstrate that some initial deficits are likely and that some materials may have been washed out through the Tetrapode armour, as 1st. sublayer material was smaller than required. Surveys after completion of works, however, are not available.

STRUCTURAL STABILITY. UNIT

Regarding unit soundness the situation is that many broken armour blocks have been found. There are 3 possibilities for these breakages:

1_1: blocks were already broken or damaged during placement. - This undoubtedly has happened. It always does.

1_2: blocks broke in situ due to compressive and bending forces caused by their own weight. Such breakages, if they have occurred, may be observed in the structure which is left intact. - There is some evidence of that. This reason probably was the main reason for the failures in the St. Cyprian breakwater in Spain (Dolos blocks).

1_3: breakage due to forces by waves causing sliding and collapses combined with in situ pressures in the mound. Such damages will reveal themselves by the large number of broken blocks, which are found in the rubble deposited on the lower slope and on the bottom right in front of the breakwater. - There is all signs that this is the main reason for the failure.

1_4: defects in the concrete, which can be observed in broken as well as in blocks, which are still intact, Shortcomings in the strength of the concrete will always reveal themselves in their distribution in the mound of damaged blocks Where forces are maximum for structural reasons, the number of damaged blocks will also be maximum. - There is, at this time, no evidence of that at Arzew el Djedid.

1_5: a very sneaky reason for damages is the deterioration of cement caused by adverse chemical reactions between the cement and certain rock materials used for the manufacturing of the concrete. There is no reported evidence of that in the CN report other than the composition of the rock which was fractured with some less substantial chemical deposits.

From the above mentioned it may be seen that wave forces, probably exeeding design forces, attacking a structure, which in several respects was too sensitive to major wave action and not built according to required standards, must be charged with the main responsibil for the collapse of the breakwater. The flaws in rock material, si as well as soundness, probably played a minor role compared to the wave mechanics reasons. Although the structure - as delivered - was weakened compared to the design, there is no direct proof that it, if built as it was supposed to be, would have withstood the forces. It is, however, certain that damages, if any, undoubtedly would have been less, if the design had been followed. From the construction angle it seems that supervision may not have been strict enough.

Reference is made to Appendix 6 and Appendix 3 by Bruun and Kjelstrup. (POAC, 1979, Trondheim, Norway).

BRIEF REMARKS ON THE TEMPORARY DESIGN

A great number of experiments have been run as mentioned in the DAREP and a number of suggestions have been proposed and evaluated. The criteria for such repairs must be:

(1) rapid execution (before the storm period)

(2) reasonable practical safety and costs

(3) repairs as close as possible for integration in the anticipated final design.

Please note: p 85 DAREP, "Pendant ce temps le risque d'une progression des degats et meme d'une destruction totale est trop élevé, et il importe de prendre des mesures immediates pour preservation de l'ouvrage, meme de facon provisoire".
Requirement No. (1) seems to be fulfilled, as works have started at this time (Sept. 1, 1981).
Requirement No. (2): Type E is recommended, (p 88). - As described in DAREP suggestion E has a platform of 16 meters at elevation -9 meters. According to laboratory experiments, "on peut conclure que le profil E a une résistance acceptable jusqu'à une grandeur de houle Hs = 5,5 m environ. Cette houle a une probabilité d'occurrence d'environ 13% durant le premier hiver et de 25% durant les deux prochain hivers", (p 81).
With reference to the above mentioned regarding waves of particular dangerous character, these probabilities may have to be checked, when tests specifically have been run with such waves or wave sequences, which may be hidden in the spectra. Regarding item (3), it is, as mentioned earlier, the opinion of the author that it may become practical to lower, or to turn, the berm (ref. profile E), making it more compatible with the investigated breakdown profile, by sloping it from about -5 m to about -8 to -9 m along to about 16 m berm - or slope 1 in 5 rounding it down at the lower "edge" and rounding it up at the upper "corner". Averagely the berm is thereby lowered 1,5 m and turned abt 12 degrees anticlockwise. Such design must be handled in a practical way, however, and the situation is that it has to be at least 2 layers of heavy rock on the top of the sublayers. This may, at some places, of necessity cause some variances in the design. In practice the profile, by the construction procedure, will by itself attain an S-shape, bending down at the outer "edge" and bending up at the inner "corner"; thereby avoiding "brittle edges" in the lower slope and decreasing the upper slope angle. This will be a definite advantage, resulting in a more practical and stable profile as a whole. No reason to build something, which we know that nature will change soon.

The temporary profile should, of course, be such that it, most easily, becomes an integrated part of the final profile at the reconstruction following the temporary repairs.

A couple of details should be mentioned. Cavities, of course, should be closed. In Norway this is often done by explosives, if the work by crane becomes too laborious.
With respect to the repair of the scour right below the wave screen an asphalt concrete or mortar is suggested to be pumped down (after the triangle in front of the screen has been filled) to form a dense but flexible wall. Behind the wall grouting of small cracks may be done by asphalt mortar, larger cracks, or openings, by concrete. The bitumenous wall right below the wave screen then functions as a mould for the concrete mortaring. A similar wall was used at Sines, Portugal, by grouting cement mortar through holes drilled in the slab behind the screen. Cement mortar, however, tends to become rigid and may thus crack. For further repair works at Sines asphalt has been recommended.

BRIEF REMARKS ON THE FINAL DESIGN

During the latest 5 years a number of severe damages have happened to breakwaters built during the latest 5 to 1o years and extending into deep water (1o to 25 meters). This includes:

Bilbao, Spain, Accident 1977	parallellopedic blocks, also recommended for repairs (9o ts). Models in Norway for final stage.
Sines, Portugal, Accident 1977-1978 [illegible]	42 ts DOLOS It is not likely that Tetrapodes will be used for the final design. Blocks will be rectangular or cubed. (P. Bruun, member of commission). Model tests at Dutch Laboratory planned. Temporary tests at Canadian Laboratory.
St. Cyprian, Spain Accident 198o	52 ts DOLOS Replacement by about 9o ts parallellopedic blocks recommended. Model tests planned.
Tripoli, Tripoli Accident 1977 [illegible] 1978 [illegible] 1981	21 ts TETRAPODES This case is now being investigated by a group of experts. DHI and P. Bruun report delivered already in 1979.
Akraness, Iceland Accident 1981	4-5 ts Rock Replacement by 6-8 ts Rock of volcanic origin. Improved profile with wide berm-to

Presently under design (under the supervision of the author) are 3 projects for industrial ports in Mexico: Lazar Dos Cardenas on the Pacific, Dos Bocas and Ostion on the Gulf. The following type of blocks will be used:

Lazar Dos Cardenas:	Parallellopedic or cubed blocks, perhaps with grooves (Antifer). Design will be based on experiences utilizing tests in Norway, Holland, and France as "models".
Dos Bocas:	Cubed blocks with grooves. Model experiments in Holland. Block size about 30% of Antifer-type.

Ostion: 'Cubed blocks with grooves. Model Experiments planned in Mexico.

In all cases the use of multilegged blocks has been discarded due to recent bad experience with these blocks. This does not necessarily mean that the blocks are bad in themselves, but could as well mean that they were used beyond their limitations. But it should still be remembered for overall as well as unit stability that "the strength of a chain (structure) is the strength of (each) its weakest member".

I would, however, suggest for consideration for final design at Arzew el Djedid that reconstruction of the breakwater be based on cubes or parallellopedic blocks, because I do expect that waves of about 1o meters can (will) occur, and for such waves the more sturdy not fragile, blocks are preferable. The mound may e.g. be built with a straight, e.g. 1:2 or 1:2,5 slope of parallellopedic blocks or a 7/5 slope of Antifer grooved cubic blocks with substantial toe, or - perhaps best - with a combined slope of upper 1:2 or 1:2,5 and lower 1 in 2 or 1:1,5 slope with a berm of about 2o meters in-between. Reference is made to Fig. 2 (Fig. 28 in the enclosed Appendix 2). Armour blocks, of course, must have adequate size and I would always include a W/2 layer between the armour and the 1st. sublayer (Appendix 2, ref. 1, Appendix 7, Chapter 3, ref. 8, and as now recommended by the US Army Corps of Engineering for navigation breakwaters in the US). See Appendix 7, Chapter 3, pp 195-223 for details. Sublayers must be of substantial rock and fulfill Terzaghi's filter ratios. This was probably not the case at Arzew el Djedid. The question of proper and reliable modelling is mandatory. Appendix 6 describes "Waves for Model Experiments" based on adequate wave data (Appendix 4 and refs. 7 and 22). Three-diemnsional tests should be included in the final testing program (ref. 28). "Load factors" may be considered to provide a safety margin (ref. 9) and tests should be carried through the failure stage to investigate modes of failures (Appendix 3 and 5).

Placement of blocks must imitate prototype conditions as closely as possible (ref. 16) and "design and construction procedures" should be included in the experiments (ref. 23). Throughout model scale effects should be evaluated (refs. 19, 26, 27) as part of tests and reports. Optimum design may be attempted already in the model stage (ref. 5), which may include risk analyses based on existing data and the new model experiences (ref. 8, Appendix 7).

The author finds reasons to repeat, what was mentioned earlier in the report about the severe collapse of the extreme ends. In about 1975 he recommended for TETRA TECH and PARSONS that the extreme ends of the breakwaters for reasons of safety of the structure be curved outward towards the open sea (like Fig. 7). Recommendations were not followed. The result is seen today.

September 5, 1981

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for

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REASONS FOR DAMAGES TO THE ARZEW EL DJEDID BREAKWATER

DECEMBER 28-29, 1980

August-September, 1981
Per Bruun
Dr. techn. Sc., Denmark
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MOUND STRUCTURES

DISCUSSION ON STABILITIES

Per Bruun

INTRODUCTION.

During the last decade considerable progress has been achieved in the design and construction of mound breakwaters. The reasons for the progress are:

1) The work by the "Waves Commissions" of the Permanent International Association of navigation Congresses (refs. 1 and 2).

2) Massive failures of mound breakwaters including Siñes (Portugal), Bilbao (Spaña), Arzew el Djedid (Algiria), Tripoli (Tripoli), and St. Cyprian (Spaña). In all cases the armour layer was concrete blocks. These massive failures may be traced back as a result of inadequate wave data, inadequate and impractical laboratory experiments, certain neglects in the design providing inssuficient friction between layers and/or insufficient permeability, the lack of a proper toe structure as well as/or to lack of a botton protective mattres

This paper analyses the components in a stability condition, reasons for failures and formulac - procedures.

STABILITY ANALYSES

To stay stable a mound structure exposed to wave action must fulfill the demands to overall stability as well as to unit stability. When mound structures were largely built of natural rock the interest was turned on overall stability and the stability of the single unit against movements. The introduction of concrete elements carried with it requirements to structural strength and health of the unit. Recent years experiences have proven the futility of ignoring structural analyses of more detailed nature. Based on bitter experience it is now known that damage to mound structures often is a chain process by which failure of one element introduces a chain of failures. The stability of the single element therefore becomes of primary interest for the stability of the entire structure (refs. 5, 7, 9 and 12). To obtain stability one therefore has to consider three different kinds of stabilities:

i) The overall stability, which is the stability of the breakwater as a whole with special reference to the armour layer.

ii) The unit stability which is the stability of the single unit or its ability to stay in place.

iii) The structural unit stability which is its structural strength.

These stabilities which all must be fulfilled are interdependent. Failure by ii) or iii) or both may cause an i) failure. Failure i) may initially occur without ii) or iii) but it may cause failures ii) or iii) or both in the failure itself. Heavy damages often tale place when the armour is completely soaked or fluidized in wave uprush. The breakdown pattern therefore may have the character of a catastrophical event which moves large masses (refs. 8 and 9).

Common reasons for breakdown of rubble mound breakwaters whether the mound is composed of natural or artificial brocks are depicted in Fig. 1 (refs. 7 and 9). They include

1. Knock-outs by plunging waves when $\xi = \tan\alpha\sqrt{H_D/L_o} < 2.5$ but > 0.5 (refs. 6 & 12).

2. Lift-outs (by uprush-downrush) usually resulting from combination of uprush and downrush and toe velocities in an arriving plunging wave. Professor Koutitas (ref. 18 in press) has made extensive theoretical studies of this subject.

3. Slides of the armour as a whole. This happens in particular at steep slopes which are subjected to high waves of periods close to resonance (that means uprush-downrush period is close to wave period), see refs. 6, 12 and 14.

Failure is caused by combinations of bouyancy, inertia and dragforces supported by the effect of hydrostatic pressure from the core of the breakwater. These forces all seem to reach their maximum value for lowest downrush which occurs at reasonance or for $\xi = \tan\alpha\sqrt{H/L_o} \sim 2.5$ (refs. 6, 12, 14, 18). Experience, however, has shown that large single or double waves (fig. 2a) may be particularly dangerous. This has been

observed directly in the field and some of the large failures of multilegged blocks may be attributed to the occurrence of such waves or groups of such waves.

4. Gradual breakdown or failure due to "fatigue". Fatigue starts with smaller movements of the blocks which gradually increases and by which the block(s) gradually is moved out of intimate contact with their neighbour blocks or from the first sublayer and perhaps simultaneously suffers from tear and wear due to their rocking or bouncing around, hitting other blocks damaging themselves and them. This is in particular of importance for multilegged blocks, when such damages may be directly observed or "heard" in coming. Occurrence of resonance making the uprush/downrush period equal to the wave period for groups of waves (Fig. 2b) has in particular damaging effects due to the continued rocking, which partly breaks down friction and inter-

knitting between blocks and partly cause structural ruptures due to bending stresses and other fatigue forces (refs. 7, 9). Other types of wave trains e.g. wave series with deep trough, Figs. 2c and 2d, causing deep run down, Fig. 2e, and therefore high downrush velocities as well as higher hydrostatic pressures from the water table in the core are very dangerous (refs. 6, 9, 12, 14). Natural rock is a compact mass and its resistance against movements is its weight and friction against other blocks. When useful weight is decreased due to buyance the resistance against movements decreases to about half. The wave situation depicted in Fig. 2d, therefore, is very dangerous because of the deep downrush with slope submerged. It is the most dangerous wave trains, hydrodynamically speaking, which determine the stability - or failure -. It is a too often repeated mistake to base tests on "spectra" without regard to sequences of waves. Laboratory people have claimed that they were included in the spectra but after the fact, and without delivering proofs for that. They most likely were not, because generators could not produce them. Sometimes it was claimed that such waves did not exist but this postulate was based on a limited time recording at only one place. It is also said that waves as shown in Fig. 2 are shallow water phenomena. Experience, however, shows that this is not correct either. They have been observed in the middle of oceans during storms, and are generally known as "freak waves" by sailors. The local wave mode or pattern varies along the breakwater and it is of course the local waves situation which determine the local forces and it may include concentrations of wave energy. For a long breakwater it is therefore necessary to know the wave situation for the entire distance along the breakwater. Breakwater failures often happen in ereas, where wave action for some reasons, which could be the bottom topography or wave interactions, concentrated. Conventional head geometry by turning the end inward creates such concentrations. It is therefore much better for stability as well as for reasons of navigation and sediment transports to curve the head outward (ref. 9 This is an old Scandinavian experience now being utilized more and more elsewhere.

Undermining of the wave screen or upper solid structure. It is common practice to provide the crown of a rubble-mound breakwater with a wave screen. This may be a solid or block concrete structure, which shall arrest the upper part of the uprush and turn it back towards

the ocean. In doing so the wave screen may be subjected to large horizontal and vertical forces that means overturning moment shear forces as well as forces directed down against the slope. Overwash and solid oversplash may be avoided by a - mostly excessive - top elevation of the screen. A vertical, or slightly curved wall will, however, always cause a downward directed force, which will try to dislocate the upper rock or block layer, thereby undermining the wall which in turn may collapse seawards. This is a common failure at walls in the Mediterranean as well as in the Atlantic.

Overwash by solid water always presents a danger to the stability of the crown as well as to the inner slope (Fig. 1). Many failures started as crown failures or failures of the upper part of the inner slope. This demonstrates the need for a strong design at proper elevation for the crown as well as for the inner slope. Model studies using irregular waves as well as wind are of great value, but it should be remembered that scale effects and two-dimensionality may cause non-conservative results on uprush as well as stability. This has been demonstrated by various accidents and fatalities during recent years. In all cases laboratory equipments were inadequate

Lift-ups happen and through-washes when the core material and/or filter layer are so coarse that they let uprush water pass through or below the wave screen, exerting heavy uplift forces on the wave screen and structural components behind it (Fig. 1). This may result in failure of the superstructure or of the inside or the middle section, sometimes producing a crater in the crown or in a slide on the inside. Crown blocks of sufficient weight, a watertight partition wall below the wave screen grouting, and vent holes include mitigative preventive steps.

It is unfortunate that it sometimes happens that the designer "makes excuses" for deficiences in the upper part of the core material by providing only "venting" by holes in the crown block or slab. Venting through relatives small holes is usually going to be minor compared to the actual needs for release of the wave-uprush induced pressure. The best is to make the sub-layer below the wave screen impermeable for water and air which may be under high pressure. It is a professionally wrong philosophy to

believe that water and/or air in quantities shall be allowed to pass through the upper part of a mound breakwater, particularly when it has a fill or other structure behind it. This has many drawbacks. Some of the most important ones are listed below:

(a) Endangers the stability of the superstructure as well as the inside slope.

(b) May burst any pavement or any screen material inside the superstructure, causing sparking springs or "geysers" damaging road pavements etc.

(c) May cause faster deterioration of the material below the superstructure by waters rushing in (uprush) and out (downrush); the superstructure may then turn anti-clockwise; diagonal ruptures may appear in the slab.

(d) May damage the upper part of theccore material below the coarse layer, which was placed below the superstructure; this layer may be a continuation of a 2nd- or 3rd-order armour layer: a by far too often occurring severe mistake on the part of the designer, who doe not realize that he is not only releasing some water but is building up pressures below the crown slab. Such pressures may be as high as lo ts/m^2 and therefore need a heavy lock to block them. It is therefore better to prevent them.

Toe erosion is also a common reason for failure at the lower part of the seaward side of a rubble mound which is placed in the shallow water or where the depth/wave height ratio is less than 2.o. Waves are then close to breaking and with standing waves which particularly occur for waves of long periods erosive forces may develop severely. The worst case, however, is when downrush from the mound penetrates down to the bottom and a longshore current exist at the same time. The measure against this is a toe apron of rock placed on a mattress extending far enough out to prohibit direct attack by downrush on the bottom (ref. 2). Model experiments often tend to forget this effect, which can be accounted for even in fixed bed models. It is a severe mistake which has contributed to major fatalities during recent years. Toes were too weak and after the disaster they could not even be found.

In coastal protection revetments with gentle slopes it is common to place a sheet piling to support the lower slope. Longshore currents may furthermore necessitate the installation of short "spur groins" along the toe wall. Sometimes, it is in this way possible

to obtain and maintain a small beach in front of the revetment, particularly if the spur groins are built as T-groins (ref. 9).

Soil failures. It happens sometimes that a breakwater has to be built on a soil which is not very strong. It may include soft silt layers with a high water content and thereby a low bearing capacity, which may cause turnover as well as sliding on or squeezing of the soft layer. Soils cannot be said to have a large responsibility for recent failures because soils engineering - contrary to wave engineering - has been handled more adequately in the design.

Discrepancies of the soundness of materials used for the construction. Natural Materials when quarried demonstrate strongly varying characteristics with respect to size, geometry, hardness, wear by rubbing against other blocks, resistance to shifting conditions of submergence and emergence, temperature variation, freezing and thawing etc. Various countries practice varying rules or standards for materials testing, some being more rigid than others. Generally it may be said that high specific gravity materials like basalts are preferable if they are relatively easily quarried and give a reasonable return of large blocks without too much differences in sideline geometry ($a/b, b/c < 1.4$ if possible). Blocks must not be too stratified, as gneissic and shaley materials may be. Porous materials almost always deteriorate faster than dense materials and freezing and thawing are detrimental to all not absolutely dense materials.
As thoroughly discussed in ref. 2 by the PIANC's 3rd Waves Commission, experience and testing should work hand in hand in selecting all materials with the best structural characteristics or durability. This needless to say is no less true for concrete blocks, including those which contain reinforcing steel. Some of the, usually patented, concrete blocks of involved geometry are rather prone to breakage during their placement as well as in the introductorily phases of adjustment of the mound after construction. In some multilegged block mounds a great number of blocks were found in broken condition at places where they - in fact - had not been exposed to wave action. Such block failures are dealt with in numerous papers during recent years with reference to particular blocks. The author has abstained from "advertising" these papers and reports which are available in proceedings by The Am. Soc. of Civil Engineers, The POAC and Coastal Engineering Conferences.

Poor workmanship. Most contractors have no desire whatsoever to produce poor quality work which may be harmful to their reputation and future business. But regardless of this, accidents do happen. The graded filter layers which may be included in a mound structure are easy to draw on a plan but always difficult to make. Variations in materials as well as in placement may cause "points" of weaker stability. Also the working procedures accepted are not equally considerate of, or adaptable to the materials used. As an example: floating rigs are often used for construction, particularly when material supplies are brought in by barges. Placement by floating rigs operating in wave-exposed areas often causes much downtime. As an indirect result this may then become responsible for more breakage of (concrete) blocks, particularly those with slim elements. This in turn may introduce sources for failures. Strict supervision is therefore an urgent requirement. Broken blocks should be rejected, at least as the first armour layer (refs. 8, 9).

As it may be realized a mound structure is no easy structure to build. It requires knowledge and skills beyond rough mechanical experience. As luck will have it, mound structures, particularly those which have no wave screen on the top to complicate matters, usually break down in a relatively "graceful" way. But damages may tend to accelerate in the second stages of development of failures. Natural blocks are best. Many artificial block layers placed in steep slopes disintegrate fast once damage has started. It is therefore most important not to allow any important first-phase damage for such blocks. If so the second, third and fourth phases may come overnight, (Fig. 1). Only a few comprehensive studies on the reasons for heavy damage to breakwaters have been undertaken. Known to the author is the report "Disasters of Breakwaters by Wave Action" by Hideo Takeyama and Tanekiyo Nakayama published as "Technical Notes of the Port and Harbour Research Institute" in Tokyo, March 1975, and with permission reprinted by the U.S. Dept. of Commerce, National Technical Information Service, Springfield, Virginia (22161). Its synopsis says:

"More than 30.000.000 Yen have been expended in restoration of some 63 breakwaters damaged by wave action between 1965 and 1972. Comparative data are presented for 63 examples in 49 harbours where some damage to the actual caissons was noted, encompassing caisson breakwaters damaged during construction to those finished at least with a

concrete covering. Diagrams show the breakwaters before damage was sustained, the damaged condition, and the restored cross section. Simple analyses are attempted on the total number of breakwaters between 1965 and 1972 in order to clarify any trends in breakwater damage."

Most failures cited are failures of caissons or combined structures, surprisingly many failures by shear and also many overturn and toe damages. Failures of mounds demonstrate the characteristic development of an S-slope geometry. Overwashes destruct the crown and structures behind it. Analyses of reasons for the damages, however, were not undertaken in detail. There is, however, a considerable experience available elsewhere. With respect to rock mounds the Scandinavian is the most comprehensive and include facts observed by still as well as moving pictures during storms and breakdowns. The recent years major failures of multilegged blocks at Arzew el Djedid, Algiria, Tripoli, Tripoli, Sines, Portugal and St. Cyprian , Spain have contributed to understanding of the problems. In all cases the lack of adequate wave data and proper analyses, insufficiant laboratory experiments, lack of consideration to basic aspects of hydrodynamic and geotechnical aspects in design must share the responsibility. Poor construction without proper supervision made conditions of stability worse.

From the above mentioned it is apparent that the massive (overall) failures may be a result of "slides" due to "mammuth" waves or wave groups causing high up - and low down rushes (resonance). Such slides may be a combined result of lifting of blocks by up and downrushes, hydraulic pressure from inside the mound and toe suction in the breaking or collapsing wave (refs. 5, 6, 12, 14, 18). Inadequate friction between armour and sublayer as explained in soil mechanic terms in refs. 5 and 9, loss of side friction and interknitting, build-up of pressures inside the mound due to low permeability of sublayers and/or core share the responsibility, (refs. 6, 8, 9, 12, 1?)

Single blocks may leave the mound by combination of impact, lift and drag forces and thereby leave a wound for further expansion. Units may suffer structural failures due to overloads of static as well as dynamic nature. In some mounds of multilegged blocks the largest number of broken blocks were found in the lowermost part of the mounds. Comparing rock to concrete blocks including box and multilegged the damage picture now experienced in numerous cases is a seen in Fig. 3, (ref. 8). Breakdown, when it first starts, takes

place rapidly for the multilegged blocks while rock mounds are more tough. Fig. 4 (ref. 8) explains this further in comparing the breakdown in relation to duration of storms of steep slopes like the multilegged to the breakdown of more gentle slopes, like rock mounds. It also gives a description of the time development of slopes of natural materials and artificial materials. In between the multilegged concrete mounds and the rock mounds one may place cubes or parallellopedic blocks of concrete on relatively gentle slopes. The experience with such blocks has been relatively good e.g. in Europeport in Holland and even better if they were grooved on four sides, releasing inside pressures, as the "Antifer-blocks" used at several places in France and in Mexico.

USE OF FORMULAS. ITS SHORTCOMINGS.

Many formulas have been proposed, but they all look alike. The original formula by Iribarren was argued semi-theoretically and included hydrodynamic as well as soils-aspects (frictions). It was the first - and probably the best - formula. Later "imitations" largely tried to "get by with less", obscuring understanding of the forces involved in foggy coefficients, often based on laboratory studies without consideration to wave hydrodynamics and hydrodynamics in the interaction processes.

Fig. 5 is Fig. 9 in Report by the PIANC (Permenant International Association of Navigation Congress) report by the 2nd Waves Commission, (ref. 1). It explains the inadequacy of reliences on formulas very convincingly in its Christmas tree of greatly varying results.

The conclusion of the Commission on the use of these formulas is expressed as follows:
"In view of the above mentioned discrepancies between the various formulae and the various questionable schematisations involved, the Commission considers the present stability formulae for rubble mound structures to have significant limitations. It is only for a preliminary assessment of the dimensions of quarrystone armour units that the formula might be applied."
It is unlikely that such statement by a highly professional international committee would have been made, unless it was well argued. It is therefor not either correct to claim that "the formula" has proven itself on normal rock structures. On the contrary. It has not. And how could it do so ? Which actual wave or wave condition does it refer to ?

Just an average "cocktail" - with a coefficient put on based on laboratory tests with some more or less arbitrary not hydrodynamically reasoned wave inputs. Two major errors in the formulas are obvious. One is as well known from multiexperiences that in nature it was certainly not the highest wave which knocked down the breakwater but rather an attenuation stage wave condition (refs. 6, 7, 13, 2o, 26). Consequently the formulae misinforms its user. The special dangerous waves, incl. wave groups (refs. 4, 6, 22, 26, 27, 28) and single (mammuth) waves (refs. 4, 9, 13, 14) are not included in the formulae, but their devastating effects are well known - now also from movies taken during storms which clearly demonstrate that it was not "height" but the "mass" and momentum which caused the extensive damage. This - not least - is seen from the famous Icelandic movie (Jan. 1981) from Akranes , Iceland. It is therefore utterly dangerous to use such formulas blindfolded, without any attempt of understanding. This does not only refer to multilegged blocks, but also to rock mounds. All "dangers" or "conditions" of wave motion cannot possibly be accounted for in a K-value. The K is by itself highly variable depending upon special characteristics of wave action, block placement, frictions and permeability. Just a little stocastic exercise should be able to convince anybody with a basic education in the physical sciences and anybody who has a comprehensive experience as well. There is a good understanding between these two. In the field we observe facts - not formulas. Many share Mr. Lacey's viewpoints (ref. 8), but progress in the waves versus structures field was always achieved by a combination of basic (hydrodynamic) understanding and practical experiences as they were observed in the field. It is in this respect preferable to let the field educate the laboratory - not vice versa, although many laboratories seem to believe in the opposite, (refs. 25, 29). Nature doesn't. To this must, unfortunately, be added that many laboratory tests were run without proper consideration to scale effects (refs. 25, 29, 31). Formula practices, however, are also illogical from an economic point of view. By analysing details of wave mechanics and interior mound hydraulics and geotechnics one will be able to make probability-based judgement as to which factors are the most pertinent for obtaining of stability in the most economic way. This is not least urgent for large structures. And there is not such things like "shallow water" and "deep water" hydrodynamics and economics. It is all the same - in different scales only. This is obvious for everybody who has seen the wave structure interaction

FUTURE EFFORTS

Efforts, therefore, must be concentrated on "field and physics" - not on further "coefficients" and not on more "cocktail-tests". And basic hydrodynamics is the same in shallow and in deep water. What was done wrong in deep was done equally wrong in shallow water. It only looks much worse in the deep waters. First of all there is an urgent need for wave data and understanding of the hydrodynamics of wave groups of all kinds of geometries and time-characteristics. But progress is being gained (refs. 4, 11, 13, 22, 24, 26, 27). We are also learning more about extreme wave heights (refs. 3, 15, 21) long-term distributions (refs. 16, 17, 23) and the limited height of breaking waves (ref. 3o). The very important joint-distribution of heights and periods have been explored extensively (refs. 6, 12, 14, 19, 2o).

Next laboratories must adjust their practices to the reproduction of hydrodynamic facts and give up "spectral tests" as a sole input. Too many failures, including those experienced during recent years in the North-Atlantic and in the Mediterranean, can in part be charged to inadequate laboratory experiments based on inadequate wave data. Wave data must be right (refs. 6, 13, 21, 25, 29). The difficulty with respect to wave data of course lies in the time needed to procure the necessary wave data. One year of observation may suffice for conditions in the low latitudes, but elsewhere a minimum of 3 years are needed (refs. 1, 5, 13, 16, 24). The result is that one must depend upon hindcasting procedures, but hindcasting still is unable to give the details of sequencies of waves of particular danger. Although we know more about grouping, the hydrodynamic implications are not fully clarified. We are able to qualify but not to quantify. Consequently we have to design on assumptions regarding extrapolation of available data and "synthetic sequencies". In doing so we need some kind of a base and no other base than extreme wave height theories and distributions (refs. 15, 16, 21, 3o) is available. Out from "the extreme height" one then has to design the details of wave sequencies. It will of course be very inadequate (senseless) to use "formulas", but it has been (is being) done. The unfortunate situation is that wave statistics has for long been "running the show" and "practicians" (often people who never saw storm waves on a breakwater) accepted whatever statistics

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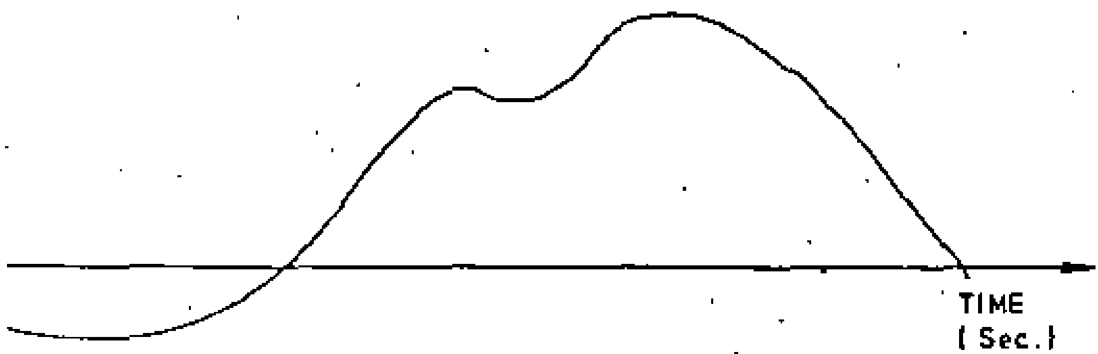


FIG. 2a Mammuth (freak) wave .

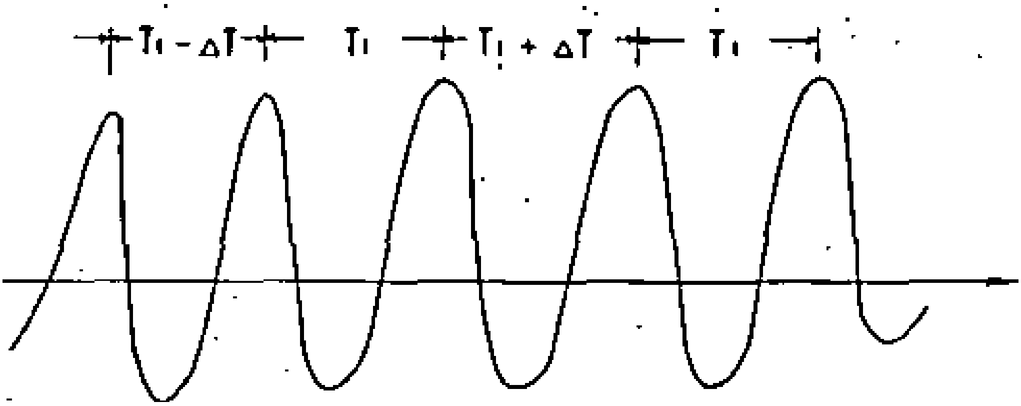


FIG. 2b Group of waves .

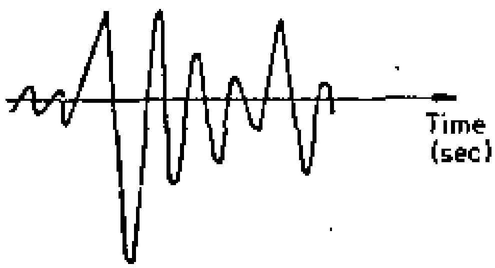


Fig. 2c - Wave series with deep trough

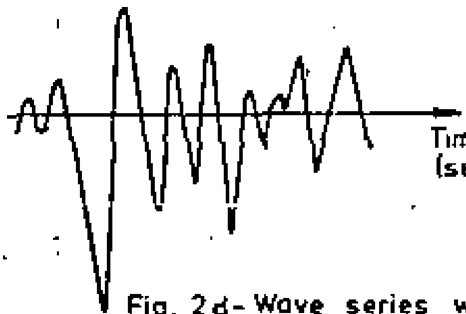


Fig. 2d - Wave series with deep trough

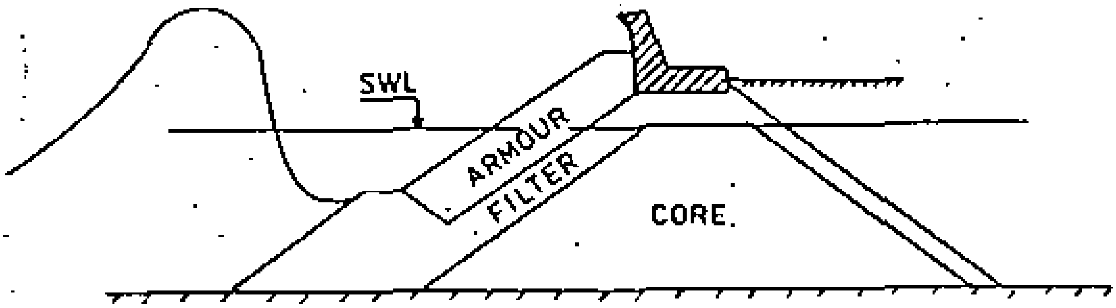


Fig. 2e - Sketch of deep run-down occuring with time series shown

Fig. 3. Damage patterns for rock and multilegged concrete blocks

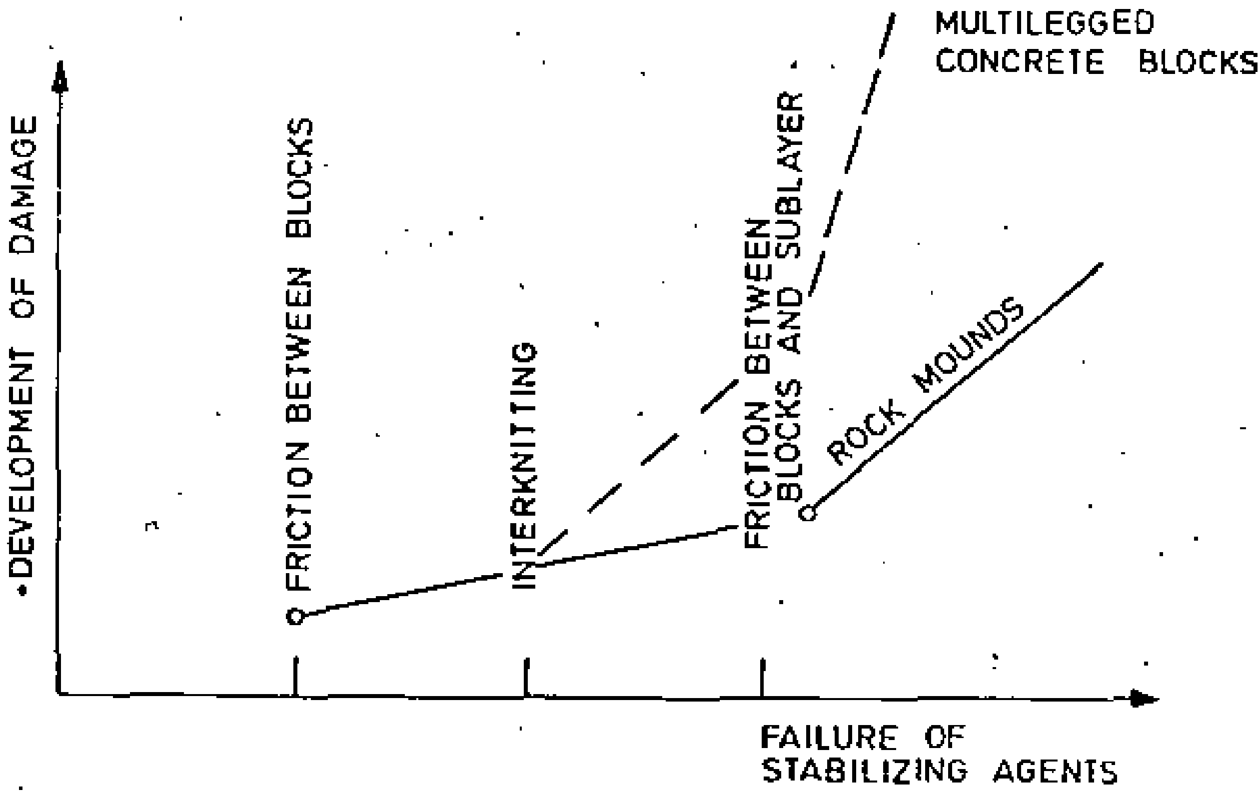
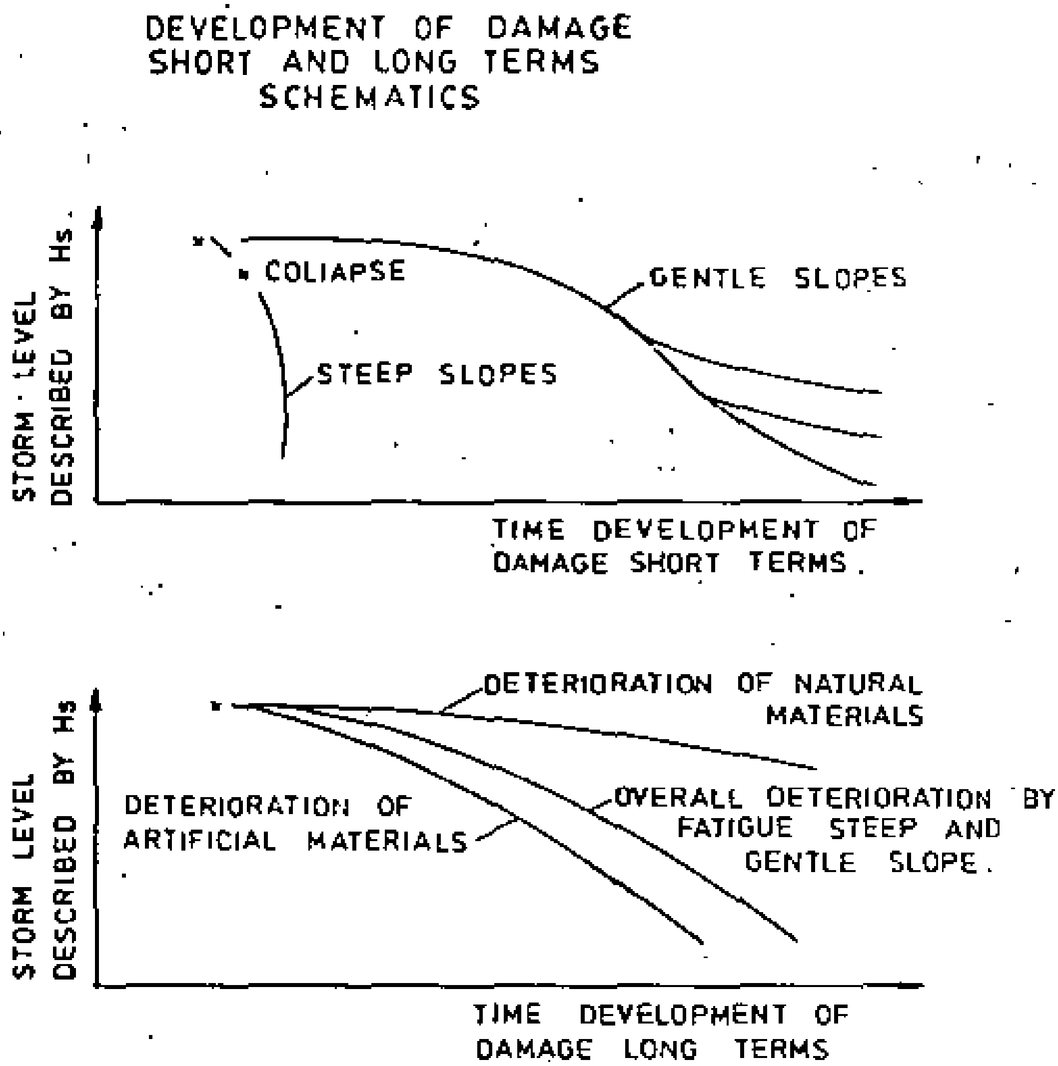


Fig. 4. Schematic description of breakdown of rubble mound in relation to duration of storms.



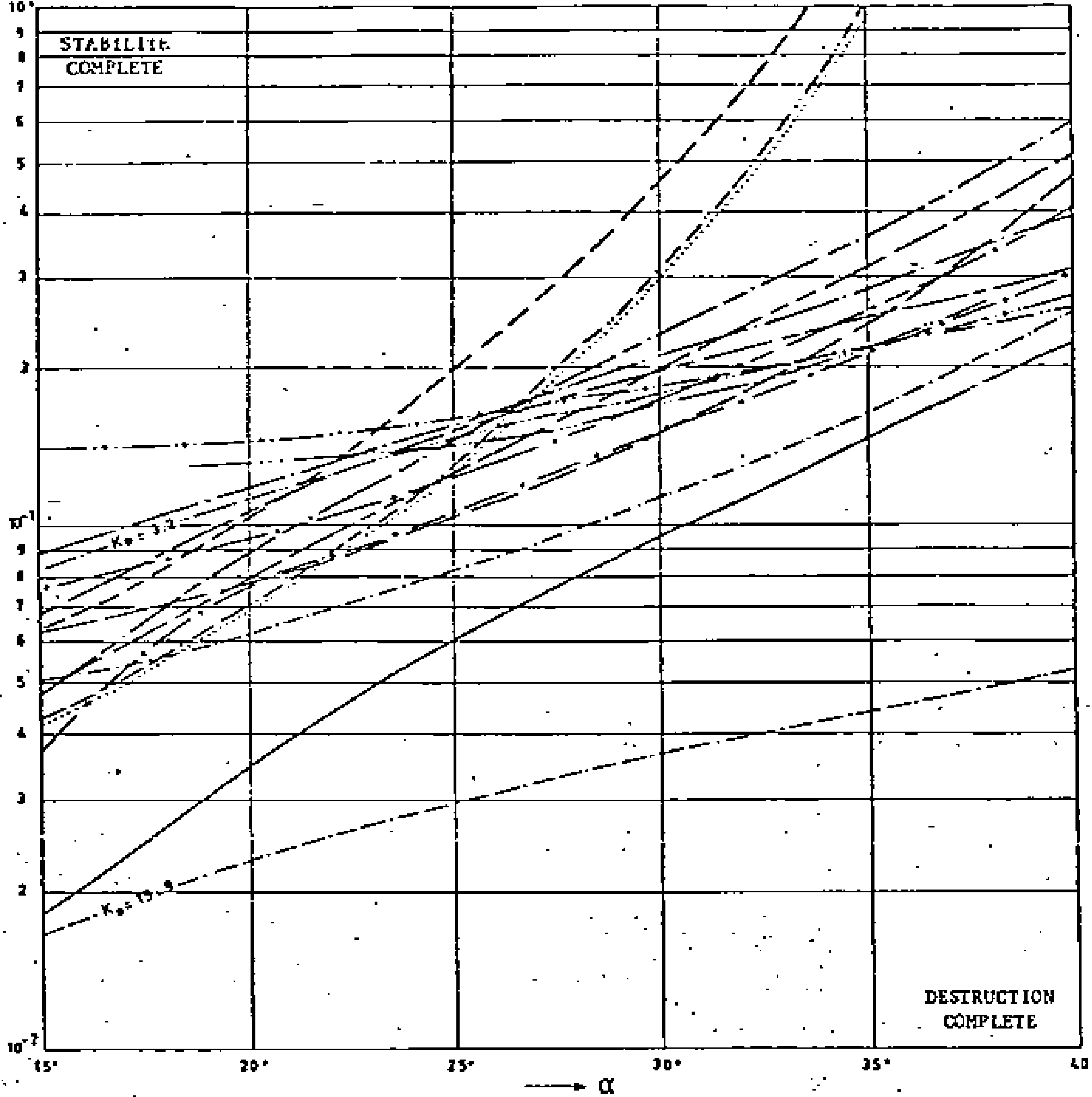


Fig. 9.

Selection of various formulae used for the calculation of artificial and natural blocks of rubble mound breakwaters in relation to height H_1.

Legends :

CASTRO	BEAUDEVIN
IRIBARREN*	HEDAR*
IRIBARREN**	HEDAR**
MATHEWS	SVEE
HICKSON/RODOLF	SN-92-60
HUDSON*	RYBTCHEVSKY
HUDSON**	METELICYNA
LARRAS	GOLDSCHTEIN/KONONENKO

5.2

FIELD PROGRAM STATUS AND NEEDS

DOS BOCAS AND OSTION

The wave program is needed for the evaluation of stabilities as well as for littoral drift calculations as mentioned later. The Ressio-report will be available in ab. 8 weeks from now, (end of March)

It shall be used for model input by wave spectra as well as for long term and extreme wave statistics.

It is pointed out that for the evaluation of stabilities it is not possible to transfer shallow water data from one site to another, unless bottom topographies are identical and they are not at DOS BOCAS and at OSTION. It is suggested that the program continues until a certain level and then levelled off as a permanent program for the future.

Current observations of Dos Bocas should continue through 1982. For reasons of navigational safety one current meter should continue operation at Dos Bocas on telemetry one current meter should be installed at Ostion.

Sediment research will completed at Dos Bocas in 1982 apart from math model studies of shore developments. It would be an advantage if a wave rider and a current water could be installed at Ostion. Transfer of deep water wave data is possible but has to be calibrated by near shore observation. Soil Mechanics by probing and sampling is elaborate at Dos Bocas. For Ostion a cambined seismic and calibration by drilling project is suggested.

MODELS STATUS AND NEEDS

DOS BOCAS

The status is that a few experiments have been run, others remain to be run. Changes have been made in designs making it necessary to re-run some tests. This refers to entrance layouts and agitation (mooring) as well as to breakwater stabilities.

Other experiments include agitation (mooring/fendering) tests on various stages of developments, seiche- tests with particular references to the interior canals and basins, water exchange experiments (pollution) and test on accidents, tests on the development of beach configurations and tests on maneouvering. Some test are mathematical, other are physical and others again are combined. See later in Table 1.

OSTION

The status is that no experiments have been run. The needs are the same kinds of experiments mentioned above for Dos Bocas, but coordination and transfer of certain types of results is possible (Table 2)

PB'mev.

Practical and Economic Approach

In order that an experiment shall run to secure reliable results of great economic importance adequate experience and equipment is necessary. Experience is something which can only be achieved with time. It takes 5-10 years to educate a laboratory engineer in an advanced laboratory, when it is always the senior engineers who take the decisions. Transfer of laboratory experience, therefore, is a slow process.

In this respect I want to call to the attention certain "standard- errors" which are often made as described below. I do that in order that Mexico can avoide making them also.

Brazil bought 4-5 years ago certain new laboratory equipment (similar to what now is being introduced in Mexico). They got certain technicians (young engineer) assistance to calibrate and run it for a year to train. The result has been numerous delays due to lack of additional equipment and errors made due to lack of experience (an inexperienced person thinks that he still can make it in three weeks, what an experienced person needs three months to do).

During the "calibration-process" it was necessary to purchase additional equipment. The lack of proper repair shops, carpenter, mechanical as well as electrical, and a "development-department", which is necessary in order to introduce the changes in an additional procurement of material was responsible

Algerie

A laboratory has been built by the government 3-4 years ago. Same equipment as in Mexico, but it has, at this time, proven to be entirely useless due to lack of trained people and adequate equipment. (Sellers of laboratory equipment told the government that they had all they needed, but this proved to be incorrect. An import (young) foreign personnel failed to change the situation.

Ceylon

A laboratory has been built, but there is a lack of trained people as well as equipment (like Algerie). An import of foreign personnel us now attemped, but only young inexperienced personnel can be obtained and it will be necessary to procure a lot of additional equipment. This is always the case, and good part of it is so special that it has to be produced on the site.

The mistakes made at these place may also be characterized as follows:

a) The car is put in front of the horse instead of the opposite.

b) It is believed that in order to become a dentist you just need to purchase some "drilling -equipment" , and then start drilling".

c) It is believed that in order to become a medical doctor in surgery you just want to buy a saw , a drill, a chiesal, etc., then go ahead, you may of course also hire a nurse or medical technician to assist you. Results were faulty (Algerie and Brazil).

d) and in order to become a fireman you just purchase a fire truck, etc.

The indeed sad situation is that "sellers" of laboratory equipment often make the purchaser believe that he just needs that. But he needs a lot more.

As an old-timer in Mexico I have the desire of aboiding errors and instead suggest a solid foundation for real progress, independent of foreign equipment and models, but to do that, we need training, and the untrained personnel often does not realize that he has only inadequate knowledge and experience, but it is much better to

learn for three years and then to do things right instead of delivering poor results " immediately".

Training my be obtained in basically two differente ways:

1. By stays in advanced laboratories for minimum three years, training to"take-over".

2. By over-lapping and parallel experiments in the case of a dedicated person who stayed three years to learn. the often used " supervision " byyoung unexperienced engineers produce "little but trouble"

Regarding 2) , I suggest the following:

Considering the above mentioned experiences and knowledge I have made up an integrated program for DOS BOCAS and OSTION under the assumptions of efficiency, training and economy (Table 1 on test, and Table 2 on timings)

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

CURSO:

"INGENIERIA MARITIMA"

IX JORNADAS DE INGENIERIA CIVIL

TEMA VI

PRINCIPALES ASPECTOS A CONSIDERAR
EN UNA EVOLUCION DE IMPACTO AMBIENTAL

ING. JORGE ROBERTO LIMON F.

12 AL 17 DE JULIO DE 1982

GUAYAQUIL, ECUADOR.

PRINCIPALES ASPECTOS A CONSIDERAR EN UNA EVALUACION DE IMPACTO AMBIENTAL.

Esta previsión requiere fundamentalmente:

1.- <u>Definición del entorno físico-social, económico, y condiciones ambientales actuales, "estado cero".</u>

(Incluímos la definición del entorno y la del "estado cero" en la misma fase, porque al ser los dos aspectos que requieren trabajos sobre el terreno es interesante realizarlos en lo posible conjuntamente, lo cual exige disponer ya en ese momento de un Estudio Preliminar de Impacto Ambiental).

2.- <u>Definición del estado del medio previsible para el futuro si el proyecto no se lleva a cabo, "estado futuro sin proyecto".</u>

3.- <u>Definición del "estado futuro con proyecto" o adición al "estado futuro sin proyecto" del "Impacto ambiental" del proyecto.</u>

Estas tres partes, operativamente se reducen a dos fases:

1. Definición del entorno y de su estado ambiental a través de recogida de información existente y de campañas de encuestas, mediciones, ensayos y análisis sobre el terreno.

2. Previsión de la incidencia del Proyecto sobre su entorno, con trabajos de gabinete basados en la información anterior (extrapolaciones, modelos matemáticos...) o realización de simulaciones sobre el terreno o en modelo reducido.

Considerando que el objetivo final de estas lecciones es el de hacer una introducción a los sistemas y métodos de interpretación y comunicación de estas previsiones, vamos solamente a revisar de forma muy esquemática la segunda fase, para conocer los datos con que nos vamos a encontrar a la hora de hacer la evaluación final del impacto ambiental de un Proyecto:

<u>Previsión de la incidencia sobre el medio.</u>

a) <u>Identificación de factores ambientales o parámetros definidores de las alteraciones o efectos que pueden sufrir los distintos medios.</u>

Estos parámetros deben referirse a efectos esperados en el medio y que pueden resumirse para el medio o biotopo y la biocenosis, en:

<u>Medio</u>	<u>Alteración en</u>
Aire	- Calidad
	- Climatología
	- Estética (Olor, Sonidos)
Agua	- Calidad
	- Cantidad
	- Distribución estacional
	- Distribución espacial
	- Disponibilidad
	- Estética (Olor, Aspecto,)

Medio	Alteración en:
Suelo	- Calidad (profundidad, fertilidad, salinidad, acidez..)
	- Estabilidad (Erosión)
	- Explotabilidad (Superficie cultivable, bosques ...)
	- Diversidad de usos
	- Estética (Paisaje)
Biocenosis	- Abundancia/escasez de especies
	- Extensión de cultivos, ecosistemas, vegetación, cosechas
	- Diversidad de especies
	- Extensión de recursos para especies migrantes
	- Abundancia/escasez de pestes y enfermedades
	- Estética (Diversidad, estado doméstico o salvaje).

Junto a esto deberíamos considerar los efectos o alteraciones en el medio humano y social; muchos de los cuales suelen ser efectos de segundo orden o finales con respecto a los enunciados anteriormente.

Medio	Alteración en:
Humano, social	- Situación económica y nivel de empleo
	- Forma de vida
	- Servicios sociales y relaciones sociales
	- Aspectos psíquicos
	- Recreo y esparcimiento
	- Salud
	- Seguridad personal
	- Cultura
	- Política

Esta lista de efectos o alteraciones es parcial y por supuesto discutible. A partir de ésta obtendríamos los factores ambientales que definen cada alteración y que es la lista final a contemplar en una identificación de este tipo. existen muchas listas de factores ambientales ordenados de muy diversas formas y con contenidos muy distintos, como más conocidos podemos citar las de Leopold (88) y Battelle (73), aunque en el caso de Leopold, como veremos posteriormente, esta identificación se facilita a traves de una primera identificación de causas que puede ir seguida de la utilización de diagramas causa efecto, para obtener la lista de éstos.

Es muy importante advertir que aunque estas listas existentes son interesantes como información para revisión, el prestarles demasiada atención puede obligar a no contemplar ciertos factores ambientales específicos del proceso en cuestión no contenidos en aquella, por lo que en cualquier caso deben hacerse unas consideraciones ambientales sistemáticas independientes.

Como ejemplo de factores ambientales citaremos los correspondientes a Calidad del Agua y Aire, en el modelo de Battelle.

Calidad del Agua

- Disminución en hidrología de la cuenca
- Demanda bioquímica de oxígeno
- Oxígeno disuelto
- Coliformes fecales
- Carbono inorgánico
- Nitrógeno inorgánico
- Fosfato inorgánico
- Pesticidas
- pH
- Variación en caudal
- Temperatura
- Sólidos totales disueltos
- Productos tóxicos
- Turbidez

Calidad del Aire

- Monóxido de carbono
- Hidrocarburos
- Oxidos de Nitrógeno
- Partículas
- Oxidantes
- Anhídridos Sulfuroso y Sulfúrico
- Otros.

Este aspecto de identificación de factores ambientales es muy importante para el desarrollo del Estudio, y debe ir acompañado de una evaluación preliminar de su importancia, basándonos en la experiencia en proyectos similares o en un simple proceso deductivo, el objeto de dedicar nuestra atención a aquellos que pueden ser significativos como impacto ambiental.

Como ayuda a la identificación, es muy importante desarrollar en paralelo una mejor identificación de las causas a través de un estudio detallado de los elementos del proyecto susceptibles de alterar el medio y de las medidas correctoras disponibles para minimizar dicha alteración, además esta información debe incluirse en el Estudio.

b) Definición de los elementos del proyecto susceptibles de alterar el medio y descripción de las medidas correctoras para minimizar dicha alteración.

La identificación de estos elementos debe hacerse en paralelo con la etapa anterior.

Estos elementos deben definirse solamente en los aspectos que interesan al medio ambiente y que pueden resumirse en.

- Proceso de incidencia sobre el medio (Emisión, vertido, ocupación)
- Parámetros que definen el proceso de incidencia (Caudal de gases ...)
- Características técnicas de la instalación que condicionan la incidencia (Materias primas utilizadas, nivel de producción, materiales de construcción...)
- Medidas correctoras previstas
- Modificaciones posibles.

c) Evaluación de la alteración del medio físico (Vectores aire, agua suelo).

Las alteraciones del medio físico, suelen ser de primer orden, como además son normalmente bastante fáciles de medir, son casi siempre el objeto de la legislación en cuanto a limitaciones de efectos, por lo que se suelen utilizar como indicadores de impacto (factores ambientales indicadores del impacto ambiental neto de un proyecto), evaluándose la importancia de su alteración en función de los efectos de segundo, tercer orden y finales (bienestar y salud del hombre) que lleven consigo.

Por razón de su importancia, dedicaremos una atención especial a la evaluación de estos parámetros.

Normalmente se les suele llamar vectores ambientales en cuanto que son los portadores del efecto hacia sus destinatarios últimos, biocenosis, hombre.

Aire.-

La alteración del aire, como hemos dicho anteriormente, se evalua a través de su calidad (variación en su composición), por aporte de elementos extraños al mismo y con efectos indeseables, la concentración de los cuales a nivel respirable se define como nivel de inmisión. El proceso causa-efecto, se traduce en el diagrama de flujo:

EMISION ⟶ DISPERSION ⟶ INMISION

(Ver fig.4.3.1 como ejemplo)

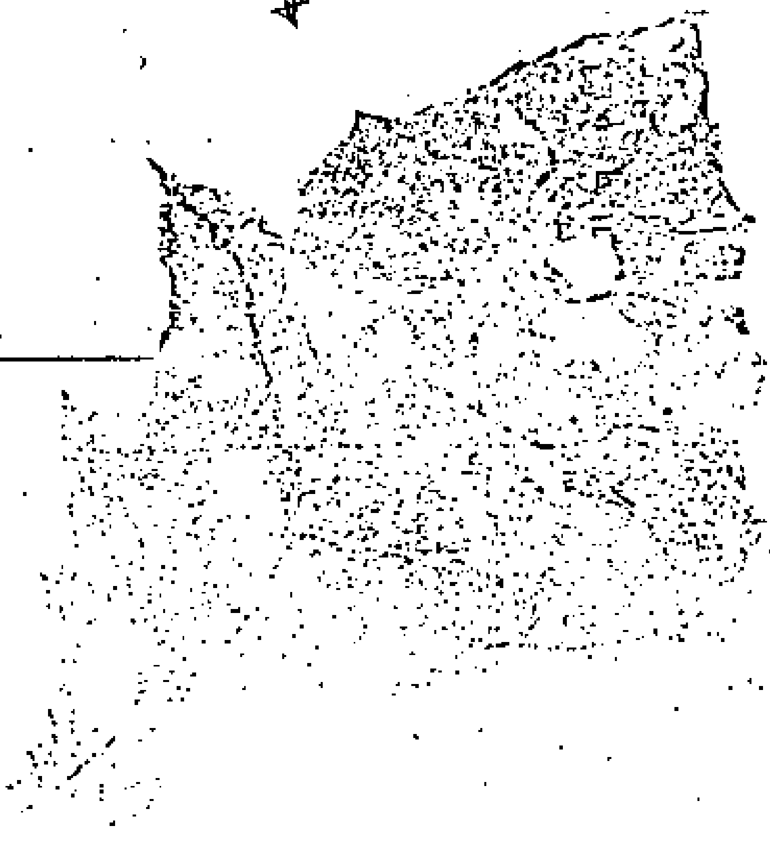
En el cual la emisión depende del proyecto en cuestión, la dispersión del medio atmosférico y físico (topografía) (climatología local) y la inmisión es el resultado final, condicionado por la calidad de la emisión y las condiciones de dispersión del medio.

La evaluación de los niveles de inmisión, o concentraciones resultantes, debe realizarse para los productos contaminantes lanzados a la atmosfera en cantidades significativas.

Siendo el proceso a seguir:

Evaluación de emisiones atmosféricas.

En primer lugar, y a traves de los diagramas de proceso y planos de implantación de la planta, se definirá la localización de los puntos de vertido a la atmósfera (dándoles números de referencia identificativos



A instancias de la Subsecretaría de Mejoramiento del Ambiente, se realizó la reevaluación de la *Ley para Prevenir y Controlar la Contaminación Ambiental,* cuyo nuevo texto fue aprobado el mes de diciembre de 1981 y publicado en el *Diario Oficial* de la Federación, el 11 de enero de 1982.

Se programó la reestructuración de la ley con la intención de adicionar varios puntos que no se habían contemplado en la legislación respectiva, de 1971, y cuyos cambios estuvieran más de acuerdo con las nuevas condiciones de la realidad del país en materia ambiental, después de una década de experiencias administrativas en el control y prevención de la contaminación.

La nueva legislación ambiental, plantea múltiples oportunidades para la realización de programas y estrategias más efectivas dentro del marco legal para solucionar los graves problemas de contaminación atmosférica, del agua y del suelo, a los que debe enfrentarse la SMA.

Entre los aspectos que permitirán ejercer con mayor facilidad las acciones de control, prevención y disminución de la contaminación, se tiene el que la SSA propondrá al Ejecutivo Federal la expedición de medidas para localizar, clasificar y evaluar los tipos de fuentes de contaminación y, en general, cualquier actividad que degrade o dañe el ambiente; determinar las medidas, procesos y técnicas adecuadas para la prevención, control y abatimiento de la contaminación ambiental; regular la exploración, explotación, producción, transporte, composición, almacenamiento y el uso y disposición final de energéticos, minerales, substancias químicas y otros productos que puedan causar o causen contaminación del ambiente; realizar, contratar y ordenar los estudios, obras o trabajos para proteger el ambiente, y proteger la flora y la fauna, especialmente aquellas especies que estén en peligro de extinción, o se consideren benéficas para el equilibrio de los ecosistemas.

Asimismo, se ha determinado lo referente a la protección del medio marino, que no estaba contemplado en la ley de 1971.

En lo referente a la protección de suelos, la SSA conserva la facultad de autorizar las obras y las instalaciones que se harán en ellos con fines urbanos, industriales, agropecuarios y recreativos. Otro tópico nuevo es el de efectos contaminantes producidos por la energía térmica, ruido y vibraciones. También se dedicó especial atención al capítulo de *protección de alimentos y bebidas por efectos del ambiente.*

Los reglamentos específicos para el control de *contaminación atmosférica originada por emisión de polvos y humos* (17 de septiembre de 1971), el de *prevención y control de la contaminación de aguas* (29 de marzo 1973) y el de *prevención y control de la contaminación ambiental originada* por la *emisión de ruidos* (2 de enero 1976), estarán vigentes mientras se elaboran los nuevos reglamentos específicos, según los cambios y adiciones de la nueva ley que abroga definitivamente la del 12 de marzo de 1971.