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SEMINARIO DE INGENIERIA
OCEANICA

Lunes 22 de Marzo de 1982

HORA	TEMA	EXPOSITOR
9:00	I N A U G U R A C I O N .	
10:00	El Estado del Arte en la Ingeniería Oceánica	R. Bustamante, Per Bruun
11:00	ESTRUCTURAS COSTA AFUERA: Fuerzas que actuan y características. Alternativas.	Dr. Roberto Meli
12:00	C A F E .	
12:15	AMBIENTE MARINO: Ondas: Tipo, origen, medición, parámetros, caracterización y análisis desde el punto de vista de funcionalidad estructural, simulación	O. G. Houmb
13:30	C O M I D A .	
16:00	AMBIENTE MARINO: Corrientes, mareas, meteorología, caracterización, origen y efectos. Playas, dunas, barras, arrastre, litoral y sedimentos.	O. G. Houmb, R. Saenger
17:15	C A F E .	
17:30	AMBIENTE MARINO: Geología, geotécnia y geofísica marinas. Suelos, muestreo, análisis e interpretación.	R. Marsal, J. Hanell

SEMINARIO DE INGENIERIA
OCEANICA

Martes 23 de Marzo de 1982

HORA	TEMA	EXPOSITOR
9:00	ARREGLO DE PUERTOS: Nuevos desarrollos en Ingeniería Portuaria y selección del sitio.	Per Bruun.
10:00	ARREGLO DE PUERTOS: Métodos y Modelos de Planeación.	E. G. Frankel.
11:15	C A F E .	
11:30	ARREGLO DE PUERTOS: Aspectos de navegación y transporte.	E. G. Frankel.
12:30	ARREGLO DE PUERTOS: Arreglo de puertos, canales y dársenas.	Per Bruun.
14:00	C O M I D A .	
16:00	DISEÑO DE ESCOLLERAS Y ROMPEOLAS: Principios básicos de diseño.	Per Bruun.
17:00	C A F E .	
17:15	DISEÑO DE ESCOLLERAS Y ROMPEOLAS: Nuevos principios de diseño. Bloques de coraza y su estabilidad.	Y. Fernández.

SEMINARIO DE INGENIERIA OCEANICA

Miércoles 24 de Marzo de 1982

HORA	TEMA	EXPOSITOR
9:00	DISEÑO DE ESCOLLERAS Y ROMPEOLAS: Diseño de rompeolas verticales, permeables y sumergibles.	Dr. P. Bruun
10:00	DISEÑO DE ESCOLLERAS Y ROMPEOLAS: Muelles, elementos prefabricados y preesforzados, atraque y amarres.	Dr. P. Bruun
11:00	C A F E .	
11:15	CONSTRUCCION DE ESCOLLERAS: aspectos prácticos en el diseño y la construcción.	F. Mendoza Y. B.
14:00	C O M I D A .	
16:00	CONSTRUCCION DE ESCOLLERAS: Preparación y procuramiento.	Ali Mesta
17:00	C A F E .	
17:15	CONSTRUCCION DE ESCOLLERAS: Problemas generales y específicos de construcción	Ali Mesta

SEMINARIO DE INGENIERIA OCEANICA

Jueves 25 de Marzo de 1982

HORA	TEMA	EXPOSITOR
9:00	MUELLES, ATRACADEROS, DEFENSAS Y DISPOSITIVOS DE AMARRE: Consideraciones de diseño y de Construcción.	V. Fernández.
10:00	DRAGADO: Tipos y procesos.	Van der Kieboom.
11:15	C A F E .	
11:30	DRAGADO: Aspectos Económicos y disposición de material.	Van der Kieboom.
14:00	C O M I D A .	
16:00	ESTRUCTURAS COSTA AFUERA: Vehículos Marinos	J. L. Sanchez B
17:00	C A F E .	
17:15	CONSIDERACIONES AMBIENTALES: Aspectos ecológicos y de reglamentación.	Luis A. Soto

SEMINARIO DE INGENIERIA OCEANICA

Viernes 26 de Marzo de 1982

HORA	TEMA	EXPOSITOR
10:00	EL PORVENIR PORTUARIO Y OCEANICO DE MEXICO.	
12:00	DISCUSION GENERAL Y SINTESIS DE RESULTADOS.	REPRESENTANTES DE PEMEX, SCT, UNAM, IPN, PMSC, FONDEPORT Coor. Proy. Des. CFE, PESCA.
13:00	C L A U S U R A .	
14:00	COMIDA Y CONVIVIO FINAL DE LOS PARTICIPANTES.	

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

SEMINARIO DE INGENIERIA OCEANICA

PROF. OLE GUNNAR HOUMB.

MARZO DE 1982.

SEMINARIO DE INGENIERIA OCEANICA

WAVE CLIMATE AND DESIGN AND OPERATIONAL CONDITIONS OF THE NORTH SEA

PROF. OLE GUNNAR HOUMB

MARZO DE 1982.

THE SHIP RESEARCH INSTITUTE OF NORWAY

MARINE TECHNOLOGY CENTER

WAVE CLIMATE AND DESIGN AND OPERATIONAL CONDITIONS OF THE NORTH SEA

By

OLE GUNNAR HOUMB

THE SHIP RESEARCH INSTITUTE OF NORWAY

Presented at The International Symposium on Port and Ocean Engineering, Mexico January 1982.

1. INTRODUCTION

Wind generated surface water waves are of major importance to design, operation and safety of marine structures. Estimates of hazards for loss of human life and for pollution of the marine environment require good descriptions of ocean winds, waves and currents. To the authors knowledge there are no waters in the world where the available environmental data satisfactorily meet the needs of the marine industry. The aim of several national wave climate studies is therefore to collect data to meet such requirements.

The aim of this paper is not to present the state of the art. of the knowledge to the worldwide marine environment. The authors intension is to describe some recent results from the collection and analysis of wave data from Norwegian waters. Work with regulations and design criteria for the offshore industry in cooperation with the Norwegian Petroleum Directorate has had a strong influence on the work as presented.

Mr. M.J. Varkey has given a written contribution to the Section on extreme wave predictions.

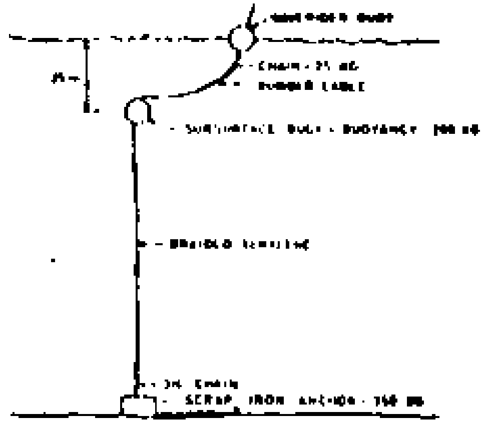

Fig. 2.1 Waverider mooring system

2. WAVE DATA AND INSTRUMENTATION.

2.1 Instruments

The most common wave recorder in the North Sea is the Dutch Datawell Waverider (WR). This is an accelerometer buoy of sperical shape. It is available with diameters 0.7 and 0.9 meters. The mooring system used in depths beyond 60 meters is shown in Fig. 2.1. Waves of height up to 30 meters can be measured. The 0.7 meter diameter buoy will not accurately measure waves of period shorter than 1.8 sec. Data are transmitted as doubly integrated accelerations in the 27MHz band, and the range is 50 km. On a routine basis 2048 data points are recorded with a sampling interval of 0.5 sec.

WRs have been used in the North Sea area since 1969, and the experience gained so far is good. The percentage of good data recovered varies between 95 when an oceanographic observation vessel watches the buoys, and 80% for unattended buoy stations.

Wave staffs are used onboard some oil rigs, but the experience from analysis of the data is not satisfactory. The main reason is that there tends to be a runup of waves on the vertical platform legs causing an overestimation of wave height. In addition the wave staffs tend to work as a good antenna for the onboard radiotransmitters. In such cases we have seen that wave heights are up to twice of those recorded by a WR.

Down looking narrow beam radars that measure the distance down to the sea surface are used on two fixed oil rigs. The types of instrument are Plessey Radar Wave Monitor and SYNINEX/TRT Radar Wave Meter. The preliminary conclusion as regards the use of these radar wave recorders is that the spectral density at high frequencies tend to be underestimated.

2.2 Sources of wave data

2.2.1 Visual wave data

As an example of visual wave data are observations undertaken at lighthouses on the coast of Norway.

At the Norwegian coast there are 41 lighthouses where meteorological observations are being undertaken. 15 of these have been selected, and data comprising visual sea state observations, instrumentally recorded wind force and wind direction and sea surface air pressure have been punched on cards. The data consist of 4 daily observations over a period of 20 years. For the other lighthouses data covering a period of 10 years are available on punched cards.

The sea state observations are ranged by the observers according to Table 2.1, and they do not contain any information on wave periods. Wind force is reported in the Beaufort scale and the sea surface air pressure is given in millibars.

As it can be seen from Table 1 the sea state observations are denoted by numbers from 0 to 9, each number representing an interval in meters wherein the observed wave heights are located.

Table 2.1. The visual wave height

CLASS NUMBER	WAVE HEIGHT INTERVALS IN METERS
0	0
1	0-0.1
2	0.1-0.5
3	0.5-1.25
4	1.25-2.5
5	2.5-4
6	4-6
7	6-9
8	9-14
9	>14

Class no. 7 includes waves from 6 to 9 meters and class no. 8 from 9 to 14 meters. This essentially excludes the possibility of basing estimates of design waves higher than, say, 9.0 meters on these data. Because of the low resolution in wave heights it is assumed that the visually observed wave heights correspond to the significant wave height $H_{1/3}$.

The Norwegian Hindcast Project has used the visual lighthouse observations in a way which will be described in connection with that project.

Visual observations are also being taken by weather ship's crew. At Weather Ship M in the Norwegian sea, such observations have been taken since 1949, and the weather and rescue ship Famita has taken visual wave data since 1959 between October and March, Fig. 1.

At Famita observations were made every third hour. The same routine was followed at M up to April 1969. The interval between observations was then reduced to one hour. Regarding the M-data, observations other than those three hours apart are neglected in this paper.

Visual ship observations of waves include the height and period of wind sea H_w and T_w, and of swell, H_{sw} and T_{sw}. The visual wave height, H_v, is then estimated by

$$H_v = \sqrt{H_{sw}^2 + H_w^2}$$

The average visual wave period, T_v, is estimated as

$$T_v = \frac{2\, T_{sw}\, T_w}{T_{sw} + T_w}$$

The wave height data are given in classes.

Class 0: 0.0 to 0.25 m, Class 1: 0.25 to 0.75 m
Class 2: 0.75 to 1.25 m, etc.

The main emphasis in the analyses is concentrated on the M-data, because they cover the longest series, and very few observations are missing. The Famita data have many more gaps, due to the rescue activity.

Both data bases include wind force and direction.

2.2.2 Instrumental wave data

The sources of instrumental, visual and hindcast wave data are listed in Table 2.3. Table 2.4 gives the locations and depths of the buoy stations. The environmental data collection programme consists of a number of stations primarily in connection with fixed oil rigs, Fig. 2.2.

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Table 2.3. Sources of wave data for the Norwegian Continental Shelf up to 1978.

Utsira	position 59°18'N, 4°49'E, water depth 220 m,
Halten	position 1. 64°11'N, 9°10'E, water depth 3-100 m, position 2. 64°27'N, 8°31'E, water depth 190 m,
Hekkingen	position 69°41'N, 17°13'E, water depth 60 m,
Loppahavet	position 70°37'N, 21°49'E, water depth 70 m.

Table 2.4. Locations of wave buoy stations on the Norwegian Continental Shelf, see Fig. 2.2

2.2.3 Hindcast data

A Northern Hemisphere deep water wave spectral climatology covering the years from 1956 to 1975 is being developed at the Fleet Numerical Oceanographic Center at Monterey, California. Presently the 20 year period is completed for the North Atlantic, whereas 4 years remain to be covered for the Pacific. They use a modified version of the model by Pierson, Tick and Baer (1966). The model calculates directional wave spectra by 15 frequencies and 12 directions, using a slightly modified Miles-Phillips growth scheme. Fully developed sea is given by the Pierson-Moskowitz spectrum. Grid size ranges from 160 to 300 km. The wind input is given by the model of Cardone (1969).

There are some problems connected with wind algorithm causing winds to be underestimated. In spite of this the H_s errors tend to be random. Dr. Lazanoff at the Fleet Numerical Oceanographic Center at Monterey estimates the results to be available in 6 to 12 months from now (Oct. 1980).

The North Sea Wave Model Project (NORSWAM) was funded by oil companies and the UK Dept. of Energy. The model was developed by Günther et.al. (1979). The actual project work was carried out at the Institute of Oceanographic Sciences and at the Hydraulics Research Station, both in UK, Ewing et.al. (1979).

The advisory board was set up by members from oil companies and the countries surrounding the North Sea.

Weather reports of the British Meteorological Office covering the period from 1966 to 1976 were examined, and gales in the North Sea and adjacent waters were selected, Harding and Binding (1978). From a total of more than 200 gales, 42 were selected for analysis. The 42 storms were chosen such that a representative distribution of extreme storm was obtained.

Wind fields were specified at 3-hour intervals on a 100 km grid in the North Sea and 300 km grid outside, Fig. 2.4. From re-analysed weather maps including ships observations the pressure fields were estimated. Then geostrophic winds were calculated, and converted to surface winds as shown by Findlater et.al. (1966).

The energy balance is given by

$$\frac{\partial E}{\partial t} + \vec{V}_g \cdot \nabla E = S$$

where E is the two dimensional spectral density and $\vec{V}_g$ is the deep water linear group velocity. S is the source function that represents the energy budget. S consists of the momentum transfer from the atmosphere to the waves by the wind, the nonlinear interaction between waves, and dissipation term. The model treats wind waves and swell separately, and the balance of energy between these parts is governed by certain empirical relations. The spectrum is modeled by

$$\hat{E} = \hat{E}(f) \cdot \Phi(\theta - \phi)$$

where Φ is a $\cos^2$ function and ϕ is the local wind direction. $\hat{E}(f)$ is given by

$$\hat{E}(f) = \alpha g^2 (2\pi)^{-4} f^{-5} \exp -\frac{5}{4} \left(\frac{f}{f_m}\right)^{-4} \quad \gamma \exp\left(-\frac{(f-f_m)^2}{2\sigma^2 f_m^2}\right)$$

where

- α Phillips' constant
- f_m modal frequency
- γ peak enhancement factor
- σ peak width parameter

For wind seas the energy equation is projected into a parameter space, such that the evolution of the spectrum is described by n parameters.

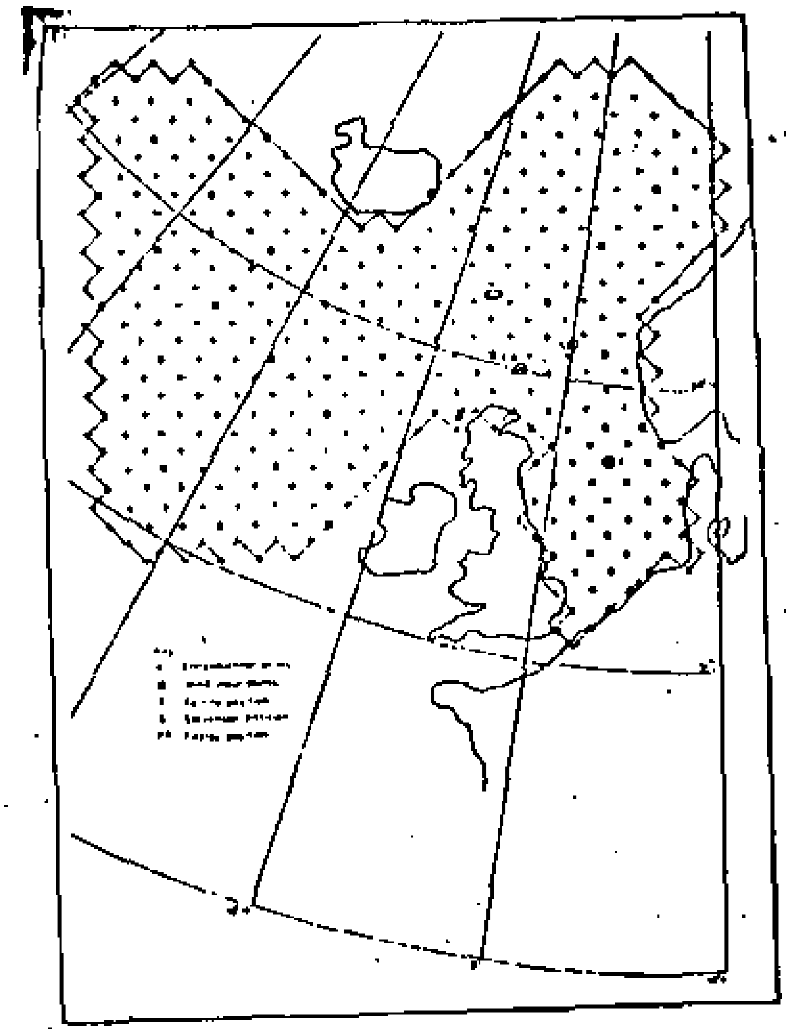

Fig. 2.4 Grid used by NORSWAM model- (From Ewing et.al. (1979)).

Swell energy was advected along predetermined straight wave rays. This was done to facilitate future refraction analysis for shallow waters.

The model uses ten frequency bandwidths from 0.05 to 0.15 Hz and 9 direction intervals for swell calculations. Günther et.al. (1979) describes the transfer of energy between wind sea and swell as the wind increases or decreases.

The model was calibrated against instrumental data from two oceanographic observation ships in the North Sea. A weighted least squares fit of the maximum significant wave height, H_s, resulted in

$$H_{smeasured} = 0.985\, H_{smodel}$$

The scatter was 25% of the mean.

Extreme value analysis was conducted using the maximum value of H_s from each storm as statistics. The first asymptotic Gumbel distribution gave a reasonably good fit.

The occurence of storms was assumed to be described by a Poisson process, with a rate of 11.9 storms per year. A return period (R_p) scale was constructed

$$R_p = \frac{1}{\lambda(1-P(H_s))}$$

where

$$\lambda = \frac{11.9N}{42}$$

and N is the number of storms that produced $H_s \geq 5$ m at the location.

The zero crossing wave period, T_z, corresponding to a given H_s is for an average JONSWAP spectrum given by

$$T_z = \frac{0.78}{f_m}$$

A typical maximum value for R_p = 100 years is H_s = 18 m.

In the Norwegian Wave Climate Study the wind fields are specified on a grid of 150 km, Fig. 2.5, every 6 hours. Interpolations are made so that the actual time step is 3 hours. The pressure fields are derived from reanalyzed weather maps, taking into account also ship observations. One major problem is that for the northern part of the hindcast area, there are very few observations available. The wind fields are therefore less than satisfactorily accurate in the northern Norwegian waters.

The model is described by Haug (1968) and Houmb (1969), whereas the hindcast project was described by Haaland (1979).

The Norwegian hindcast model, is based on the growth curves by Neumann (1952), and the fully developed sea is described by the Neumann spectrum. Some adjustments have been made to the growth curves based on empirical evidence from calibration studies. The model output is given in terms of the scalar wind wave spectrum - so directionality is not included.

Swell from distant storms is computed for some locations of special interest. For such cases a spectrum of mixed wind sea and swell is obtained.

The philosophy behind the use of a relatively simple model as outlined above, is that there is very little to gain by sophistication of the model when the wind output is as crude as in the present case.

Results of calibrations show that the predicted H_s are normally with ± 10% of measured H_s. In a number of cases the model predicts the peak of a storm prior to or after the actual measured peak, but that is not important as long as the actual value is within satisfactory limits. In the north there are during winter, frequent small intense squalls moving too rapidly in time and space to be detected based on weather maps. During the most of such events the model will underestimate the sea state.

In the Norwegian Hindcast Project the most extreme storm every year back to 1949 were selected for a numer of locations of the Norwegian Continental Shelf.

One part of the project consisted of transforming frequency tables of visual wave observations from offshore weather ships and lighthouses on the coast of Norway. Assume that Fig. 2.6 is a histogram showning the long term probability density distribution of visually observed wave heights, H_v.

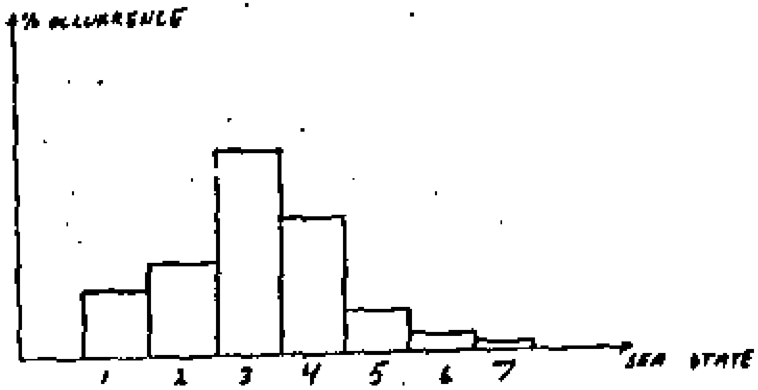

Fig. 2.6 Histogram of visual wave heights

If we for e.g. class no. 5 go back to the observations, reanalyze the weather maps of a number of situations that were reported as sea state 5, and then run the hindcast model, we find that not all of these situations correspond to sea state 5. Some might belong to class no. 4 or 6, or even 3 and 7. Based on a large number of such hindcast runs, class no. 5 will be "smeared out"

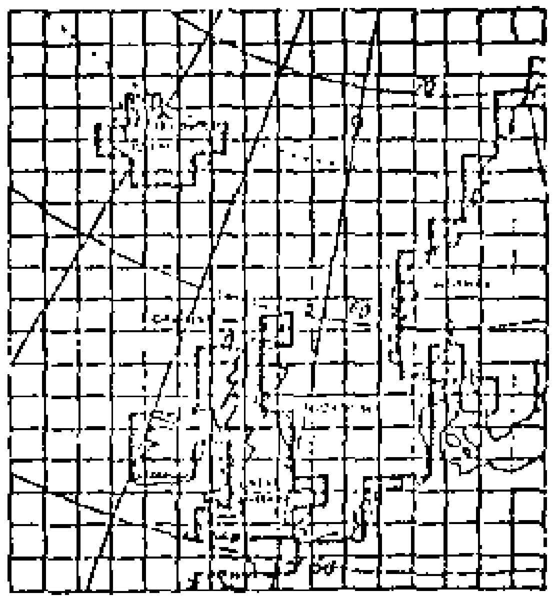

Fig. 2.5 The grid of the Norwegian Hindcast Model

to contribute also to other classes. Then the same procedure is followed for all other clas intervals, and the result is a transformed frequency table that is suitable for long statistical analysis.

3. SHORT TERM WAVE STATISTICS

3.1 General

The short term description of the water surface elevation in a reference system is treated as a redom process, quasi stationary in time and homogenous in the horizontal direction.

The assumption that the surface can be treated as a Gaussian process implies that it is the result of a linear superposition of an infinite number of spectral components.

In this section we shall consider some short term properties of wave statistics that have evolved from analysis of data mainly from the North Sea.

3.2 Distribution of wave heights and periods

It is well known that the effects of waves on structures vary not only with the wave height, but with the period as well. Therefore the joint distribution of wave heights and periods is important.

Longuet - Higgins (1975) presented a bivariate distribution of wave height and period given by

$$p_s(\xi,\nu) = \frac{\xi^2}{\sqrt{2\pi}} \exp(-\xi^2(1+\eta^2)/2)$$

where ξ and η are dimensionless wave height and period respectivel given by

$$\xi = \frac{a}{\sqrt{m_0}} = \frac{2H}{H_s}$$

$$\eta = (\frac{T}{T_z} - 1)/\nu$$

where

and

$$\varepsilon_s^2 = (1 - \frac{m_2^2}{m_4 m_0})$$

is a spectral width parameter and

$$m_n = \int_0^\infty f^n S(f)\,df$$

is the n the order moment of the spectrum. For a narrow spectrum Longuet-Higgins (1975) derived the following relation

$$\nu^2 = \frac{m_2 m_0 - m_1^2}{m_1^2}$$

The spread in ν as calculated from our data, was remarkably small with 70% of all values between 0.5 and 0.6 It is also seen that the percentage of breaking waves increases with the value of ν. The data also show a tendency for ν and the spectrum width to increase with time during wave growth.

Longuet - Higgins (1975) compared his theoretical results with measured data and found that the predicted conditional inter-quartile range was in good agreement with the data.

Cavanie et.al. (1976) consider the distribution of positive maxima and arrive at the distribution

$$p(\xi,\tau,\varepsilon) = \frac{2\alpha^3\xi^2\tau^{-4}}{(2\pi)^{1/2}\varepsilon(1-\varepsilon^2)\mu^3}\exp\left[-\frac{\xi^2\tau^{-4}}{2\varepsilon^2\mu^4}\{(\mu^2\tau^2-\alpha^2)^2+\varepsilon^2\alpha^4\}\right]$$

where

$$\tau = \frac{T}{\bar{T}}$$

and

$$\alpha = \tfrac{1}{2}(1 + (1-\varepsilon^2)^{1/2})$$

and

$$a = \frac{\varepsilon^2}{1-\varepsilon^2}$$

μ is the ratio of the mean value $\bar{T}$ to the mean time interval between positive maxima.

Based on a total of 182 records the Authors and they concluded that their theoretical distribution agreed well with that of the data.

Houmb and Overvik (1977) applied the bivariate distribution by Longuet-Higgins and derived short and long term distributions of breaking waves and also a long term distribution of wave height and period. They concluded that the long term distribution of heights of breaking waves gives conservative results. Furthermore, they also concluded the bivariate long term distribution of H and T underestimated the values of T.

3.3 Distribution of wave steepness.

Wave steepness s is defined as

$$s = \frac{H}{L}$$

Bretschneider (1959) assumed independent Rayleigh distributions for wave heights and periods squared. By his results the steepness distribution becomes

$$P(s) = \frac{s^2}{1.35} + s^2$$

Battjes (1972) arrived at the same result based on bivariate Rayleigh distribution of normalized wave height and period assuming a positive correlation between them.

Overvik and Houmb (1977) derived a distribution function for wave steepness for a two dimensional sea surface consisting of an infinite number of sinusoids of random phase. The theoretical derivitation was based on the theoretical work by Rice (1944). For a wide band spectrum they obtained

$$h(s) = \frac{4(1-\epsilon^2)}{s_s^2} \; s \cdot \exp\left[-2(1-\epsilon^2)\left(\frac{s}{s_s}\right)^2\right]$$

The cumulative distribution of s is then

$$H(s'<s) = \int_0^{s'} h(s)ds = 1 - \exp\left[-2(1-\epsilon^2)\left(\frac{s}{s_s}\right)^2\right]$$

where

$$s_s = \frac{2\pi H_s}{gT_z^2}$$

Assuming a narrow band spectrum (ϵ=0) they obtained

$$h(s) = \frac{4}{s_s^2} \; s \cdot \exp\left[-2\left(\frac{s}{s_s}\right)^2\right]$$

and

$$H(s) = 1 - \exp\left[-2\left(\frac{s}{s_s}\right)^2\right]$$

which is a distribution of Rayleigh type. The mean value is found by

$$M_1 = \frac{s_s}{\sqrt{2}} \Gamma(1.5) = 0.6267 \; s_s$$

The standard deviation is

$$\sigma = \sqrt{m_2 - m_1^2} = \frac{s_s}{\sqrt{2}} \sqrt{\Gamma(2)-\Gamma^2(1.5)} = 0.3276 \; s_s$$

Both the mean and standard deviation of the steepness have a linear relationship to the parameter s_s in the steepness distribution.

An example on the fit between theoretical and measured distributions is shown in Fig. 3.1.

3.4 Spectral shape

In the North Sea area the JONSWAP spectrum is commonly used. The JONSWAP spectrum formula is given by

$$S(f) = \alpha g^2 (2\pi)^{-4} f^{-5} \exp\left(-\frac{5}{4}\left(\frac{f}{f_m}\right)^{-4}\right) \gamma^{\exp \frac{-(f-f_m)^2}{2\sigma^2 f_m^2}}$$

$$\sigma = \begin{cases} \sigma_a = 0.07 \text{ for } f \le f_m \\ \sigma_b = 0.09 \text{ for } f > f_m \end{cases}$$

with the five parameters α, f_m, γ, σ_a and σ_b. f_m is the peak frequency and α is the Phillips constant. α, γ, σ_a and σ_b relate to the shape of the spectrum, σ_a and σ_b gives the width of the left and right side of the spectrum respectively, whereas γ is the ratio of the maximum JONSWAP spectral energy to that of the corresponding Pierson-Moscowitz spectrum.

Houmb and Overvik (1977) presented a parameterization of this spectrum. According to more recent work that will soon be published at our department, the scatter of parameters is much more pronounced than predicted by Houmb and Overvik (1977). Based on analysis of 5984 spectra, each with 16 degrees freedom, the parameters α, γ and σ were calculated with significant wave height and peak grequency fixed. Spectra for each class as shown in Table 3.1 were averaged and a least squares fit was made. Table 3.2 shows the resulting parameters.

Exept for α there is no strong systematic trend in the numbers of Table 3.2. This table also shows that there is no meaning in discussing the value of one of the parameters without considering the value of the others.

This study also showed that the JONSWAP spectrum underestimates the

PEAK FREQUENCY (Hz)

SIGNIFICANT WAVEHEIGHT (M)

Table 3.1

Scatter of H_s and f_p for spectra to which the JONSWAP spectrum was fitted

SIGNIFICANT WAVEHEIGHT (M)

PEAK FREQUENCY (Hz)

The numbers shown are:
upper — …
middle — …
lower — …

Table 3.2 JONSWAP parameters

spectral density in the high frequency region, with important consequences for fatigue life calculations. We therefore conclude that a frequency dependence of $f^{-4.5}$ gives a better fit than the traditional f^{-5}. Two typical examples are shown in Fig. 3.2.

Houmb and Due (1978) examined 2510 wave spectra from deep water Waveriderdata off the coast of Norway for the purpose of estimating the percentage of spectra with more than one peak. Their conclusion was that 4.2% of all spectra had more than one peak. Furthermore they found that such spectra are most likely to occur when $H_s \leq 3$ m. Based on a simple statistical/physical model Houmb and Due also estimated the percentage of spectra with more than one peak at 4%.

Thompson (1980) observed that 50 to 70% of shallow water spectra from the coasts of the U.S. had more than one peak, with some variation between different measurement sites.

Based on the two investigations reported above, we may conclude that for 4% of all deep water sea states and for up to 70% of those in shallow water, a spectral model like e.g. the Pierson-Moscowitz is erratic.

3.5 Wave groups

Goda (1970), Houmb et.al. (1977) and Johnson et.al. (1978) and several others have demonstrated significant deviations from the hypothesis of random wave phase. They and several others have shown that waves tend to appear in groups more often than expected based on the hypothesis of random phase.

Goda (1970), Bruun and Günbak (1977) and Johnson (1978) showed that waves appearing in groups cause considerably more damage to coastal structures than waves of random phase. Wave groupiness is therefore of much importance in coastal engineering, and it needs to be better understood.

The investigations reported above all refer to deep water data. By visual inspection of deep and shallow water wave records it easily seen that groupiness is more pronounced in shallow water. Differences between deep and shallow water waves are also expected to be found in connection with the shape of one and two dimensional wave variance density spectra.

There is a widespread consensus among scientists working in the field of wave statistics that shallow water waves deviate so much from the general random hypothesis that new statistical models have to be developed. In this section we focus the attention on wave groupiness and characteristics closely related to it.

Goda (1970) mentions that the quantity of overtopping of sea a wall, by irregular waves, is affected by the number of large waves that appear in sequence. He also assumes that if large waves appear at random, the ratio of overtopping may be estimated, for a design purpose, as the average over all the waves. If large waves have a tendency to come in groups, the average of the large waves only may be recommended for design purposes.

Bruun and Günbak (1977) also report that groupiness of larger waves reduces the stability of sloping structures.

Johnson (1978) concludes that wave grouping is an essential parameter in model testing of rubble mound breakwaters.

Funke (1978 and two non-dated reports) has studied the problem of laboratory reproduction of irregular waves of a given groupiness.

Ramamonjiarisoa and Mollo-Christensen (1979) showed that the wave field can be described in terms of modulated Stokes-like carrier waves propagating with an angular spread and a lag between amplitude and phase modulations.

Wave groups are also known to influence wave forces and wave responce in general. The occurence of surf beat is also caused by wave groups. It is concluded that wave groupiness should be included in the wave characteristics when wave runup on beaches is studied. Battjes (1972) studied the runup distribution of waves breaking on slopes. He used a bivariate distribution of wave height and period squared, and converted it to a distribution of wave steepness. Thus he was able to estimate the percentage of waves breaking on the slope.

Battjes (1972) applied Hunt's results (Hunt 1959) and concluded by recommending a study of wave data to obtain better a estimate of the distribution of wave steepness. He also stressed the importance of field measurements of runup of waves.

Overvik and Houmb (1977) prescribed an analytical expression for the distribution of wave steepness based on the classical work by Rice. Based on this work a long term distribution of wave steepness has been developed at the Norwegian Institute of Technology by Houmb and others. The report is now in print.

Van Oorshot (1968) studies the influence of spectral shape upon wave runup, and concluded that the runup increases with spectral width.

When irregular waves are reproduced in a laboratory flume, two approaches are common:

i) generation of a prescribed spectral shape of given peak frequency and significant wave height. The wave phase is then not controlled and may vary from test ot test

ii) reproduction of an actual wave record taken in the field. Then the succession of waves will always be the same. Questions may be raised regarding the representativity of such a record, because the exact details of it will never be recorded again.

The realistic goal is to describe the statistics of wave phase from field data and reproduce it in the laboratory for prescribed spectral shapes.

Some engineering applications for which wave groupiness is

important are:

- beach processes like wave runup
- stability of rubble mounds
- forces due to waves on marine structures
- harbour resonance
- slow drift oscillations of moored ships
- mooring forces
- surf beat
- dune erosion

Nolte and Hsu (1972) used the rectified wave envelope function for the purpose of isolating wave groups. They obtained this envelope function by connecting neighboring crests by straight lines after folding the troughs about the record's MWL.

Funke (1978) reports that the application of this technique was not successful. Instead he proposed to compute the square of the water surface elevation over a period being a function of the peak frequency. Using Bartlett smoothing Funke arrived at a "smoothed instantaneous wave energy history" (SIWEH) that is very useful for the purpose of isolating wave groups.

The smoothed SIWEH is given by:

$$E(t) = \frac{1}{T_p}\int_{-T_p}^{T_p} \eta^2(t+\tau)\cdot Q_k(\tau)\,d\tau \qquad (41)$$

where

$$Q_k(\tau) = \begin{cases} 1 - \frac{|\tau|}{T_p} & \text{for } -T_p \le \tau \le T_p \\ 0 & \text{elsewhere} \end{cases} \qquad (42)$$

is the Bartlett window.

Now one can also calculate the spectrum of the SIWEH because of its usefulness in describing the extent of groupiness. The SIWEH spectral density is defined by Funke as:

$$E(f) = \frac{2}{T_n}\left(\int_0^{T_n}(E(t)-\bar{E})e^{-j\omega t}dt\right)^2$$

where T_n is the length of the wave record and overbars denote averages. In particular one has that

$$\bar{E} = \frac{1}{T_n}\int_0^{T_n} E(t)\,dt = \int_0^{\infty} S_n(f)\,df$$

where S_n is the spectral density of the sea surface elevation. Funke computed E(f) without smoothing in order not to lose any details.

An example from Funke (1979) is shown in Figs. 3.3 and 3.4.

Presently little is known about the relations between E(f) and $S_n(f)$ so some attention should be given to that problem.

The wave group activity estimator proposed by Funke is:

$$GF = \sqrt{\frac{1}{T_n}\int_0^{T_n} (E(t)-\bar{E})^2\cdot dt}\,/\bar{E} = \sqrt{m_{0E}}/m_0 \qquad (43)$$

where m_{0E} and m_0 are the zeroth moments of the SIWEH and thesurface elevation variance spectral desities respectively. It would be very interesting to evaluate the use of SIWEH analysis for the detection of wave group activity in recorded field data. This technique has many potential advantages because its frequency domain approach allows the power spectrum techniques which have been so successfully applied to sea surface elevation statistics, to also be applied to groupiness phenomena.

The E(f) spectral estimates have wide confidence limits because the number of groups is much smaller than the number of waves. In order to improve the confidence of the SIWEH spectral estimates it is proposed to calculate E(f) both by the averaged periodogram method, Welch (1969), Howell (1980) and the Maximum Entropy Method (MEM), Burg (1967), Houmb et.al. (1979 and 1980). MEM is equivalent to an extrapolation of the autocoveriance function, being consistent with some model assumptions. The result is an increased spectral resolution. Analysis in the time domain as indicated by Houmb and Overvik (1977) should also be considered.

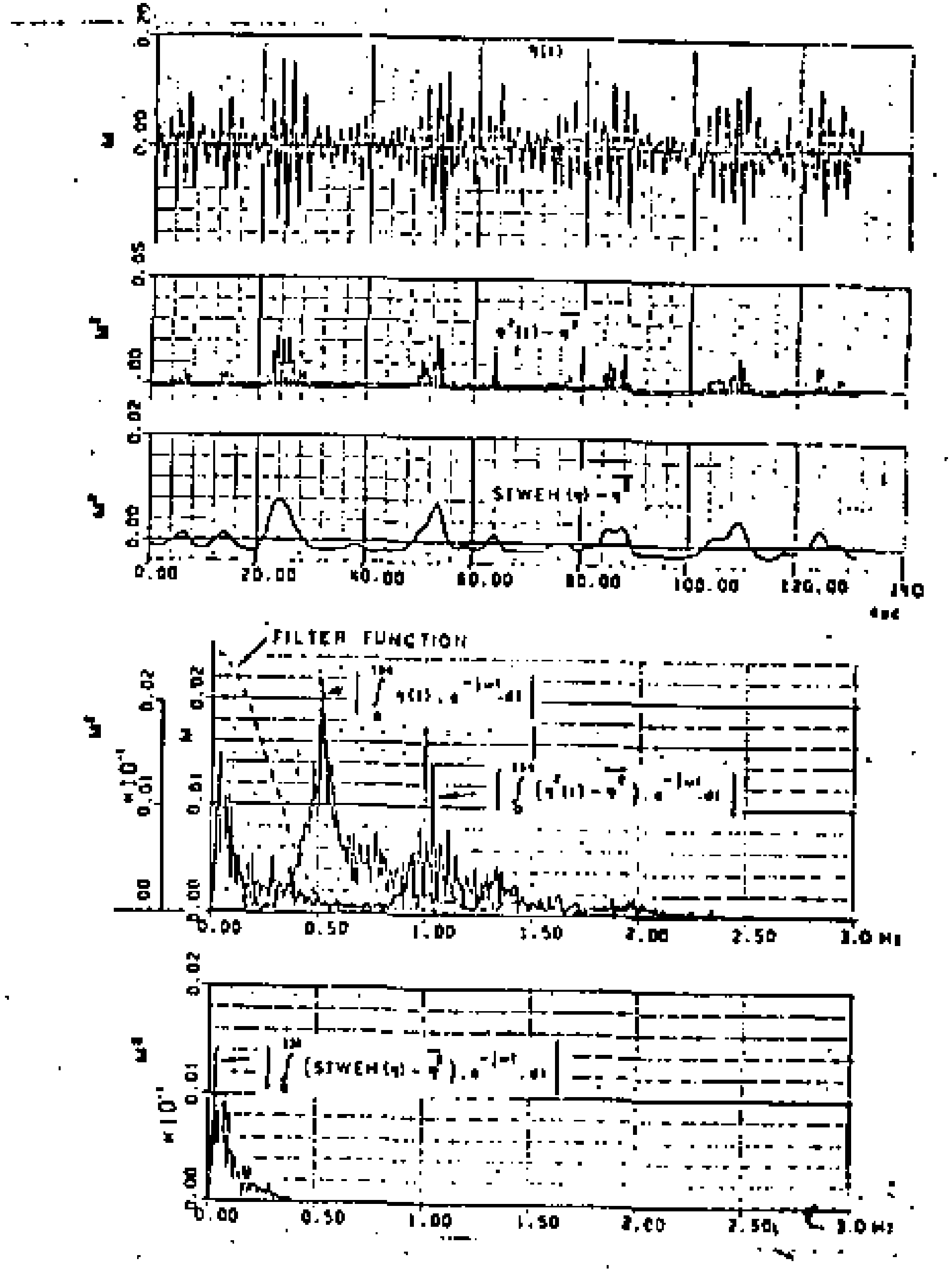

Fig. 3.3 Spectrum of squared water surface elevation (ref. Funke (1979)).

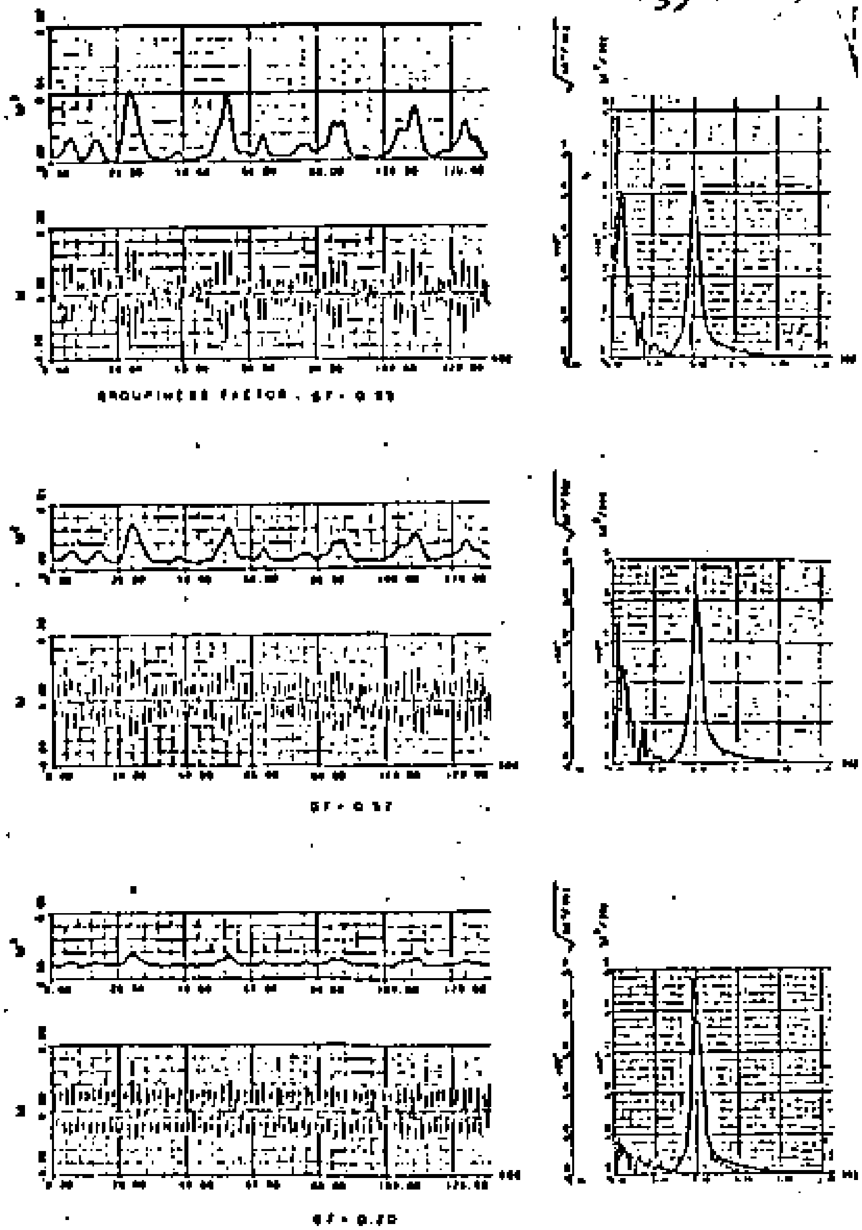

Fig. 3.4 Three wave trains with common variance but different SIWEH spectral densities (ref. Funke (1979)).

It is assumed that $E(f)$ and $\Xi(f)$ together provide information enough to reproduce the statistics of wave phase. Nevertheless the phase spectrum should also be calculated and considered used in connection with reproduction of waves in the flume.

Successfully detecting wave groups using the techniques described above will have broad implications for the ways in which wave data statistics are applied to engineering problems.

3.6 Maximum entropy spectral estimation

Automatic regressive (AR), moving average (MA) and mixed ARMA models have been used to describe geophysical and economical processes during the last decade. Examples are Ulrych et.al. (1973) and Wells (1972). Recently the authors found it worth while to join efforts in an attempt to use this technique to estimate spectral densities of ocean surface waves.

It is well known that contributions mainly by Pierson and Longuet-Higgins have led to the Gaussian hypothesis for the description of a random sea surface. Deviations from this model have been reported by several authors, but it is still the number one sea surface model.

One strong reason for our interest to describe ocean surface waves by an ARMA model, was its congruence with the method of maximum entropy (MEM).

One weakness of traditional spectral estimation (including Fourier transformation of the auto covariance function and the Fast Fourier Transform) is the assumption made concerning the data outside the sample being zero.

In traditional spectral analysis this is taken care of using either data or spectral windows. This problem is avoided in MEM.

MEM describes information available from the sample, but it does not commit itself with assumptions with what is going on outside. In particular, we derive our estimates from realizations of the process, and we assume nothing on what is going on outside that realization. The philosophy that our knowledge of the unknown process should purely rely upon realizations of it, and not on unrealistic assumptions beyond the realization, led to the work presented in this report. The problem of windowing is eliminated, but it is replaced by the problem of model estimation.

The approach used in this paper is an autoregressive moving average (ARMA) process. The surface is then modelled by approximately 20 coefficients and a noise term. Verifications shows that the ARMA model gives estimates of spectral shapes and moments that are in good accord with Fast Fourier Transform (FFT) spectra. The spectrum is given on an exact form when the ARMA coefficients are known.

Because the spectrum as well as the model time series are given when the coefficients are known, we have obtained a short cut between time and frequency space in comparison to traditional spectral analysis.

A data aquisition and analysis procedure, which includes the use of ARMA models, is discussed. Concerning this topic, more details are available in the report by e. Holm and J.M. Hovem (1978), who proposed a system for real-time processing and data reduction in the wave measuring buoy.

A method for simulation of the sea surface is outlined. It is also indicated how the response of a linear system can be described such that response time series can be generated. The ARMA models can also be applied to generation of waves in model tanks.

In the following, a brief outline of ARMA processes is given. For a more detailed discussion reference is made to Houmb, Overvik, Fines and Ho (1979) and Box & Jenkins (1970).

An ARMA process can be regarded as generated by passing a white noise series $\{a_t\}$ → |AR filter| → |MA filter| → z_t

The filter have transfer functions $\phi^{-1}(B)$ and $\theta(B)$ respectively. The ARMA (p,q) process may be written:

$$z_t = \phi^{-1}(B) \cdot \theta(B) a_t$$

$$= \phi_1 z_{t-1} + \phi_2 z_{t-2} + \ldots + \phi_p z_{t-p} + a_t - \theta_1 a_{t-1} - \ldots - \theta_q a_{t-q}$$

where

$$\phi(B) = 1 - \phi_1 B - \ldots - \phi_p B^p$$

$$\theta(B) = 1 - \theta_1 B - \ldots - \theta_q B^q$$

The two sets of constants $\{\phi_i\}$ and $\{\theta_i\}$ together with the white noise variance σ_a^2, define an ARMA (p,q) model.

The autocovariance at lag k of the process is:

$$\gamma_k = \phi_1 \gamma_{k-1} + \phi_2 \gamma_{k-2} + \ldots + \phi_p \gamma_{k-p} - \theta_1 \gamma_{za}(k-1) - \ldots - \theta_q \gamma_{za}(k-q)$$

where

$$\gamma_{za}(k) = E[z_{t-k}\, a_t]$$

The power spectrum of the ARMA (p,q) process is:

$$P(f) = 2\sigma_a^2 \frac{|1-\theta_1 e^{-i2\pi f} - \ldots - \theta_q e^{-i2\pi f}|^2}{|1-\phi_1 e^{-i2\pi f} - \ldots - \phi_p e^{-i2\pi f}|^2}, \quad 0<f<\tfrac{1}{2}$$

where the frequency f is normalized with respect to the sampling interval Δt of the discrete stochastic process.

One of the major problems using ARMA models is that of determining the order (p,q). In the case of wave data experience shows that q = 1 and 10<p<30 give good model estimates. For further details of order estimation, reference is made to Houmb et.al. (1979).

Fig. 3.5 is a typical example of a spectrum estimated by Fast Fourier Transform (FFT) and by an ARMA (30,1) process. The difference of spectral parameters and spectral shape are very small.

One of the interesting properties of MEM spectrum analysis is that the autocorrelation function (AKF) can be extrapolated in a way consistent with the model assumptions. Extrapolation of the AKF is equivalent to an increase of the number of lags. In spectral terms this leads to an increased resolution in frequency space as compared to traditional methods (FFT).

ARMA models have been successfully applied for estimation of spectra of linear response, and for simulation of wave samples. Other applications are generation of waves in flumes and data collection procedures.

For a Waverider the following data collection procedure is proposed:

The accelrometer delivers an analogue signal (representing vertical acceleration). This signal is integrated two times and the resulting signal (representing wave height) is digitized to obtain a time series with sampling interval 0.5 seconds. This series is passed through a digital filter with transfer function $(1-\phi_1 B)^{-1}$. Since the filtering is done continually, no transcient is present in the resulting series, and no values of the series need to be discarded.

At intervals found appropriate, a microprocessor stores a specified length of the series, (at present recording is done 17 minutes every third hour). The partial autocorrelation coefficients are calculated and the cut-off lag estimated. The cut-off lag defines the order of the AR-model. The resulting coefficients are coded and transmitted to the recording station by use of satellite telemetry.

What is gained and what is lost by using this procedure?

The main advantages over the present method are:

- A greater amount of data reduction, all information is contained in 30-40 numbers. The storage problem is reduced.
- Transmission of coefficients instead of an analogue signal. By use of coding techniques the problem of radio interference is reduced. Due to errors in the wave records, introduced mainly by the transmission of the analogue signal, the wave records by use of the present method must be corrected before any further analysis.
- The output of the latter procedure is ready for use directly, the calculation of the power spectrum from the coefficients is very simple. The model is also well suited for simulation purposes. In combination with transfer function models for the calculation for response of a system excited by wave loads, it is also simple.

There is no problem to introduce a microprocessor in e.g. a Waverider to do this, Hovem and Holm (1977). The Waverider could also be equipped for satellite telemetry, and the commercial price for a such buoy would be less then $ 20.000. The system would therefore have an unlimited telemetry range. In addition, the receiving station could be equipped with a simple microprocessor that can control the telemetry and e.g. order the buoy to retransmit coefficients until it has received in succession two series of coefficients that are identical. The microprocessor could as well give the spectrum along with the moments and all parameters derivable therefrom.

The main disadvantage in using our proposed method is that there exists no wave record, and no zero-crossing parameters.

4. LONG TERM AND EXTREME WAVE STATISTICS

4.1. General

Design of any marine structure requires an accurate knowledge of the probable extreme sea state conditions for very large geographical areas. This needs a very long data base. Goda (1979) classifies the different methods into three categories.

1) Extrapolation of longterm distribution of individual or characteristic wave heights.

2) Extreme value analysis with peak wave heights of major storms above a certain level.

3) Extreme value analysis with annual maxima.

Under group one, different models are used and each method is summarized below. None of the methods are universally accepted and the estimates vary widely. Generally, about 30 years' data is required for a good estimate, to account for the climatic fluctuations from year to year.

4.2 Models for extreme wave prediction

4.2.1 Individual wave height model A and B

The long term distribution of H_s is found to be Weibull. The distribution function is decided by only goodness of fir and there is no theoretical relationship to support it.

$$P_L(H_s) = 1 - \exp\left(-\left(\frac{H_s - H_o}{H_c - H_o}\right)^{\gamma}\right)$$

where H_o, H_s and γ are the location or threshold parameter, the scale parameter and the slope parameter respectively to be determined from the data. The grouped data is plotted on a Weibull probability paper with $\ln(H_s - H_o)$ as abscissa and $\ln(-\ln(1-P_L(H_s)))$ as ordinate. $P_L(H_s)$ is obtained as $m/(N+1)$ where m is the cumulative number of data less than the upper limit of the class re-

presenting $(H_s)_i$. γ is the slope of the straight line and is determined by least square method. It is generally found that the plotting position of the lowest H_s-class (upper limit) has a tendency to deviate from the fitted straight line (two parameter). In order to account for this the parameter H_o is introduced. The value of H_o is normally the deviation from the straight line by the lowest plotting position. In some cases a better fit (decided by the correlation coefficient or X^2-value) can be obtained by trial and error method, changing H_o value. H_c is the value of H_s for which $\ln(-\ln(1-P_L(H_s))) = 0$. Generally, a two parameter ($H_o = 0$) distribution is first tried and a second trial is attempted only when found necessary and the two parameter distribution is used for the computation of the individual wave heights using Nordenstrøm (1969) method.

The probability of occurence of a particular sea state $H_s(R_p)$ is:

$$Q(H_s) = 1 - P_L(H_s) = \tau H_s/R_p = 1/N$$

where R_p is return period and τH_s is the duration between observations during which period the sea state is assumed to be stationary. Generally τH_s is taken to be 3 hours for instrumentally observed data and as 12 mins for visually observed data. But this is a point of subjective definition and is debated much currently. If we substitute for R_p we get the sea state which would be exceeded on an average once every R_p years as:

$$H_s(R_p) = (H_c - H_o)[\ln(R_p/\tau H_s)]^{1/\gamma} + H_o$$

The return period of a given wave height is propotional to τ and hence small τ values give conservative estimates.

The long term distribution of individual wave heights can be obtained by summing up all the individual seastate height distributions (Rayleigh) weighted by the sea state variation. This weighting is done by a joint distribution function of H_s and T_z and T_z^{-1}.

$$P_L(H) = \frac{1}{\overline{T_z^{-1}}} \int_0^\infty \int_0^\infty T_z^{-1} p(H_s, T_z) F(H/H_s, T_z)\, dH_s \cdot dT_z$$

The integration is performed by neglecting the variation of T_z to get

$$P_L(H) = 1 - \int_0^\infty p(H_s) e^{-2(H/H_s)^k} dH_s$$

This gives conservative estimates since the correlation of H_s and T_z are neglected. This equation is approximated by a Weibull distribution

$$P_L(H) = 1 - \exp[-(\frac{H}{C H_c^{1/d}})^D]$$

where d = 1.33 for visual data and 1.0 for instrumental data. C and D are determined from Table 4.1 in which γ is the slope of $P_L(H_s)$. The wave height of return period, R_p is

$$H(R_p) = C H_c^{1/d} [\ln(R_p/\tau_H)]^{1/D}$$

Eide (1979) in his critical review states that this model would have a tendency to overestimate the wave heights as it disregards the mild correlation between wave height and periods. This need not be considered a serious drawback. Nolte (1973) pointed out the necessity for a grouping correlation as it is likely that two design high waves, $H(R_p)$ may successively occur in a single storm.

The most probable highest wave (model B) in a particular sea state of return period R_p is given by:

$$H(R_p)_{max} = H_s(R_p) \cdot \sqrt{\ln N/2}$$

where N is the number of waves τ_{H_s}/τ_H, assuming a reasonable average individual wave period (τ_H). This is derived by assuming the long term distribution of H_s to be Weibull and does require an integration as done by Nordenstrom (1969). One serious limitation of the method is the dependence of H_d on τ_{Hs} and τ_H. In

	d=1.33	d=1.00	
γ·d	C	C	D
∞	1.189	.707	2.000
10.00	1.056	.628	1.780
8.00	1.029	.612	1.712
6.00	0.992	.590	1.614
4.00	0.930	.553	1.444
3.33	0.901	.536	1.354
2.86	0.876	.521	1.276
2.50	0.855	.508	1.208
2.22	0.837	.497	1.144
2.00	0.820	.488	1.086
1.82	0.807	.480	1.034
1.67	0.794	.472	0.988
1.54	0.783	.465	0.944
1.43	0.772	.459	0.904
1.33	0.762	.453	0.868
1.25	0.754	.448	0.834
1.18	0.746	.444	0.802
1.11	0.739	.439	0.774
1.05	0.732	.435	0.746
1.00	0.726	.432	0.722
0.67	0.689	.410	0.538
0.50	0.666	.396	0.428
0.40	0.656	.390	0.356

Table. 4.1 Constants for the long term Weibull distribution for individual wave heights.

this method the duration of sea state is a matter of subjective definition. Eide (1979) concludes that the most consistent way is to use τH_s = 3 hrs. in $(N_p/\tau H_s)$ and τH_s = 12 mins. in $N = \tau H_s/\tau H$. This leads to the most probable largest wave in the N_p-year sea state when sea states are observed over 12 mins. every 3 hours.

4.2.2 Most probable maximum wave height model

This model utilizes the most probable maximum wave height (H_{max}) for each observation H_s i.e. each sea state.

An estimate of the H_{max} is given by

$$H(N)_{max} = H_s \cdot \sqrt{\ln N/2}$$

where N is the number of waves in each record and is computed by dividing recording duration τH_s by an average period τ_m.

Hence we obtain:

$$H(1000)_{max} = 1.86\ H_s$$

and

$$H(1000)_{max} = 2.15\ H_s$$

Hence as a rule of thumb H_{max} does not vary much from 2 H_s. The H_{max} values thus obtained are fitted by Weibull law and the desired statistic is evaluated. Saetre (3.3.2.1) used Gumbel probability law in his study. As a limitation Eide (1979) points out that H_{max} is not strictly an extreme value variable as each sample sea state is from different populations.

4.2.3 The storm model

This model in one way applies a filter to all the sea states

occurred during a period of time for which the hindcasting was done for numerical models. A storm is defined to start when the H_s exceeds a threshold value, H_{ST}, and to end when H_s has stayed below H_{ST} for a day or drops to less than $H_{ST}/2$ after half a day. From the H_s values the most probable maximum wave heights (H_{max}) are computed. There exists two versions of storm model, one uses the most probable maximum wave height during a storm and the other uses the complete distribution of possible maximum heights during the storm. Hence a sea state lasting for several observation periods may give a higher H_{max} than a more severe sea state observed only once. The computed H_{max} are fitted by a three parameter Weibull distribution. Then the probability of the m year wave is given as:

$$Q(H_m) = 1/\lambda_m$$

The design wave is computed from

$$Hd = H_c(\ln \cdot \lambda \cdot m)^{1/\lambda} + H_o$$

where λ is the number of storms per year averaged over the observation period. Hd is not the same as the individual wave of return period N_p.

4.2.4 Annual extreme wave height model

This model used one observation from each year - the largest one. Eide (1979) considers two ways to select the suitable extreme annual wave.

1) This uses the annual maxima of significant wave heights to find the m-year sea state value, H_{sm}, and from that the m-year design wave as

$$Hd = H_{sm}\ (\tfrac{1}{2} \ln nd)^{\frac{1}{2}}$$

where nd is the number of waves in the sea state H_{sm}.

2) The second uses the highest most probable largest wave among all sea states taking into consideration that the largest H_{max} does not necessarily occur during the largest H_s.

After computing the suitable annual extreme value they are ranked from the lowest to the highest resulting in say n independent values each from a population of N samples. If all the n distributions of N samples are identical and of the exponential type the largest value of N samples is given by Gumbel (1958).

$$\text{Prob.}\ (H_N \leq H) = P(H) = e^{-e^{-\alpha(H-U)}}$$

where α and U are functions of the initial yearly distribution and are generally determined from a plot of the annual maxima. The plotting is done with $x = H$ and $y = -\ln(-\ln \cdot P(H))$ and a straight line is fitted by least square method to give $y = \alpha(H-U)$, where α is the slope of the line and $(-U/\alpha)$ is the regression constant. Then the m-year wave height is

$$Hd = U - \frac{1}{\alpha}\cdot \ln(-\ln(1 - \frac{1}{m}))$$

Gumbel (1958) specifies that n should be at least within 15 - 20 to minimize the uncertainties. Initial and their extreme value distributions are given in Table 4.1.

4.3 Results on extreme value prediction

Eide (1979) analyzed visual wave data from Famita and H, Fig. 2.2 using the models

A - the individual wave model
B - the maximum significant wave model
C - the most probable maxim wave model
D - the storm mode
E - the annual extreme wave model

Distribution	Initial	Extreme Value	Gumbel's Extreme Value
Rayleigh	$1 - e^{-2(H/H_s)^2}$	$e^{-e^{-[2(H/H_s)^2 - \ln n]}}$	$\exp\left[-e^{-2(\ln n)^{\frac{1}{2}}(\sqrt{2}\frac{H}{H_s} - (\ln n)^{\frac{1}{2}})}\right]$
Weibull (H_s)	$1 - e^{-\left(\frac{H_s - H_o}{H_c - H_o}\right)^{\gamma}}$	$e^{-e^{-\left[\left(\frac{H_s - H_o}{H_c - H_o}\right)^{\gamma} - \ln n\right]}}$	$\exp\left[-e^{-\gamma(\ln n)^{1-\frac{1}{\gamma}}\left[\frac{H_s - H_o}{H_c - H_o} - (\ln n)^{\frac{1}{\gamma}}\right]}\right]$
Weibull (H)	$1 - e^{-\left(\frac{H}{CH_c/\beta}\right)^{\beta}}$	$e^{-e^{-\left[\left(\frac{H}{CH_c/\beta}\right)^{\beta} - \ln n\right]}}$	$\exp\left[-e^{-\beta(\ln n)^{1-\frac{1}{\beta}}\left[\frac{H}{CH_c/\beta} - (\ln n)^{\frac{1}{\beta}}\right]}\right]$

Table. 4.1 Different distributions generally used for

The results are shown in Table 4.1.

Station	A	B, 12 min.	B, 3 hrs.	C, 12 min.	C, 3 hrs	D	E
Famita	32.7	24.5	27.0	24.0	28.3	27.2	35.4
OWS "M"	27.7	20.3	23.0	20.8	24.9	24.7	26.8

Table 4.1 Predicted wave heights in meters of return period 100 years. For mode B and C distances between observations of 12 min and 3 hrs. are used.

Table 4.1 shows a large spread of the extreme wave predictions Models A, D and E are considered most correct of reasons mentioned in Section 4.2.

By elimination of erratic data and reanalyzing the reduced data sets Eide (1979) obtained the results as shown in Table 4.2. Here models B and C were not used, and the spread is very much reduced.

Station	A	D	E2
"Famita" 1963-1977	32.7	29.7	34.2
OWS "M" 1961-1977	26.8	25.3	24.7

Table 4.2 Predicted wave heights of return period 100 years after elimination of erratic data. Eide (1979).

Annual extremes as calculated using the Norwegian hindcast model are shown in Table 4.3

(1977) showed that both log-normal and Weibull distributions can be employed to describe the storm waves. Vik and Houmb (1977) studied the sea state persistency characteristics in the Norwegian sea from waverider data. They defined a storm by assigning a definite value level for the significant wave height. They computed the cumulative distributions of H_s, the average number of storms defined by two levels, average duration and the standard deviations; monthwise, seasonwise and yearwise. Their data base covered only 31 months and is short to make very reliable conclusions. Houmb and Vik (1977) developed a model for estimates of storm durations which exceeded definite assigned values. The average duration of H_s above level H_s' in hours is given as:

$$\tau_s(H_s') = \frac{(2\pi)^{\frac{1}{2}}\,(H_c - H_o)^{\gamma}}{\gamma\cdot\sigma_h\,(H_s' - H_o)^{\gamma-1}}$$

where H_o, H_c, γ are parameters of the long term distribution $P_L(H_s)$ of H_s, σ_h is the standard deviation of dH_s/dt. The average duration of sea states below the level H_s' is given as:

$$\tau_c(H_s') = \tau_s(H_s')\left(\frac{1}{1-P_L(H_s')} - 1\right)$$

They found that no systematic variations of σ_h with H_s over the year for different months. The standard deviation of sea state was of the same order as the average value giving large spread and hence the unreliability.

Calculated and predicted durations by Houmb and Vik (1977) are shown in Fig. 4.1.

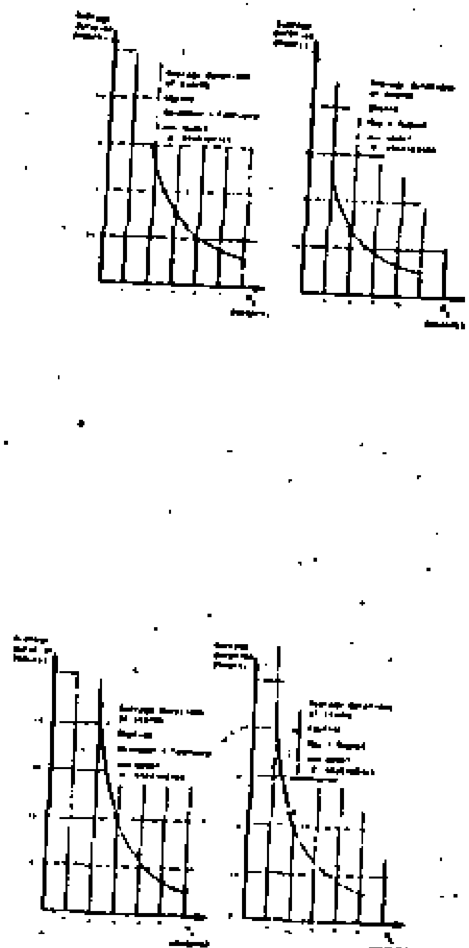

Fig. 4.1 Calculated and predcited durations of sea state

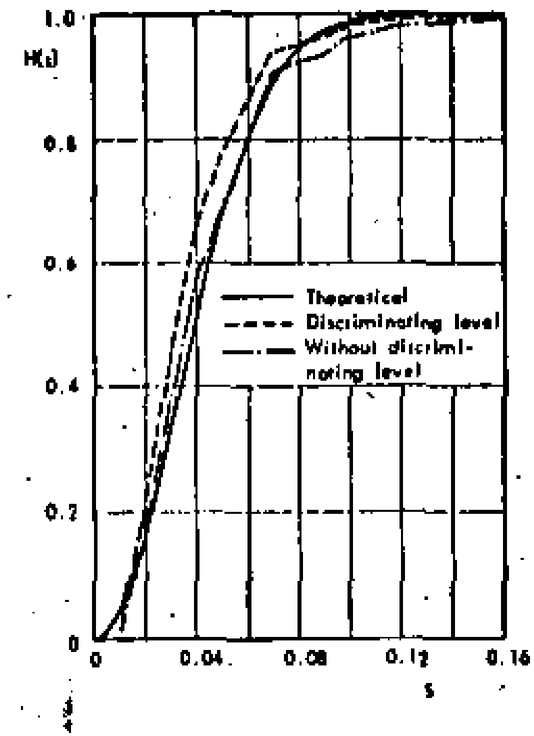

Fig. 3.1 Theoretical and measured distributions of wave steepness. The case of discriminating level is low pass filtered.

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DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

SEMINARIO DE INGENIERIA OCEANICA

PROF. PER BRUUN

MARZO DE 1982.

Reprinted from the Proceedings of the 13th Coastal Engineering Conference, July 10-14, 1972, Vancouver, B.C., Canada, by the American Society of Civil Engineers

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") 1576-1579.

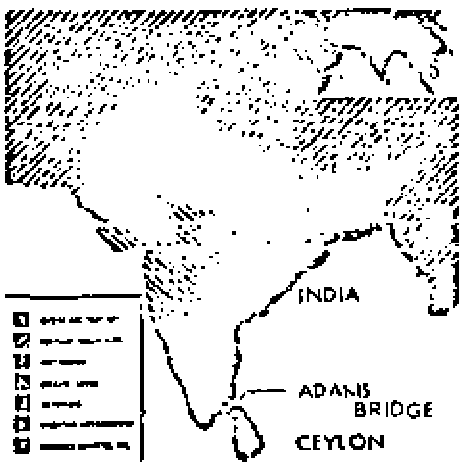
"Water shall not be compelled by any 'force', or it will return that force unto you".

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 away 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

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 any rate, 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 1260 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 Mycenius 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 poetical 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 heighten 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 Ems. 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 s. 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 ship-wrecks 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:
Water will not be compelled by any 'forsse' (force), or it will return that force unto you.

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 12 the following areas have gai 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 1850, that is just before the advent of steam dredging, we had about 1750 miles of them, containing about 250,000,000 cubic yards of material. Moreover, there were many old deserted dikes, whose contents may be estimated at 50,000,000 cubic yards. Those 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 4800 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."[4]

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 1714 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, rough 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 3-4th, 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

Grevelingen 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 telpher, (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 360 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 3,500 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 on 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 N.S.L. Occasionally groins are provided with piggy backs (Plate 3, Fig. 12) to break the longshore currents. Analyses by Bakker and Joustra (2) 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.43), which gradually increased in length to several hundred meters at the Thyboron Barriers due to continued shore recession, is just one of these misinterpretations by

which enormous quantities of materials were sacrified 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 Maletscha there were 46 Dutch villages in 1836; the district Chorcitza 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 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 composed 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 t, now often provided with side slopes of 2 to 8 ts 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 1946, 1956, 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. permable 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 force 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 drift and partly to the sides by imbalance in longshore drift or

$$|dE_{erosion}| = dT_{transversal\ drift} + dL_{longshore\ drift}$$

$dT_{transversal}$ - 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, realising 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. 15), it can be understood how an average rise of just 1/8 in per year may cause shoreline recessions ranging from 2 to 5 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 (15). 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	.210 "
New London, Connecticut	1939-70	.229 "
New York, New York	1893-70	.287 "
Sandy Hook, New Jersey	1933-70	.457 "
Baltimore, Maryland	1903-70	.259 "
Washington, D.C.	1932-70	.344 "
Portsmouth, Virginia	1936-70	.341 "
Charleston, South Carolina	1922-70	.180 "
Fort Pulaski, Georgia	1936-70	.198 "
Mayport, Florida	1929-70	.155 "
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".

$dL_{longshore}$ - 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) or:

$$\frac{dQ}{ds} = k\frac{dE}{ds}$$

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

$$\frac{dQ}{ds} \gtrless k\frac{dE}{ds}$$

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 affect 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 leeside erosion, up to 10 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, 24, 69). A great number deal with seasonal changes (13, 14, 18).

Space photography is now providing a tool for large scale checking of coastal movements (23 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 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. 24-33.

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 63).

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	Artificial nourishment from land sources
Rivers	
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 volcanoes, when erupting, occasionally cause discharges of the order of 1 billion cubic yards of material including bed load "particles" of 20 tn.

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 4. 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

[illegible]

TABLE 7. DETAILS OF THE PERFORMANCE OF SEA WALLS.

		Comments:
1. What is wanted:	Storm tide and/or extreme storm protection of shore and beach. 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 erosion.
4. Design:	Energy-absorbing (sloping and/or mound type).	Contributes 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.	Less reflection, less loss of sand.
	Non energy absorbing.	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:[illegible] depending upon quantity of drift and beach material. Most common ratio is 1:[illegible].	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, that means in rivers 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 on littoral drift shore both are usually obtained.
2. Layout and geometry:	Parallel to shore or largely following depth contours.	Tombolo formation will result if 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 tested in 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: suggest 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 25-55 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:

$$Q = (E\ wc) \sin 2\alpha_b$$

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, Derbirh) 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 wherever 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 (54, 57, 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 (< 0.15 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 35 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 3 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 1973 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 2,500 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 14 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 63,000 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. 700,000 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 "platform-dredge" (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 22 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
Less factor for overrunning	-	0.5 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, particularly 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 1966-1972 on 17-18 m depth, crown elevation at approximately 7.5 m below M.S.L., maximum waves ab. 6 meters (ref. 55 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 Caracas 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 up-drift jetty connected break-water 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/pro-posed
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 ex-tended authorized
New Pass, Fla.	Ocean shoal dredging	Occasional transfer from ocean shoals
Newport, Calif.	Undetermined or being studied	Recirculation by trap at lower end of [illegible]-mile beach 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 up-drift breakwater
St. Lucie, Fla.	Depressed weir and trap	Construction recommended
Santa Barbara, Calif.	Transfer from shoal inside updrift breakwater	Extension of west jetty, con-struction 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 ex-tended 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 de-pressed 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 Nagapatan, State of Madras, India. The jetty is left open during the monsoon period.

Plate 30, 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 37 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 preceeding 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 advan-tageous 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 the planning. It is also experience that advocators of patented "super marked" 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 re-concile 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 always be an absolute requirement that what you do and 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 expand 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 16. 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 16. 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 fortse 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", 1958)

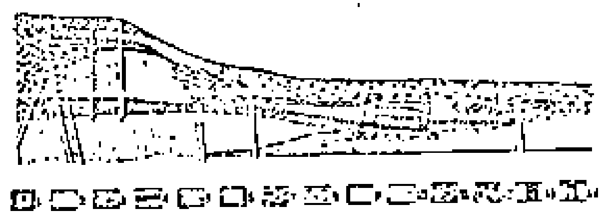


Fig. 3. Cross section of ancient dike with pile support. ("Antiquity and Survival", 1958)

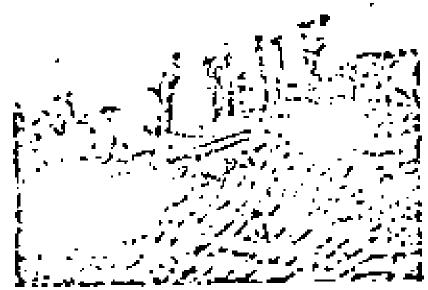


Fig. 4. Polder boys repairing dike with willows, Netherlands (van Veen, 1962)

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

PLATE 2

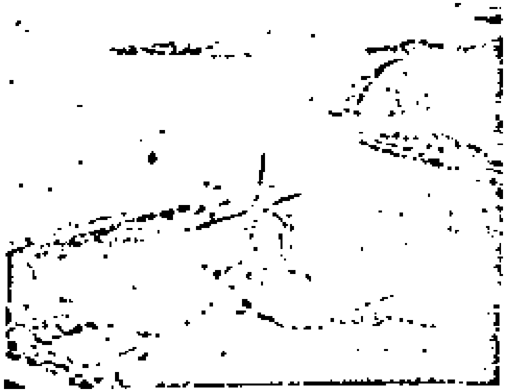


Fig. 6. Breakthrough at Vossel, Netherlands, Febr., 1953 (van Veen, 1962)

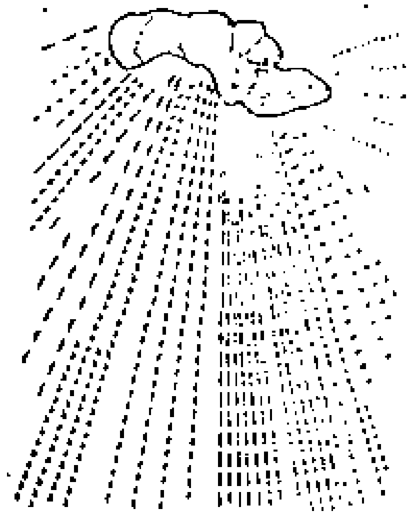


Fig. 7. Equipment parade at closing of breaches in the dikes of Schouwen-Duiveland (van Veen, 1962)

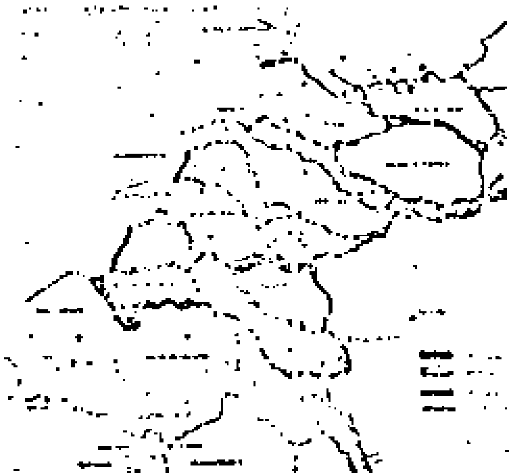


Fig. 8. The Delta Plan. (Rijkswaterstaat, 1971)

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

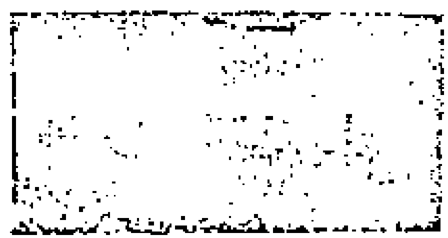


Fig. 10. Placement of elements of synthetic materials. (Deltawerken no. 51, 1971)

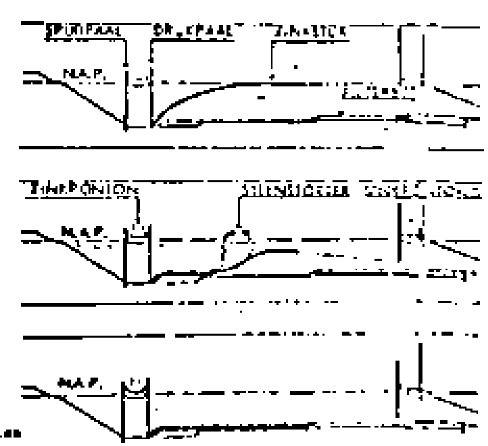


PLATE 3

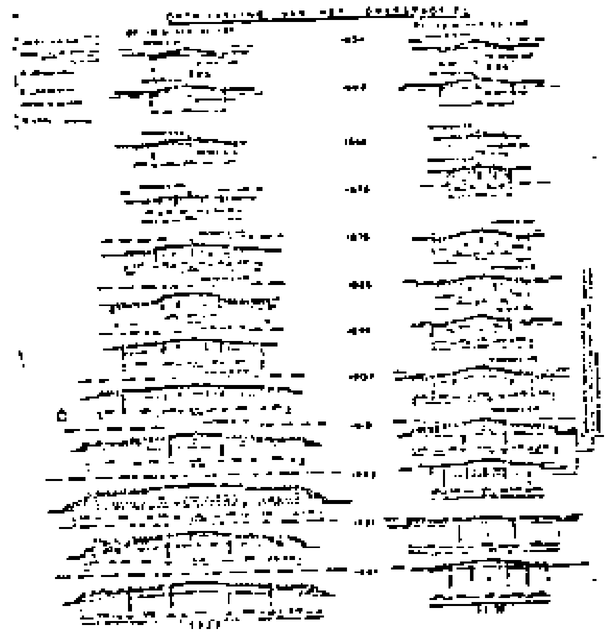


Fig. 11. Development of groins in Holland ([illegible], Visser, 1953)

Fig. 12. Dutch groin with [illegible] [illegible]

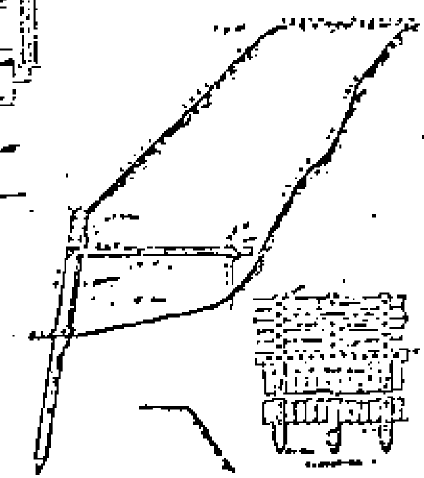


Fig. 13. [illegible] sea wall. (Journ. Inst. Civ. Engrs., 1877-78)

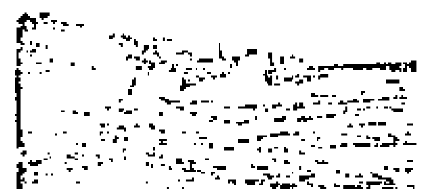


Fig. 14 a. Beaconsfield sea wall, Bridlington, under wave action

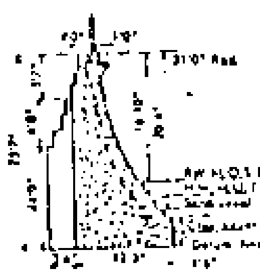


Fig. 14 b. Beaconsfield sea wall, Bridlington (Matthews, 1934)

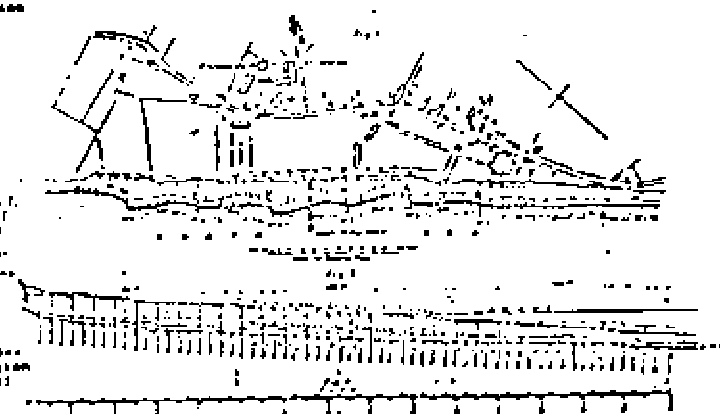


Fig. 15. [illegible] Groins (Journ. Inst. Civ. Engrs. 1877-78)

PLATE 4

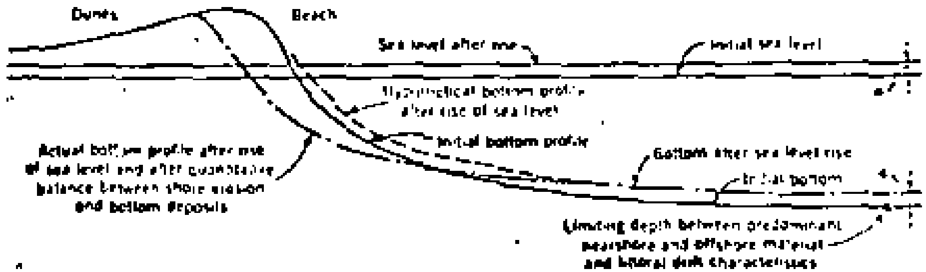


Fig. 16. The influence of sea level rise on erosion (Bruun, 1962)

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

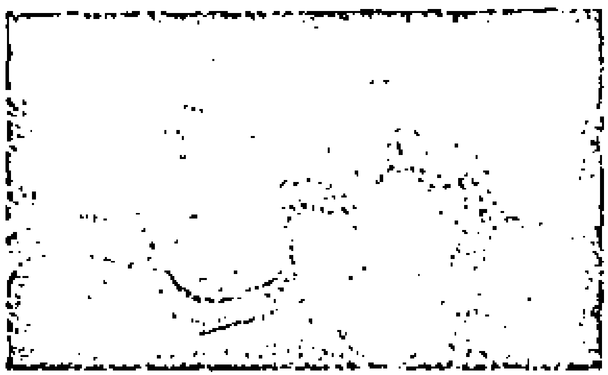


Fig. 18. Rock outcroppings make natural sea walls and form pocket beach in Puerto Rico

Fig. 19. Man o'War Rocks, Dorset, makes off-shore breakwater (Steers, 1947)

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

PLATE 5

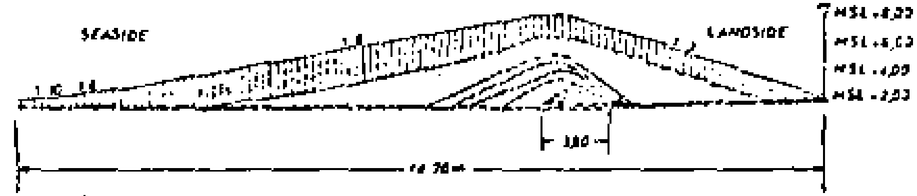


Fig. 21. Comparison of older with modern dike profiles (Kramer, 1971)

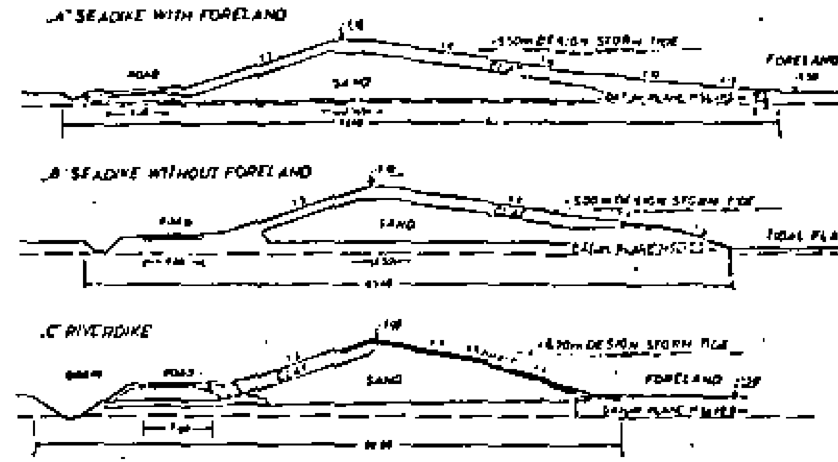
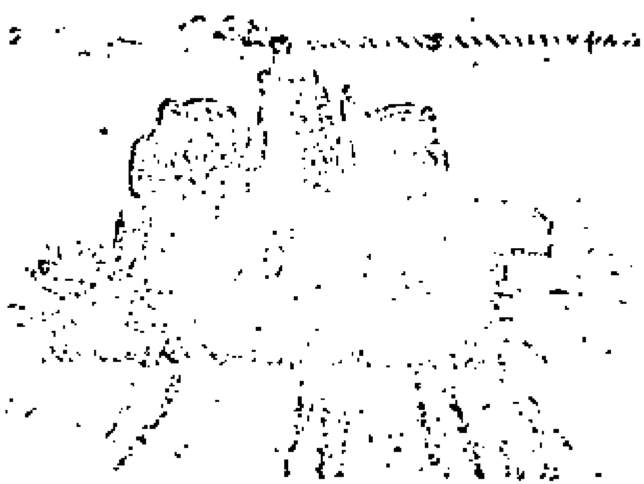


Fig. 22. Cross sections of dikes with sand core and clay or asphalt cover (Kramer, 1971)



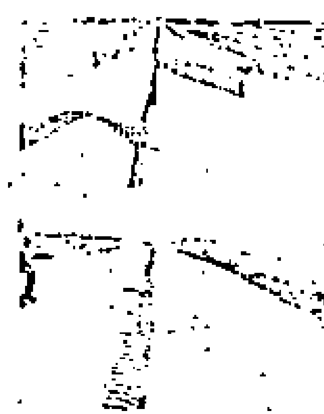


Fig. 24. Dune building by sand fences (NPS, USA)

PLATE 6

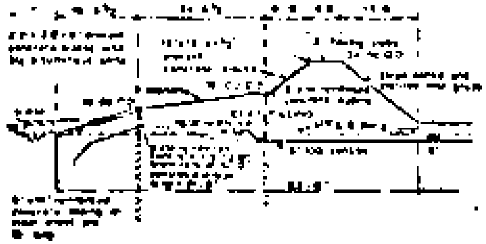


Fig. 25. Petit Leval Sea wall (Thorn, 1960)

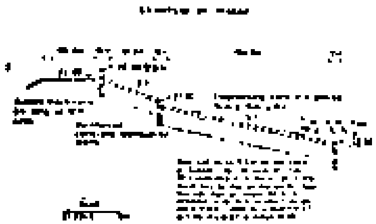


Fig. 26. Volland Sea wall (Thorn, 1960)

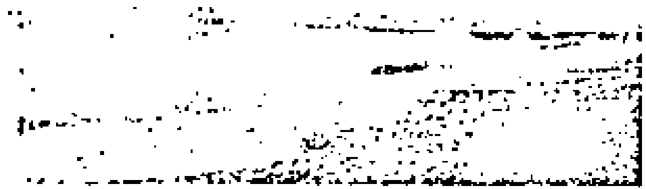


Fig. 27. Stone pitching revetment and stone pitching piggy back groin (Rijkswaterstaat)

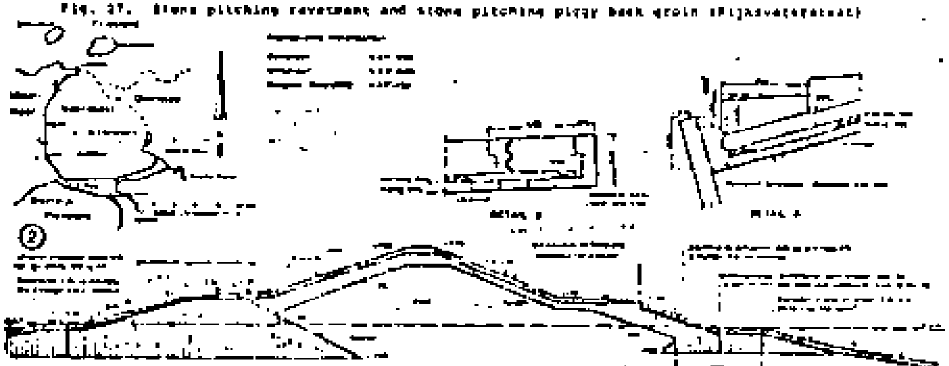


Fig. 28. Noordoostpolder revetment on dike (Rijkswaterstaat)
Legend: basaltzuilen (basalt prisms), steenslag (gravel layer), klinkers (layer of bricks), kramat (straw mat), kraagstuk (willow mattress), breuksteen (rock), beton (concrete), klei (clay dug from dunes and marshes)

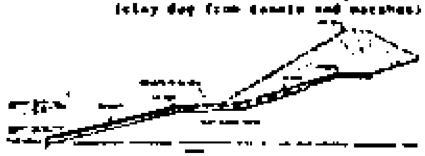


Fig. 29. Revetment at Zeelande (Rijkswaterstaat)

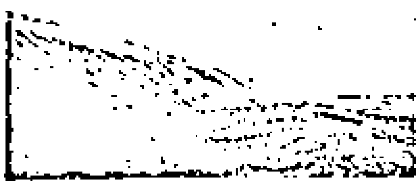


Fig. 30. Sea Carpet at Europoort, Netherlands (Hydrospace, Oct. 13)

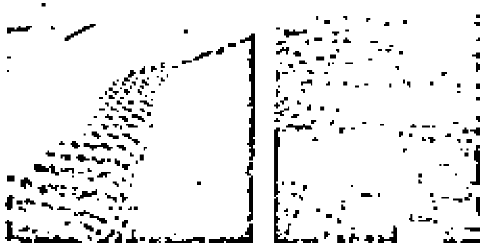


Fig. 31 a and b. Concrete blocks with friction arrangements (Rijkswaterstaat)

PLATE 7

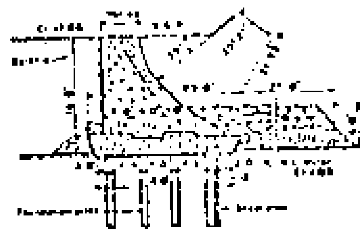


Fig. 32. The Galveston Wall, Texas (USCE = U.S. Army Corps of Engineers)

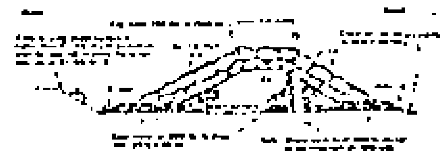


Fig. 34. The Fernandina Wall, Florida (USCE)

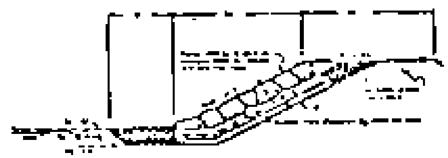
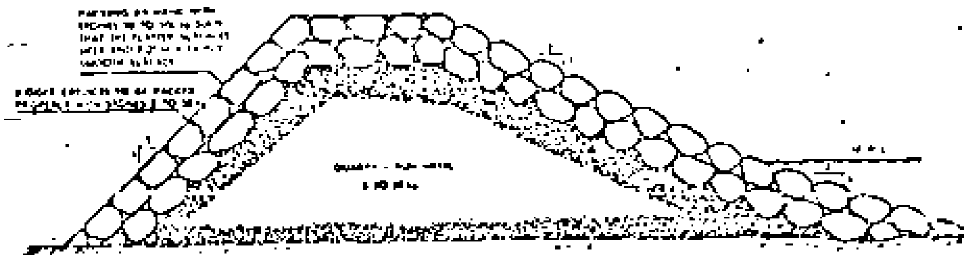


Fig. 33. The Ft. Story Wall, Va. (USCE)

Fig. 35. Indian Sea Wall (Kerala State)



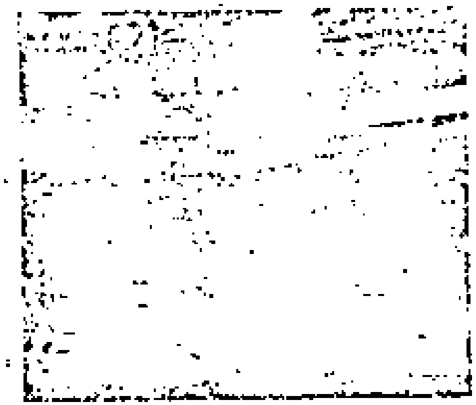


Fig. 36. Vertical wall on Jupiter Island under heavy wave attack, 1962 (Bruun and Manohar, 1963)

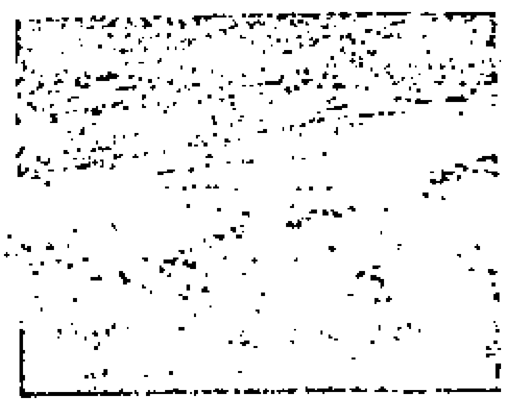


Fig. 37. Interlocking block revetment on Jupiter Island under heavy wave attack (Bruun and Manohar, 1963)

PLATE 8

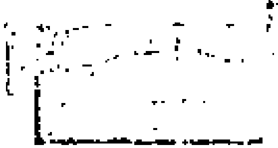


Fig. 38.
Dutch Fascine pitching groin at ...

Fig. 39. Dutch group of groins, Walcheren (Rijkswaterstaat)

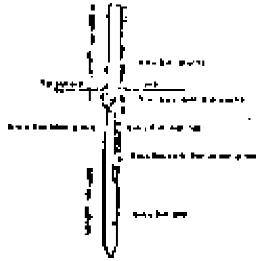


Fig. 40 a and b. British ... groin (...)

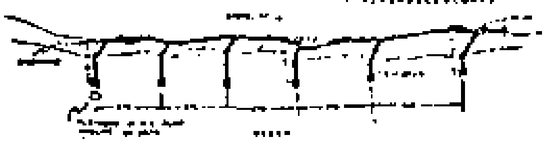


Fig. 42. Group of groins with slanting land ends at the ... Shore, ... (Bruun and Manohar, 1963)

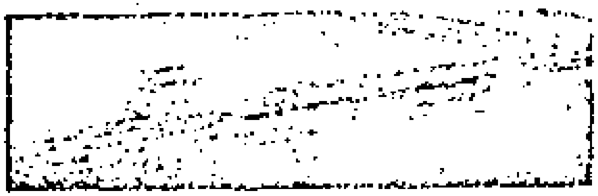


Fig. 43. Groin built of ... concrete blocks in the crown and ... to granite in the slopes. The ... lined ... Danish North Sea Coast (Mathiesen)

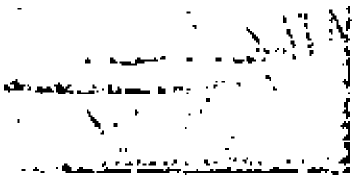


Fig. 41. Adjustable groin at Palm Beach (Graham)

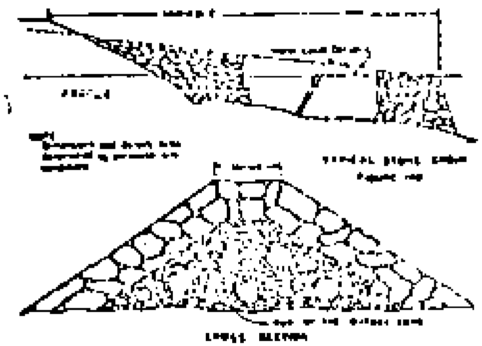


Fig. 44. American rock groin (CERC)

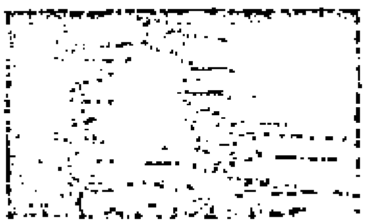


Fig. 45. Groins, mostly steel sheet piling, at Miami Beach, Florida

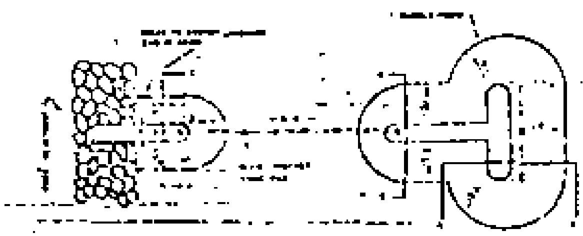


Fig. 46. T-groin design for Deerfield Beach, Fla. Adjustable king pile stem, rock T-head (Bruun and Manohar, 1963)

Fig. 47. T-groins at Deerfield Beach, Fla. (Bruun and Manohar, 1963)

PLATE 9

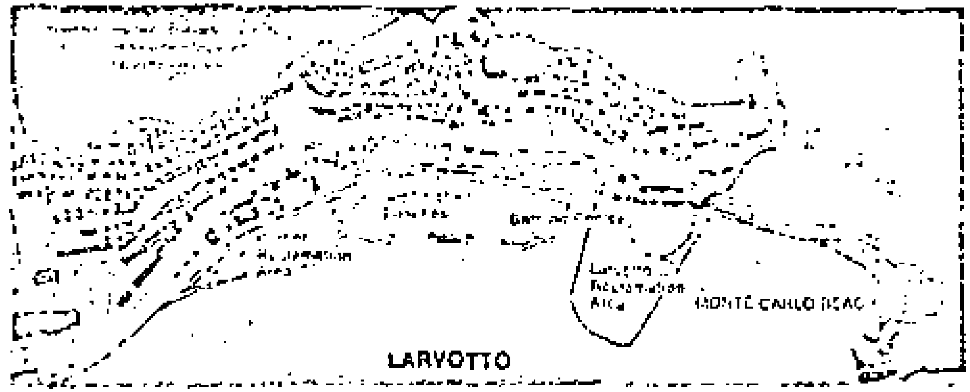


Fig. 48. Offshore breakwaters form pocket beaches, at Monte Carlo (Terra, Holland, No. 1, 1977)

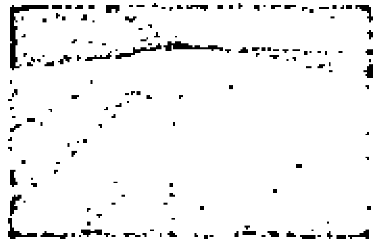
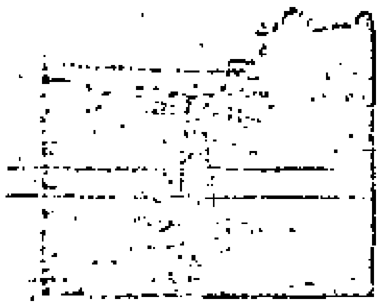
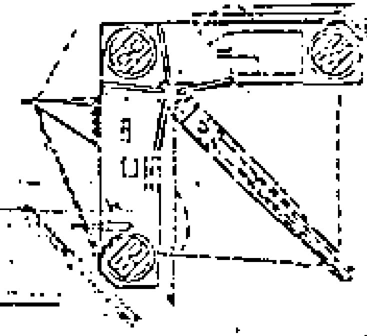


Fig. 49 a and b. Artificial nourishment from land sources, 1.6 mill cu yds on 16,000 ft of shore, Hilton Head Island, S.C.

a b





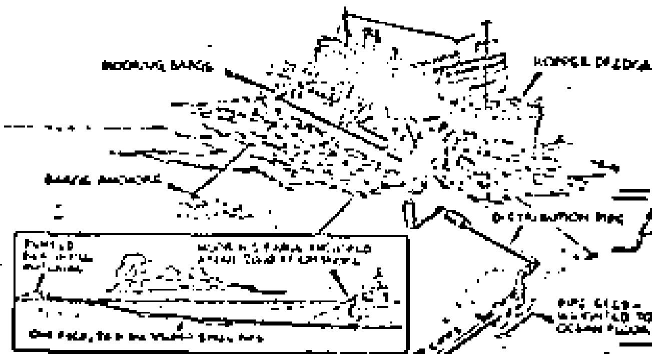


Fig. 50. Artificial nourishment from offshore by hopper dredge. ... (Mauriello, 1968)

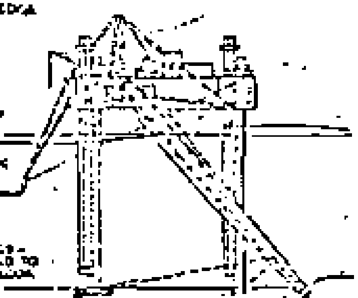


Fig. 51. Dutch platform dredge (Dutch Dredging and Construction, Jan. 1972)

PLATE 10

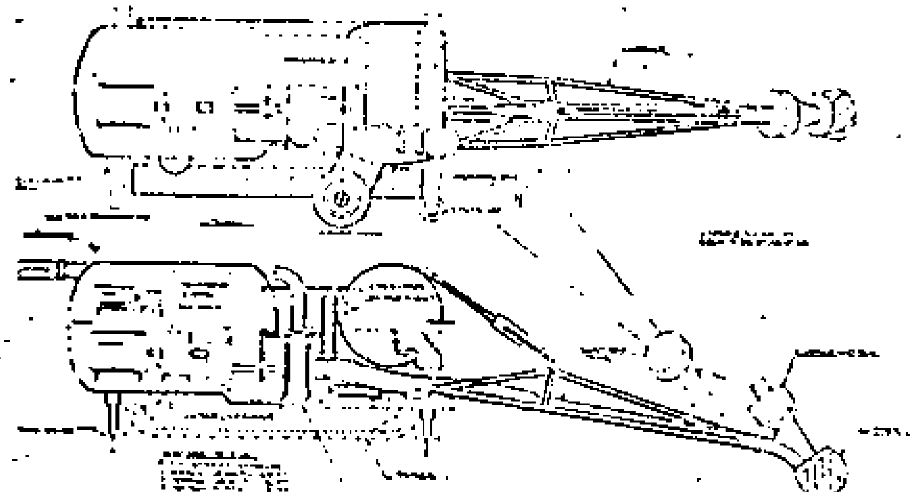


Fig. 52. Submersible Dredge (Hardesich, 1971)

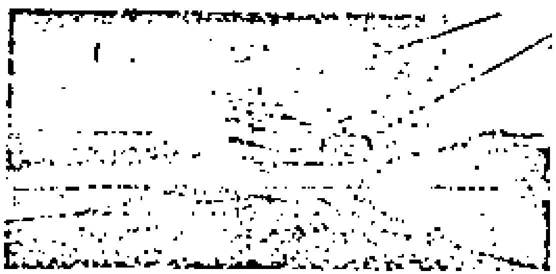


Fig. 53. Artificial Nourishment by dragscraper on Jupiter Island, Fla. (Brown, 1967)

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

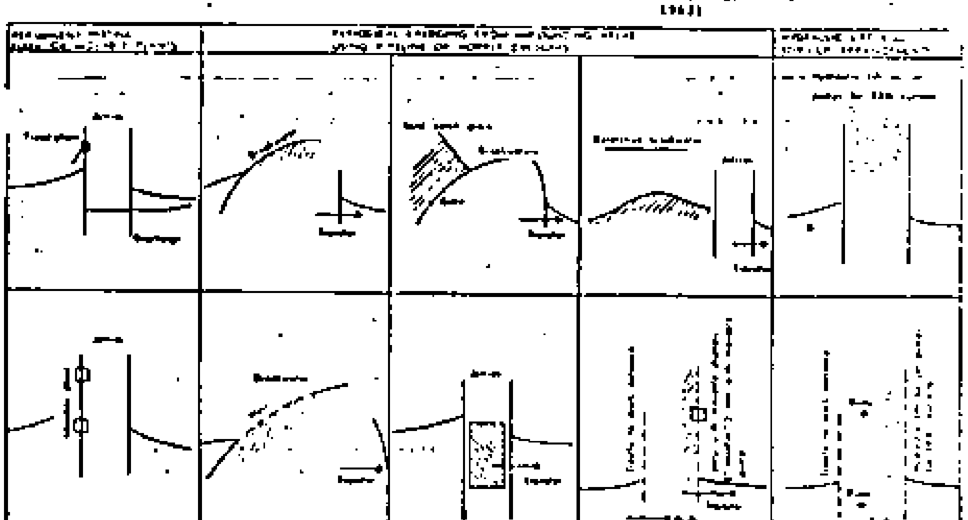


Fig. 55. Various bypassing arrangements (schematic)

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 (29, 30). See Plate 5, Fig. 24.

They may be protected by dune vegetation of various kinds (26, 27, 31, 32, 33). Most common are species of Ammophila - (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, 36, 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 Battjes(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 44) and to several papers on this topic in the Proc. from the 11th Conf. on Coastal Engineering, London, 1968, part 1. 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 affects 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 bituminous 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 the 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 as 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 uni-directional flow (C.F. Nichols, 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.

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Coastal Engineering, 5 (1981) 171—192
Elsevier Scientific Publishing Company, Amsterdam — Printed in The Netherlands

PRACTICAL VIEWS ON THE DESIGN AND CONSTRUCTION OF MOUND BREAKWATERS

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(Received February 4, 1980; accepted January 11, 1981)

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 and the fatal "lift-out".

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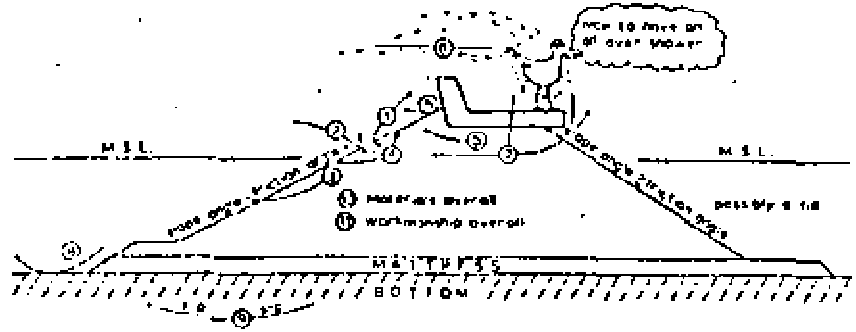

Fig. 1. Various reasons for damages to rubble mound breakwaters. Numbers refer to locations of failure.

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-

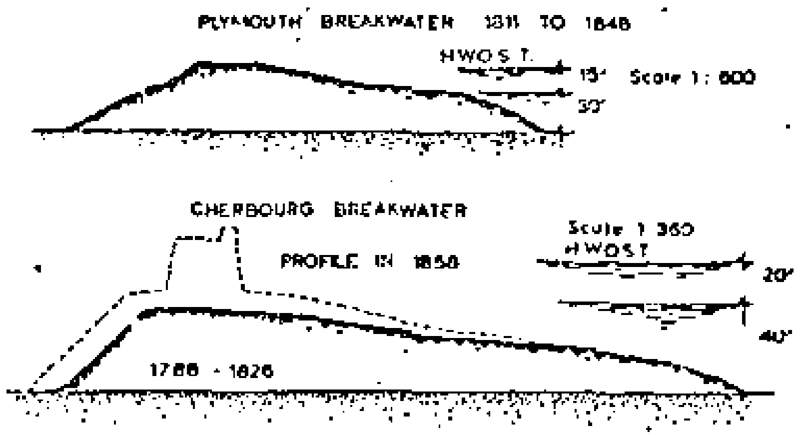

Fig. 2. Ancient breakwaters at Le Havre and Plymouth, where S-shape developed.

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 overall 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.

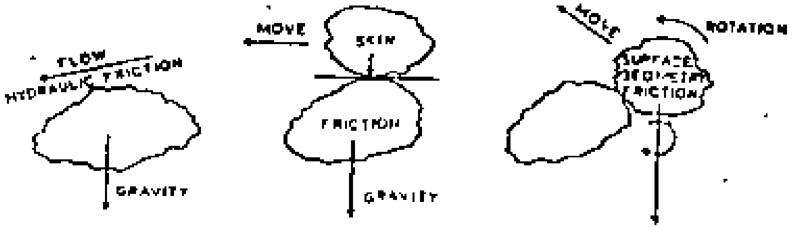

Fig. 3. Various types of friction occurring in a rock mound.

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

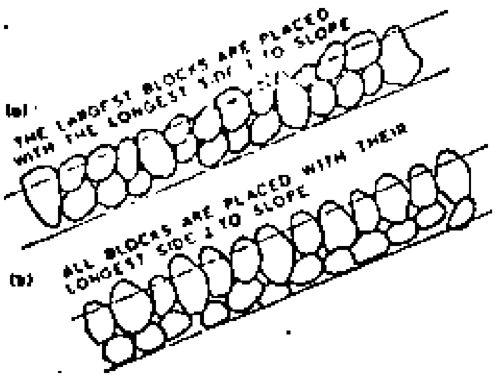

Fig. 4. Placement of blocks with the long side perpendicular to the slope to increase stability.

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.

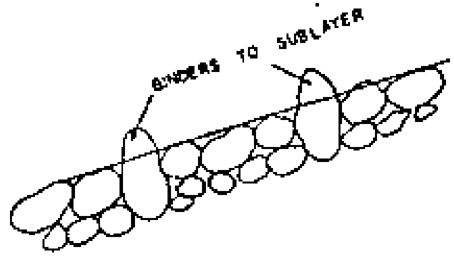

Fig.6: Binder-connection between armour and sublayer.

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

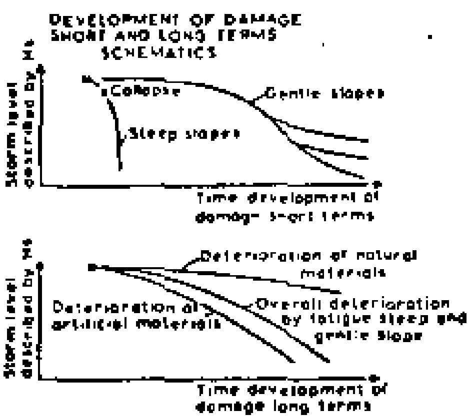

Fig. 6. Schematic description of breakdown of rubble mound in relation to duration of storms.

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.).

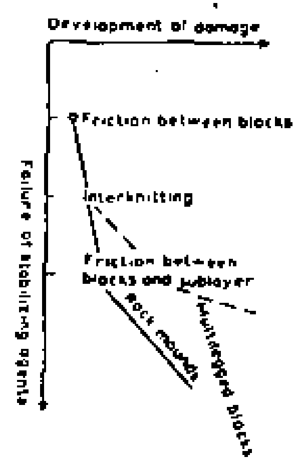

Fig. 7. Damage patterns for rock and multilegged concrete blocks.

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 4.61×10^{11} 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

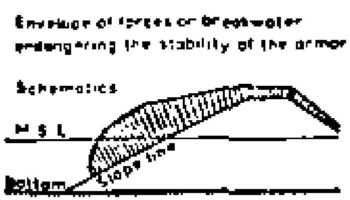

Fig. 8. Envelope of maximum forces on rubble mound breakwater.

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 Gunbak, 1976, 1977, 1978; Goda, 1976; Chakrabarti and Cooley, 1977; Houmb and Vik, 1977; Gunbak, 1978; Houmb et al., 1978). Report No. 7 mentions recording of damages by rational methods (Bruun and Gunbak, 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.

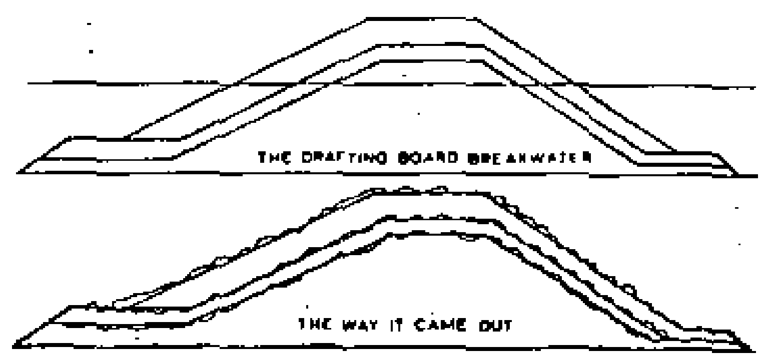

Fig.9. Theory and practice.

(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 Eliasson (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-

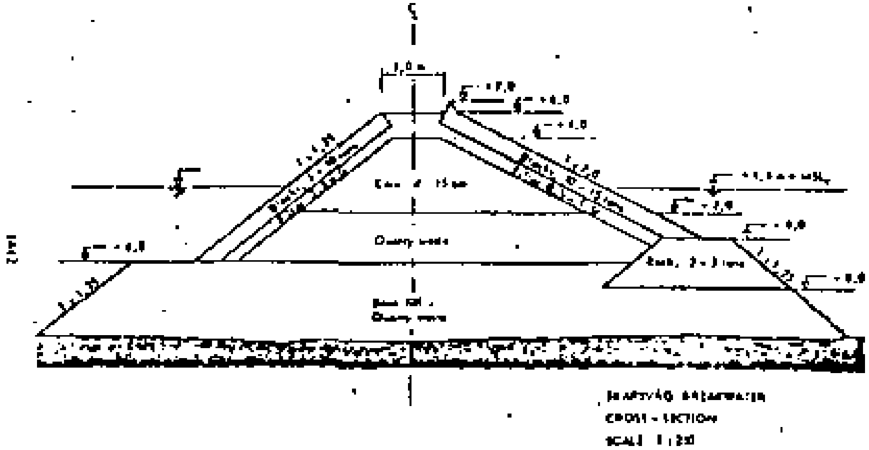

Fig. 10. Breakwater profile for a medium exposed construction on the Norwegian arctic coast.

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.

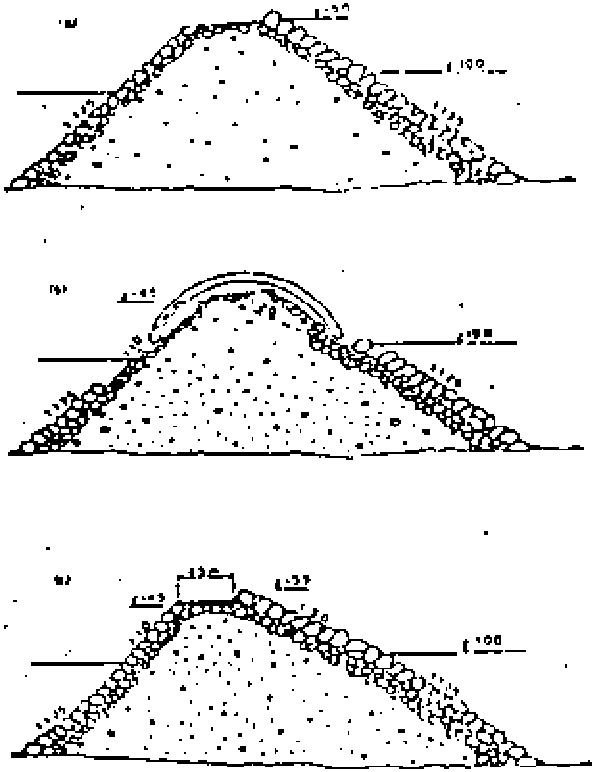

Fig.11. A. Breakwater after first season of construction. Both slopes with natural angle (1:1.25). The jetty has an extra height of 0.5 m (Gryllefjord, Troms).
B. After one or two years the blocks are removed from the slopes, and some of the core is moved from the outside to the inside to achieve a different slope of 1:2.0. All done in one operation with a medium sized (48 metric ton) backhoe barger.
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.

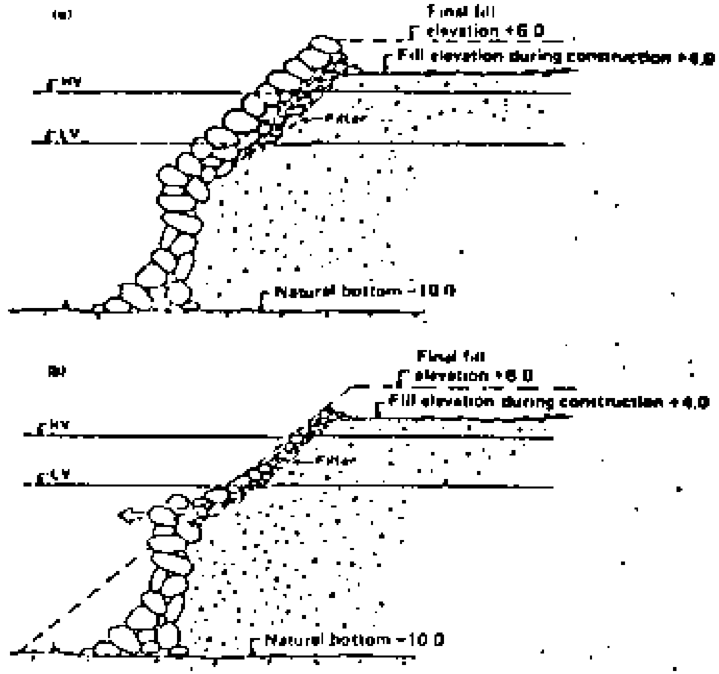

Fig.12. A. An unstable breakwater head, built up on a steep slope below water.
B. Down blasting of the underwater armour blocks with small explosives to obtain the prescribed slope.

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 "interlangling" 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

August-September, 1981
Per Bruun
Dr. techn. Sc., Denmark
Dr. Hon. Causa, Spain

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REASONS FOR DAMAGES TO THE ARZEW EL DJEDID BREAKWATER

DECEMBER 26-29, 1980

Abstract - The main topic of this report is an evaluatic 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 3, 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 procedur 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 hydro-dynamic 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):

$$H \sim H_b \sim 6{,}25\text{ m},\ T = 10\ (13)\text{ sec.}$$

$$\xi = \frac{3 \cdot 1{,}25 \cdot 10}{4 \cdot \sqrt{6{,}25}} = 3{,}75\ (4.9)$$

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 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. 26-29, 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/down-rush 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 exceeds 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 resonance. 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 absorption 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. Brun and Johannesson (ref. 1 and Appendix 2) analyze 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 pressure or "interknitting-intertangling" is most important.

From Table 1 it may be seen that the optimum stability for multilegged 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 γ_W = 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:

$$\Psi(\alpha = 4/3) = 0{,}05$$

$$\Psi(\alpha = 2) = 0{,}01 - 0{,}02$$

Scatter and Confidence Bands-Tests

have shown that there exists an important randomness in the structural response. The stability of the main armour layer varies with the

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:

$$\psi = Ir - Iro \qquad (6)$$

$$\xi = \ln (\psi / Ir - Iro) \qquad (7)$$

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

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

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

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

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 Tetrapodes (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. (5) 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	1,91
	0,66	0,61	0,52
1,50	1,99	2,27	2,93
	0,50	0,44	0,34
2,0	1,73	1,93	2,37
	0,58	0,52	0,42

Apparently slope α = 1,50 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 = 2,0.

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, the velocities in the plunging, collapsing or turning 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 [illegible] the deepest possible downrush, which is located

1,5 to 2,o wave heights (Hs) or about lo-12 meters maximum in the case of a lo 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 lo meter maximum wave - below minus lo 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 No, Figs. 2,8, one of the characteristic profiles. It has a platform between -lo and -12 m and a deposit slope below -12 m. Other profiles are more complex. Profile 7o B (p 23 of DAREP) has some smaller platforms created by deposits of Tetrapodes down-slope in stairs or ridges, supporting the material behind them, giving a sawtooth geometry, including "vertical" and "horizontal" faces. This in turn has also caused the formation of cavities by brid 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 squeez-ing force from the blocks above caused by incidental bridging. A recent example 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 formation 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 mentioned 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 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 -6 (-9) meters to -5 meters in the about 16 meters wide berm to the toe of the upper slope rounded up towards this slope and supplied with larger "corner" 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 No, 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 "resonith 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 2it 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 5 (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 exceeding 3 meters were recorded 11 times. In April 78 waves were 5.6 to 6.o 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 Sines, 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 witness 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·Hs on the slope.

Overwashes at Arzew el Djedid - As waves of 9-1o meters height may have, or probably have, been integrated in wave trains during the Dec. 22-29, 1980 storm, and such waves may cause a 9 meter uprush, (refs. 2, 9, 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 Arzew 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 (3.2 specific gravity) and sometimes filled with holes and less dense layers, e.g. of hardened vulcanic bombs or ash (2,o specific gravity). All materials must be selected carefully before construction and [illegible]

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 H 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 petite que 6t" (p 26, p 29, and p 29), "les roches paraissent 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 35), etc., including remarks or information, which refers to "workmanship" or "controls" rather than to health or soundness of materials. The combined DHI and C H (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 pendant les travaux" (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 ont 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 eu lieu pendant 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 ensevelie

(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, 2.10, 2.11, 2.12, 2.13, 2.14, 2.15, and 2.16 and numerous photos incl. 21, 22, 24, 25, and 25. 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 25 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 10 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 "Rational 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: / Data to be used:	Wave spectra. Extreme wave heights "Design wave"	Materials and equipment available for construction. Wave conditions of most dangerous character for stability of structure. Probability and risks of occurrence of such conditions.
Design Procedure:	Empirical 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.	Optimization of [illegible] 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.
Result:	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. Undexpected influence by structure on the environment.	Optimized design of maximum stability and economy, all aspect considered. Reliable time schedules. All influences on the environment predicted, considered and accepted with the necessary corrective steps.

RESEARCH AND ENGINEERING

Hydrodynamics:	Wave conditions criteria for stability and operatic:
Structural:	Overall stability Unit stability Maintenance Economy
Soil Mecanics:	Exterior as well as interior stability.
Environment:	Influence of structure and construction on the environment.
Construction:	Practical. Adherence to design, capacity and time. Controls before, during, and after constructions. Coordination. Reserve capacity available.

Future expansions must be considered and discussed.

It distinguises between "Earlier and Present Practices" and "New Practices". It calls to the attention, as mentioned earlier 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. 1o and 22) and "Jumps in Wave Heights (ref.1o) 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 buoy 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, 198o seems to have exceded by approximately 1 meter other storms on record since 1976. But it corresponded to the hindcasted storm of 6.5 m (Hs) with 18% 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. 1o). 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 55%. 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 B1 layer presented a smoother surface against the Tetrapodes than assumed. This circumstance has - at least - weakened the design. Photo 60 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 30, 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 30 "Au profil 154, 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 retained "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 SCHITRAM 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 10 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 1980."

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

The conclusion of the tests run on profile Fig. 4.9 (p. 70 of DAREP) reads: "Le dernier profil testé (avec deficit de materiaux) est moin stable que le profile théorique" - "Il n'est pas probable qu'un déficit en materiaux soit la seule et peut-être même pas la principale cause des dommages occassionnés à 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 17o 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 Akranes, 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, 17) to the lower slope and base of the mound (photos 5o, 51, 53, 57, 58) and numerous profiles, example: Fig. 28, p 25 says: "Malgré qu'il y ait une double (et quelquefois triple) couche de Tetrapodes sur toute la longueur du profil, il y a de nombreux signes de dégâts. Le pourcentage de Tetrapodes endommagés dans la couche supérieure va jusqu'à 1oo, et diminue toujours dans les couches inférieures (par example 0% sur le profil 7oB, 5% sur le profil 69B, 30% sur le profil 64). - Further: "Un grand nombre de ces Tetrapodes ont glissé le long du profil en laissant des cornes encastrées 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 eu 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 shocks - 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 witness 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 movements 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 deficiencies 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, 198o 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. 25-29 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 weak 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 crest 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:

l_1: blocks were already broken or damaged during placement. - This undoubtedly has happened. It always does.

l_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).

l_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.

l_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.

l_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 CU 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 exceeding 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 responsibility for the collapse of the breakwater. The flaws in rock material, 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 dégats et même d'une destruction totale est trop élevé, et il importe de prendre des mesures immédiates pour préservation de l'ouvrage, même de façon 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 -5 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 15% durant le premier hiver et de 25% durant les deux prochain hivers", (p 61).
With reference to the above mentioned regarding waves <u>of particular dangerous character</u>, those probabilities may have to be checked when tests <u>[illegible]</u> have been run with <u>such waves</u> or <u>wave sequences</u>, 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 ab. 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 mortering. 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 <u>further repair works at Sines asphalt has been recommended.</u>

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 10 years and extending into deep water (10 to 25 meters). This includes:

<u>Bilbao, Spain</u>, Accident 1977	parallellopedic blocks, also recommended for repairs (90 ts). Models in Norway for final stage.
<u>Sines, Portugal</u>, Accident 1977-1978	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.
<u>St. Cyprien, Spain</u>, Accident 1980	52 ts DOLOS Replacement by about 90 ts parallellopedic blocks recommended. Model tests planned.
<u>Tripoli, Tripoli</u>, Accident 1977, 1978, 1981	21 ts TETRAPODES This case is now being investigated by a group of experts. DHI and P. Bruun report delivered already in 1979.
<u>Akranes, Iceland</u>, Accident 1981	4-5 ts Rock. Replacement by 6-8 ts Rock of volcanic origin. Improved profile with wide berm-top

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 10 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 parrilellopedic 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 20 meters in between. Reference is made to Fig. 2 (Fig. 23 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. 6, and as now recommended by the US Army Corps of Engineering for navigation breakwaters in the US). See Appendix 7, Chapter 3, pp 195-225 for details. Sublayers must be of substantial rock and fulfil Terzaghi's filter ratios. This was probably not the case at Arzew el Djedid. The question of proper and reliable modelling is mandatory. Appendix 8 describes "Waves for Model Experiments" based on adequate wave data (Appendix 4 and refs. 7 and 22). Three-dimensional 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. 15) 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: 6; 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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Elisabeth Bruun
for
P.B.

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APPENDIX 5

DISCUSSION ON WAVES AND WAVES VS. STRUCTURES, [illegible], Trondheim, Norway

Introductorily it shall, once for ever, be stated that the "H_s -Xyr-occurrence" criteria is inadequate and obsolete. So is the sole spectral procedures with no consideration to details.

It is certain sequencies of waves including single, double or even triple very high and sometimes also very long waves, which swamp the structure, and very steep waves and combinations of such waves, which are most dangerous to stability. Long and high waves penetrate into the structure and lift the blocks, next cause a strong and deep downrush, which may draw blocks that turned loose with them down.

It was also realized that a single wave recording a certain distance - e.g. 1 kilometer - in front of a structure cannot describe wave action on the structure, as the wave action may be strongly influenced by even slight changes of the bottom topography, of various 3-dimensional effects in the wave condition and of reflective effects by the breakwater itself, cutting down wave steepness by breaking.

Grünbek and Bruun (Vol. 2, p. 13o1) described particular dangerous wave combinations based on various tests.

Dr. St. Denis, U.S.A. (Vol. 1, p. 49o) emphasized the irregularities in sea condition, which included "mountainous seas". A reliable model must be produced, "which takes into account all the non-linearities that are manifested in heavy seas."

H. E. Krogstad, et al., Norway (p.587) in paper on the analysis of wave spectra from the Norwegian Continental Shelf mention the results of instrumental wave-observations from four locations on the Norwegian Continental Shelf. These observations show large variations of the spectra for waves with different time histories. "It has not been possible to present simple parametric forms that characterize all situations."

Consequently spectral methods solely are not useful as they are unable to present the actual conditions in a reliable way. "The direct spectral long-term statistics may represent an alternative way". The problem should be pursued by the use of joint and marginal distributions of significant wave heights and mean periods and cumulative distributions of spectral densities. (p.576)

Houmb et al., Norway, (Vol.1, p. 579) mention the estimation of wind wave spectra by parametric methods, modelling the sea surface by a combined auto regressive moving (ARMA) process. "The estimation of such a model consists of determining a number of coefficients, 5-3o, that together with a white noise term define the model."

The great number of coefficients makes this model very laborious without giving any guarantee for correct representation of the wave geometry in a sequence of waves. It is the wave geometry, which determines the forces, as it determines the hydrodynamic forces.

A. F. Whitlock, U. K. in discussion of paper by Grünbek and Bruun on "Wave Mechanic Principles of the Design of Rubble-Mound Breakwaters" (Vol. 2, p. 13o1) on p. 53o of Vol. 3, says as follows: "An occasional large wave in an irregular wave train can produce a jolt in a random block arrangement, which can then settle back more or less into place." - "If, however, in a long-duration storm from a long fetch, a train of large waves can be formed, then this is equivalent to a significant height of a much larger spectrum not considered for the design." (p. 53o). On p. 531 he mentions, based on observations, how "a wave of lifting moves upslope just in front of the wave front". This is observed earlier by Bruun and Johannesson ("Hydraulic Parameters for Design", 1975) and is due to the static head of the breaker being transmitted to the underside of the blocks through any porosity, which exists. The effect is augmented by the arrest of any down flow and may make the blocks almost bouyant, so that they are relatively easy to turn loose (p. 531).

The long period waves are very dangerous because: "following a greater penetration of flow from long period waves, an intern locked layer can be eased from its foundation to throw the entire weight onto its lower members and their footings". (p.533).

Bruun and Grünbek in their reply to Whitlock, Vol. 3, p. 534, emphasizes very strongly that wave geometry and sequence must be correctly represented in the model. "The results of model tests without proper check and imitation of details of wave geometry are questionable."

The general impression of the discussion is very similar to the consensus of the results of the Santander-Conference, 198o, in Spain. Also here the spectral reproduction, without caring about details of sequences and geometry and the irregularity of the wave action, invalidating the results of recordings by a single wave recorder, placed a distance from the subject structure, was miscredited. Model tests must be based on possible adverse combinations, whether they have ever been recorded or not. Sequences of waves recorded from a single recorder at "point A" give no proof of a similar sequence at "point B", and tests must be based on actual forces right in front of the structure occurring at any point. Only in that way it is possible, e.g. enveloping, to obtain the necessary safety of design and avoid mishaps due to inadequate wave information and interpretation of wave data to include wave action right in front of the structure. Or: - we should avoid that the Hs = 2o-year-occurrence occurs 5 times a year and that the Hs = 1oo-year-occurrence happens once a year. It is very unfortunate that, due to the often lack of practical experience by laboratory people, the test procedures have been concentrated too much on "academic similarities", instead of practical observations and imitations of the true wave condition right in front of the structure or right at its location. The variances, which occur along the structure, can only be determined by observation.

Refraction diagrams are of less or little use in confused seas with considerable effect of shortcrestedness, where diffraction phenomena are very important. Representation of a confused sea cannot either be done in a wave tank, as the confused sea is a result of variances in the direction of wave approach. It is known that 3-dimensional effects are very important for the stability of multilegged blocks. Such 3-dimensional effects are common in all storms with centers moving often in erratic paths. Variances in wave reflection furthermore increase the confusion and 3-dimensional effects. Generally speaking, it must therefore be concluded that wave tank tests in two dimensions cannot be expected to give correct answers on the true stability of a breakwater. They must be supplemented by 3-dimensional tests.
To secure field data, useful for the calibration of such tests, is not an easy task. Airial photography is difficult or impossible to undertake under storm conditions. Observations by radar or - if possible - stereo airial photos seem to be one of the measures, which would contribute to reliable records of wave action during a storm condition. Based on such observations the reproduction of "synthetic waves" in a tank may be possible, so that prototype and model may approach each other closely.

In the discussion at POAC - 79 as well as in Santander, 1980, it was all agreed upon, and strongly emphasized, that model experiments must be continued beyond what is justified by field data (almost always modest) and that, under all circumstances, breakdown pattern and modes must be studied - to see, how the breakwater behaves during a breakdown as well as to "feel the breakwater on the teeth" with respect to what it is able to stand up to !

June 81

P. Bruun

WAVES FOR MODEL EXPERIMENTS

Earlier (present) practice calls for representation in the model of a wave action, which was recorded at a certain point (A). Although the representation, spectral-wise, may be fully correct in the model two questions arise:

(1) has the recording been undertaken long enough to allow a reliable estimation of "all future storms". For a long-term and extreme height statistics a recording time of minimum 3 years is usually required for temperent regions. But it is still no guarantee that an extrapolation based on statistical principles will give correct answers regarding the extremes. This is true, in particular, when detailed wave geometry is considered. Correct representation is very important, however

(2) is the point of recording representative for a larger sea territory e.g. for 2-3 kilometers along the line of a proposed breakwater ? The answer to this is: Most likely not! Even small changes in the refraction pattern will cause changes in wave pattern and geometries of the waves including shortcrestedness.

The question, which arises, is then: How is it possible at all to obtain a reliable wave picture at the site of a proposed breakwater useful for models ?

The answer to this question seems to be:

(A) First of all details of the geometries of the waves recorded should be examined carefully with special reference to waves, which are close to breaking or are breaking. This involves shortcrestedness.

(B) Refraction diagrams for irregular waves should be drawn for various occurring directions of wave approach. Time history of storms may cause crossing orthogonals.

(C) 3-dimensional effects should be investigated by studying the time history of the storm. Mountain waves may be a result of waves of different fetches from different directions.

(D) If possible, observations by radar during storms should be made. This, however, require a high (location) tower or mast.

(E) Adverse wave combinations should be imitated and dangerous conditions be extrapolated to even more adverse conditions, even if they have not been recorded. Such adverse conditions, which meteorologically speaking may be related to gustiness, may be developed from recorded data by means of crossing orthogonals, wave trains of slightly higher periods overtaking waves of slightly lower periods or simply by the generation of single waves of solitary "mounteneous" nature, which are possible, although possibly not recorded from a single wave station. If such waves should actually occur, the structure, if designed accordingly, will be able to resist them and mishaps caused by insufficient recording time and insufficient information about 3-dimensionality and non-linearities (Appendix 3) can better be avoided. Extreme wave height statistics is helpful in developing waves for such tests. It is normal to use Gumbel distributions for extreme statistics, but other distributions exist.

(F) Model experiments in 3-dimensions will, of course, be advantageous and will probably develop to become a standard requirement for all final designs. Model experiments must include breakdown pattern and modes.

(G) The hydraulic properties of the model - breakwater should, of course, correspond to the prototype, that means reflections, up and downrushes, and static pressures from water in the mound should be similar. This is a discussion on model laws and similiarities, in which field progress is being made. It is certain that it is very important that wave breaking criteria are imitated correctly, using the $\xi = \tan\alpha / \sqrt{H/L_0}$ factor. Model porosities shall not be larger, hydraulically speaking, than those of the prototype - or, conversely they shall be "large enough" to correspond to the prototype.

(H) The prototype is designed, next built, based on the results of the model study. The lack of proper similiarities caused by inadequate planning and design, or lack of adequate supervision during construction, has a responsibility for many failures of the prototype. Such mishaps can only be avoided by a change in attitude towards and better understanding of what a model is or is supposed to be. A model first of all is not "a piece of toy", but a "true-to-be representation" of what is going to be built to withstand actual forces. The person, who is in charge of the model experiments from the side of the sponsor, should not be "discharged" from the project upon completion of the experiments, but instead he should rather be put in charge of or be associated with the supervision of actual construction. It has happened, quite often, that larger discrepancies between model and prototype were allowed to pass or did pass, because nobody objected or protested against them. The result could be, or it actually became, failures sometimes of mass character. The same is true for materials used for construction. Very often they did not live up to the necessary qualities and standards.

June 1981

P. Bruun

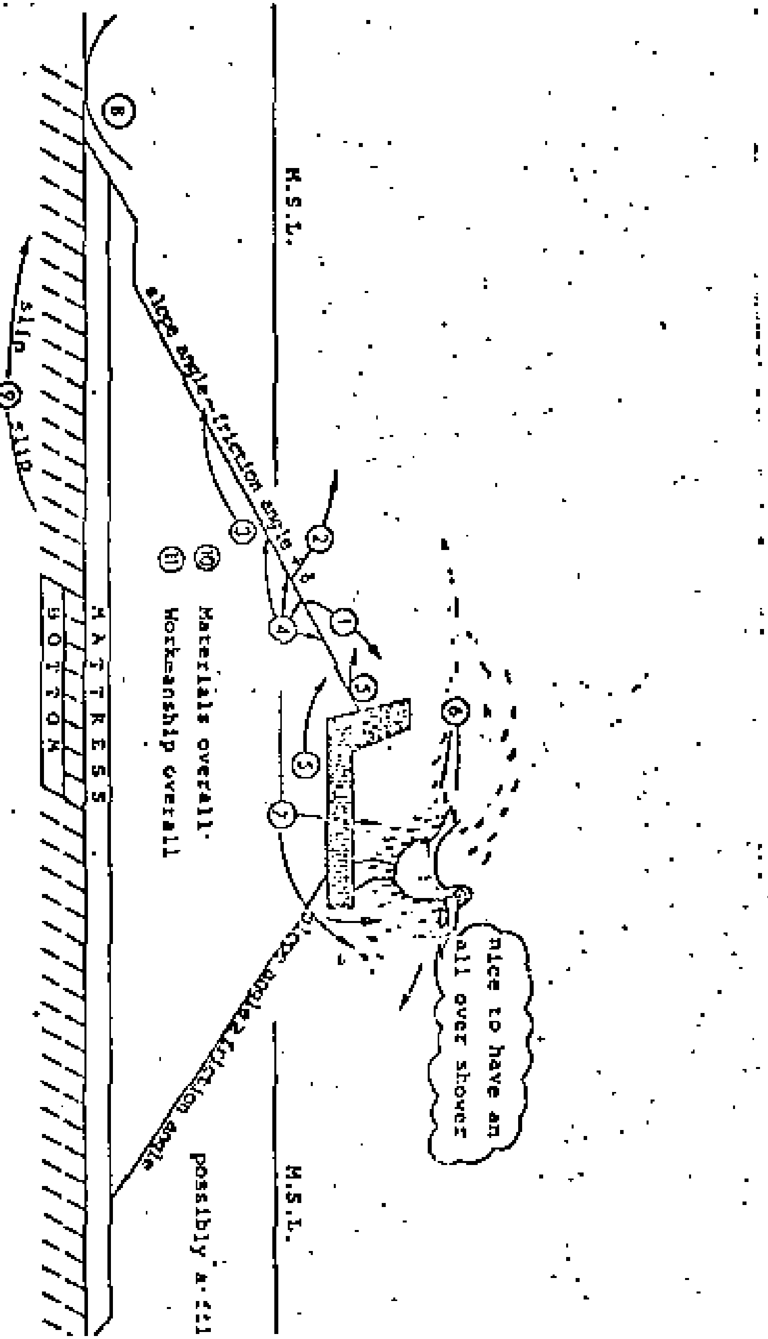
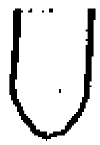


Fig. 1. Common Reasons for Damage to Mound Breakwaters

(ref. 3, Appendix 1).

Fig. 2. Ideal Profile for a Mound Breakwater

(refs. 1 and 8). See Appendix 2 and 7.



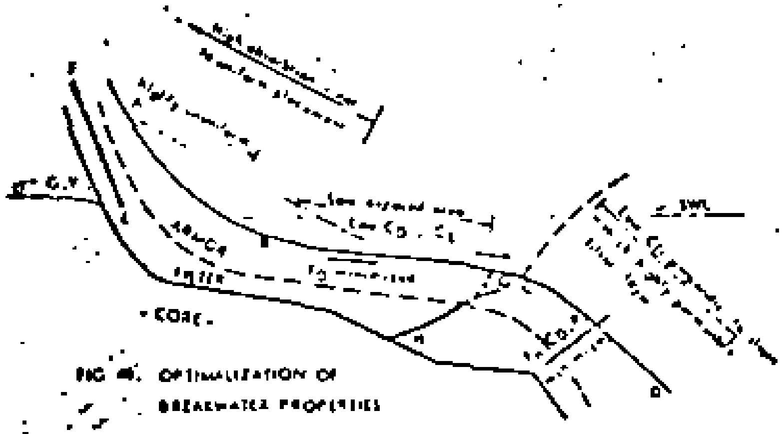


FIG. [illegible]. OPTIMALIZATION OF BREAKWATER PROPERTIES

[illegible]

[illegible]

[illegible]

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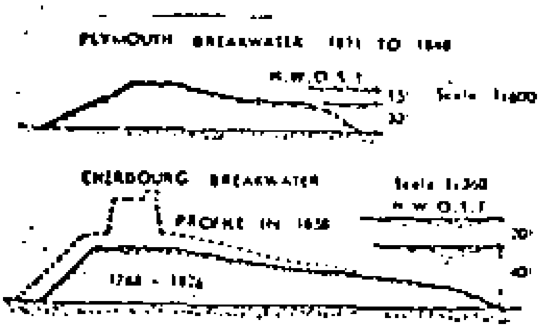


FIG. [illegible]. ANCIENT BREAKWATERS AT PLYMOUTH AND AT CHERBOURG.

Fig. 3. The Plymouth Breakwater in England

(refs. 1 and 8). See Appendix 2 and 7.

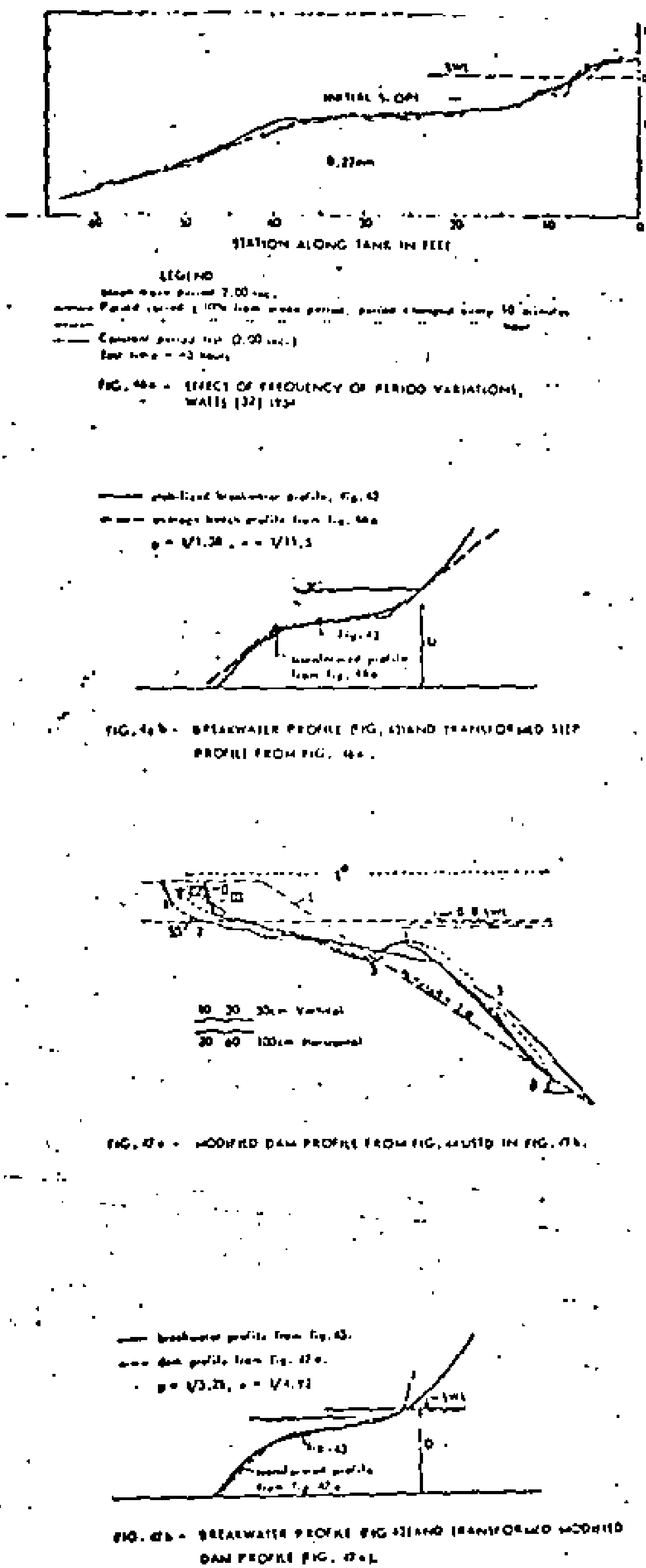


Fig. 5. Beach berm, step profile and rubble-mound profile

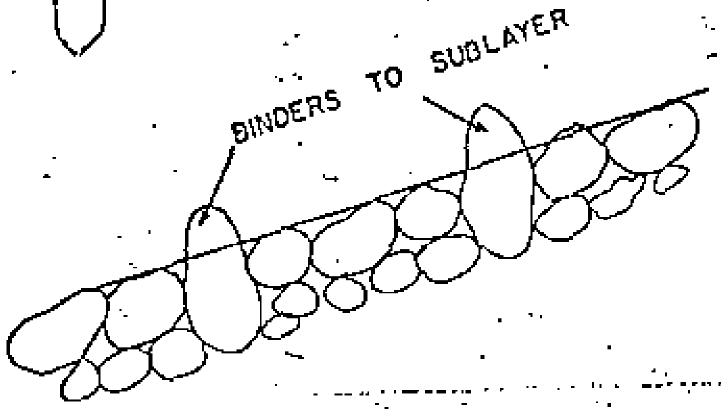
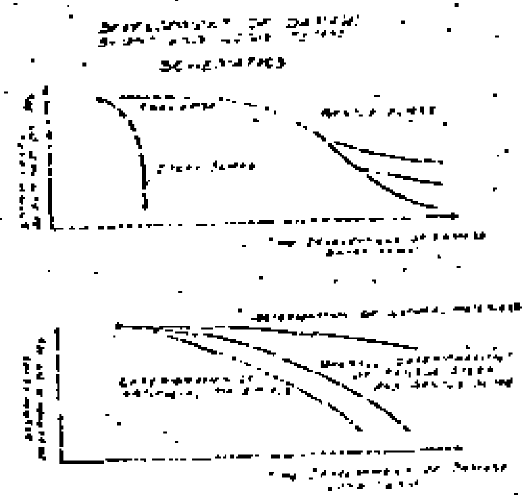


Fig. 6. How to roughen the first sublayer to provide better friction connection between armour and first sublayer (refs. 4 and 8). See also Appendix 7, Chapter 3.



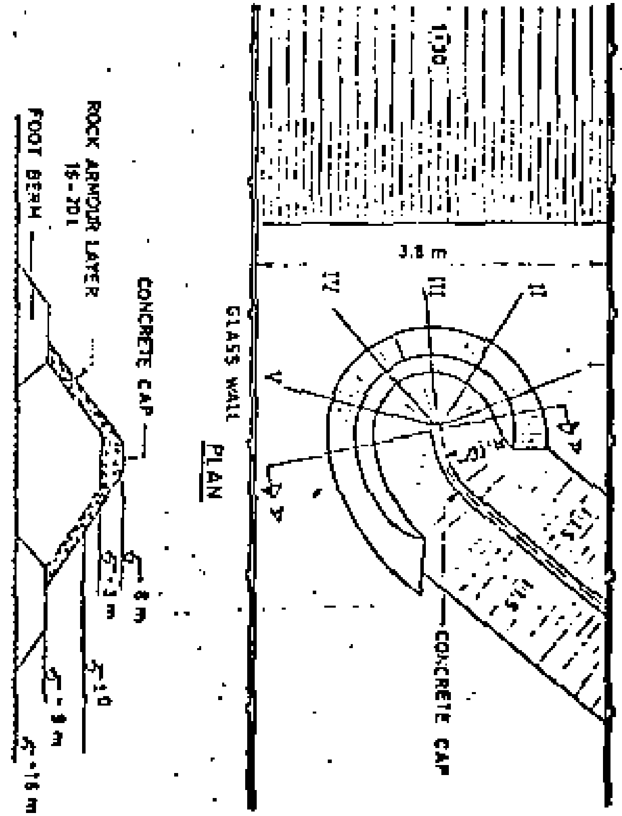


Fig. 7. Stabilization of Breakwater Head at Sorvaer Harbour, Northern Norway (ref. 25).

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 Sines (Portugal), Bilbao (España), Arzew el Djedid (Algiria), Tripoli (Tripoli), and St. Cyprian (España). 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 matires

This paper analyses the components in a stability condition, reasons for failures and formulae - 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 take 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 $\xi = \tan\alpha/\sqrt{H/L_o} < 2.5$ 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 areas, 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 thecore 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 does 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 10 t/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 ... shallow water or where the depth/wave height ratio is less than 2.0. 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 ($a/b, b/c \leq 1.4$ 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, insufficient 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, 14).

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 multi- legged 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 "Antifér-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 (Permanent 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 formulae 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 therefore 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 formulae 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, 20, 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 formulae 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 in the field.

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 geometrics 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. 30). The very important joint-distribution of heights and periods have been explored extensively (refs. 6, 12, 14, 19, 20).

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, 30) 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

turned out. No-one was concerned about wave-hydrodynamics, because nobody actually knew what it was - therefore the numerous formulas, which became some kind of religous believes. The fact is that mathematics is an excellent stick to which the man can support himself. But the difference between the man and the stick is that while the former has (is supposed to have) a brain the latter does not outside its own limited scope. Consequently, the man should always run the stick, not vice versa. But this is often not the way things were handled. The "Design Wave", today an imaginary and absurd figure, has often been the prime mover in design - because it was so easy to plug it into a "formula". Thinking and understanding of the process involved was not necessary. What we need for an intelligent, responsible and safe design in 1982, however, is design conditions related to inertia, drag, lift, suction etc. in all hydrodynamic and hydraulic forces + facts. The PIANC in its above cited warning (ref. 1), therefore, took a well-reasoned stand in its recommendations on future designs: "Avoid formulas". Replace fragility by stability. Another question is: Do we need these massive breakwaters at all ? (ref. 32).

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LIST OF FIGURES

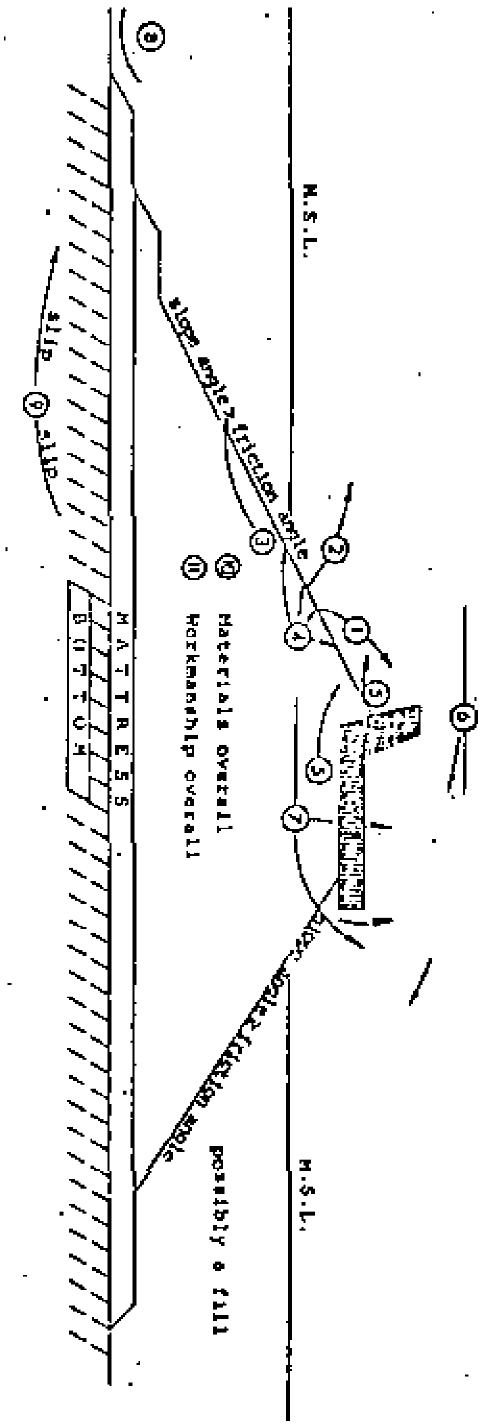


Fig. 1 Various Reasons for Damages to Rubble Mound Breakwaters.
Location of Failures

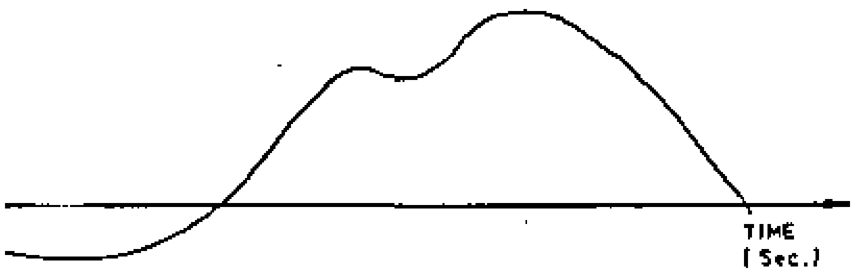


FIG. 2a Mammuth (freak) wave.

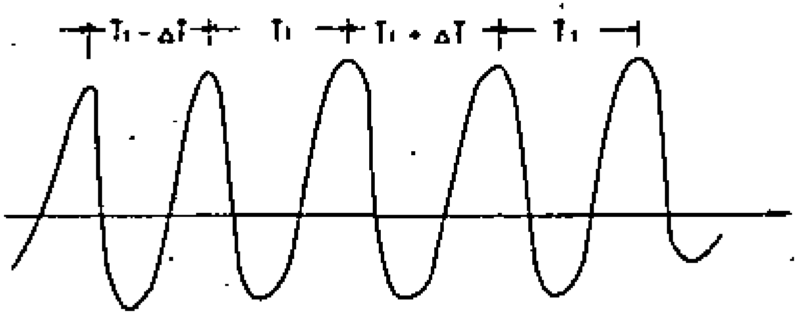


FIG. 2b Group of waves.

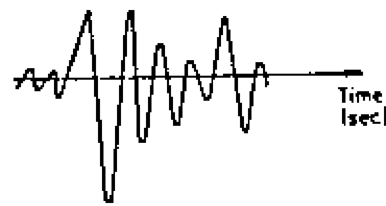


Fig. 2c - Wave series with deep trough

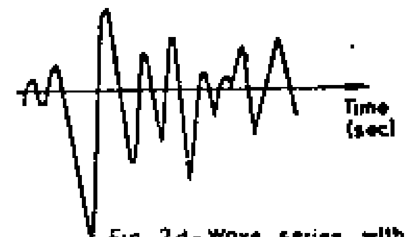


Fig. 2d - Wave series with deep trough

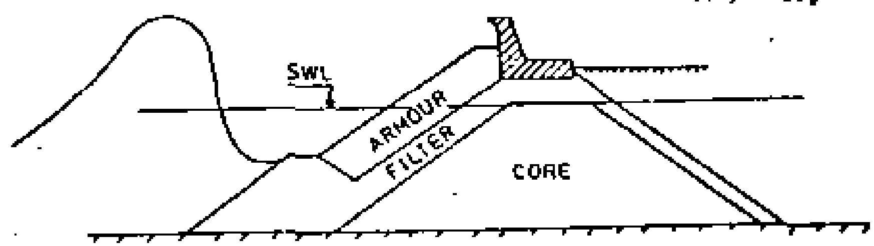


Fig. 2e - Sketch of deep run-down occuring with time series shown

Fig. 3. Damage patterns for rock and multilegged concrete blocks

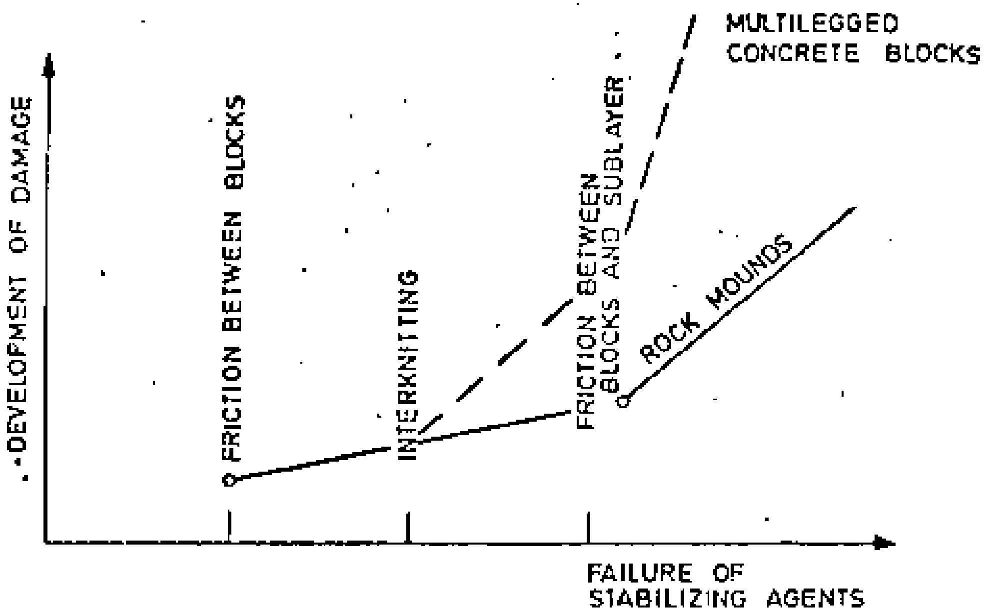
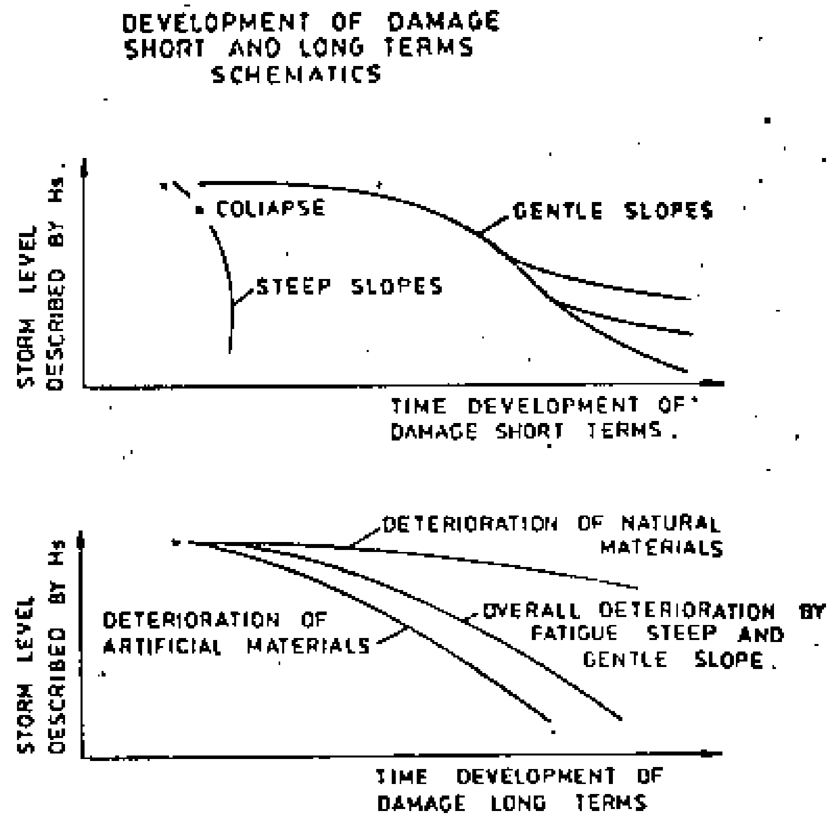
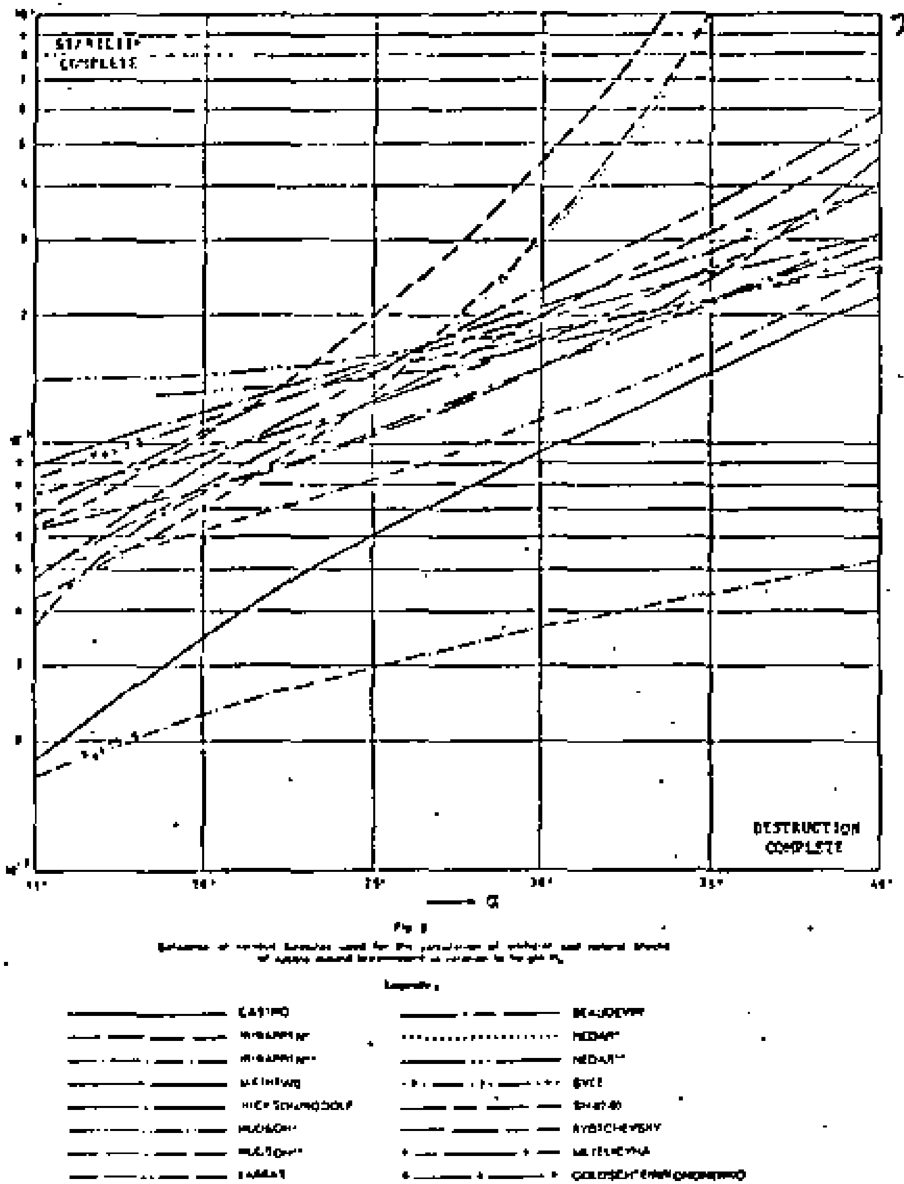


Fig. 4. Schematic description of breakdown of rubble mound in relation to duration of storms.





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 on 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 chiesel, etc., then go ahead, you may of course also hire a nurse or medical technician to assist you. Results were faulty (Algerie and Brasil).

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 produced "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 OCEANICA

BASIC STRUCTURAL SYSTEMS
A REVIEW OF THEIR DESIGN AND
ANALYSIS REQUIREMENTS
BRIAN J. WATT

PROF. ROBERTO MELI

MARZO, 1982.

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Chapter 1

Basic Structural Systems— A Review of Their Design and Analysis Requirements

Brian J. Watt

1.1 INTRODUCTION

A casual observer at one of the many engineering symposia on numerical methods in the past decade would probably have been left with the impression that all of the interesting boundary and initial value problems had been solved, or at least that this was very close to happening. The power and versatility of numerical analysis methodology are such that he or she could readily be forgiven for believing this. While it is probably true for finite element methods that most of the major advances of a formulative kind have been made, much remains to be done before the developer of such systems can relegate all his programs to 'ordinary user' status. This is certainly the case for their application to offshore engineering.

The purpose of this introductory chapter is thus to identify some of the basic design problem areas and to help set the scene for the ensuing more detailed discussion on specific analysis topics.

There are many different activities and requirements offshore and hence many different structures used. The ensuing discussion will be limited to so-called 'fixed' structures where this is interpreted to mean structures installed and operated at one location throughout their design lives. This presentation is not intended as a comprehensive audit of all analysis problems in offshore engineering. It is hoped that it will at least provide a framework in which to view the more refined analytical discussions to follow in later chapters.

1.2 THE OCEAN ENVIRONMENT

1.2.1 Physical oceanography

It is appropriate to allocate a few words to the environment in which the offshore engineer works. The ocean can vary very widely at different places and

times. In attempting to understand its behaviour, it is useful to refer to the analogy drawn by Vine[1] between physical oceanography and meteorology. Both embrace complex short term variations superimposed on predominantly seasonal cycles that are in turn affected by long-term climatic changes. The physical and chemical properties of seawater itself are also quite variable and thus whether one is interested on a macro or micro scale, the single dominant characteristic of the ocean environment is its changeability.

1.2.2 The meaning of 'offshore'

In its most general application, the term 'offshore' is usually taken to mean that part of the ocean where the present mudline is below the level of the lowest astronomical tide (L.A.T.) It is frequently also assumed to be restricted to the area of the continental shelves or a water depth of less than 200 m. This latter definition embraces an area comprising 8 per cent of the total wetted surface of the earth which is in turn equivalent to nearly 20 per cent of the total dry land surface.

1.2.3 Wind, waves and currents

The motions of the sea are the result of a superposition of many disturbing forces. Mathematical tools have been created to define these motions, but despite all efforts, any such model requires tremendous approximations which seldom represent the true facts with all their complexities.[2] The problem of wind offshore can be severe on parts of a structure and is especially important with regard to floating stability. Nevertheless, for most fixed structures in deeper waters the wind load represents less than 5 per cent of the total environmental loading. Current loads can be severe in some locations but it is usually wave loads which dominate the designer's thinking. Factors such as earthquakes and floating ice can change this completely but it is appropriate in this book to concentrate on waves, since this form of environmental loading dominates for most offshore installations currently being planned.

The exact mechanism by which wind creates waves is not yet understood. Nevertheless, mathematical bases exist for predicting the wave profile and associated water particle kinematics for wind induced waves. The relative validity of the various theories was reviewed by Dean,[3] however, it does not need much observation of the sea to suggest that its surface is not characterized by regular, long-crested waves, whether Cnoidal, Airy, Stokes Fifth or otherwise. Its generally rather chaotic behaviour is better defined by the term 'sea state' and it is becoming increasingly recognized that realistic handling of the engineering problems requires some form of stochastic treatment.

1.2.4 Seabed geology

The geotechnical properties of a soil deposit are a complicated function of the origin, deposition, weathering and loading histories. While it is obvious that the morphological processes in the marine environment are different from those on land, it must be remembered that large parts of our present continental shelves have in recent geological time been raised above sea level, loaded by ice and sediment and subjected to subaerial weathering processes. Features such as buried river valleys, peat deposits, outwash channels and moraines have to be identified and avoided or quantified, along with other peculiarly marine features such as sandwaves and wave induced landslides.[4] Probably the most significant factor is the large percentage of the continental shelf area covered by recent deposits of marine clays and silts. These typically are under or normally consolidated with mudline shear strengths of about 4 kN/m^2, occurring in deposits up to many hundreds of feet thick. In summary, complex soil deposits covering if anything a greater range of strength and deformability than on land form the province of the offshore foundation engineer. Any numerical models of geotechnical problems are therefore likely to be at least as complex as those on land.

1.2.5 Earthquakes

The foci of severe earthquakes are located in the tectonic discontinuities between the plates which comprise the earth's mantle. Most of these discontinuities occur offshore and some are associated with major hydrocarbon deposits. Examples are the continental shelves off California and the Gulf of Alaska. The offshore engineer is faced with the usual land based seismic problems of predicting structural and foundation response as well as some unwelcome additional phenomena such as tsunamis and submarine landslides.

1.2.6 Floating ice

Exploration for mineral deposits has been actively pursued in polar regions for more than a decade. Although relatively little has been developed in a true offshore polar region, the first steps have been taken and planning is already underway for offshore structures to cope with the floating ice problem. A recent review of the information available on the genesis, morphology, physical properties and failure mechanisms of floating sea ice[5] indicated that the art of predicting ice loads on fixed structures is in its infancy. A great deal of research is needed to give the offshore engineer reliable numerate answers.

1.2.7 The ocean environment and design constraints

The above picture is brief and superficial but it serves to introduce some key characteristics of the design problem offshore. These are:

(a) environmental loads originate from several sources

(b) they are generally larger, more dynamic in character and more highly variable than on land

(c) bottom conditions cover a wide range of soil types from very soft underconsolidated soils to rock

When one adds to these a fourth factor, namely the difficulty of underwater working, it is clear that the reliability aspect of design assumes a new perspective in the offshore environment.

1.3 THE CHOICE OF STRUCTURAL FORM

1.3.1 Historical perspective

The earliest engineered offshore structures were usually installed for either navigational or military purposes. The major share of offshore engineering experience, however, is associated with the exploitation of hydrocarbon deposits. The ensuing discussion is inevitably dominated by the requirements of typical oil and gas installations, but the engineering implications are relevant to several other offshore applications. It is entirely appropriate to view the problems through an oil industry perspective since this industry will continue to dominate the offshore scene for some time.

1.3.2 Foundation constraints

The structural options which are open to the offshore engineer are conditioned in large measure by foundation considerations. Figure 1.1 shows a highly simplified concept of an offshore structure. Fluid flow phenomena induce a

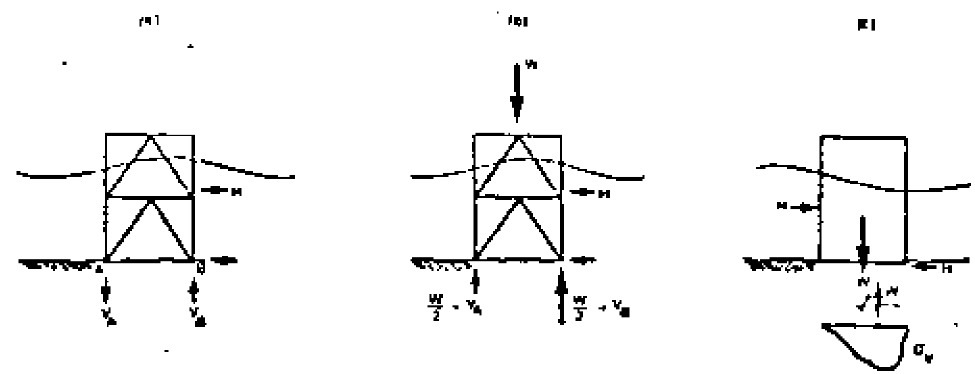


Figure 1.1 Piled and gravity foundations

high horizontal load and moment at the foundation level (mudline). For the two point support system shown in 1.1(a), a push-pull system of vertical reactions is required for moment equilibrium. Providing resistance to foundation uplift can sometimes be difficult and the requirement can be eliminated by providing additional vertical force as in 1.1(b). This is the concept of the so-called gravity structure, namely, the elimination of uplift problems by the provision of structural self weight. This is most often accomplished by using massive monolithic structures with continuous foundations as in 1.1(c). Framed gravity structures with discrete foundations as in 1.1(b) have been installed, though they are less common than the monolithic type.

It is evident from Figure 1.1 that:

System (a) implies a deep foundation (piles) to resist uplift

System (b) could use a deep or shallow foundation but downthrusts are larger than in (a)

System (c) implies much larger horizontal loads and requires strong soils close to the mudline.

1.3.3 Common structural systems

1.3.3.1 The jacket or template

It is proper to commence the discussion of structural systems with the so-called 'jacket' or template type of platform since it represents by far the largest number of offshore platforms installed to date. It developed to meet the needs of offshore drilling and production operations in the Gulf of Mexico and Lake Maracaibo. Both areas are characterized by large thicknesses of soft, recent marine sediments and foundation support has to be mobilized at some depth below the mudline. In order to reduce fluid loads and foundation movements a relatively transparent but still structure such as a space frame attached to long piles is an obvious solution.

A key difference between an offshore and onshore structure is the increased difficulty of construction offshore. A 'template' or 'jacket' structure is simply a space frame designed to make pile driving easier by obviating the need to provide temporary support for the piles during first driving. The principle is illustrated schematically in Figure 1.2. The jacket or template is placed in position, the piles are fed through the legs of the template and driven by means of a pile driver supported on a surface vessel. After driving to the design penetration depth, the piles are cut off at the head of the template and a prefabricated deck section is stabbed into the piles and field-weld connected. The deck weight is directly supported by the piles themselves. A typical eight pile jacket is shown in Figure 1.3. The method of construction for small jackets is quite simple. The structure is prefabricated at some waterside facility, skidded onto a flat-topped barge and towed to location where it is lifted into

position by a derrick barge. The only severe construction loads to be considered are those associated with the lifting operation.

As exploration and production proceeded further offshore and the location water depths increased, two things began to happen. First, the installation problems became more severe as the capacity of the seagoing derrick barges was exceeded. It thus became necessary to launch the larger jacket from its barge and to utilize ballasting and buoyancy procedures to bring about uprighting and sinking of the jacket, sometimes assisted by derrick barges. The second factor was the increase in foundation loads. It was no longer possible to make do solely with piling through the legs. Skirt piles driven in other plan positions and pile clusters around main legs began to be introduced.

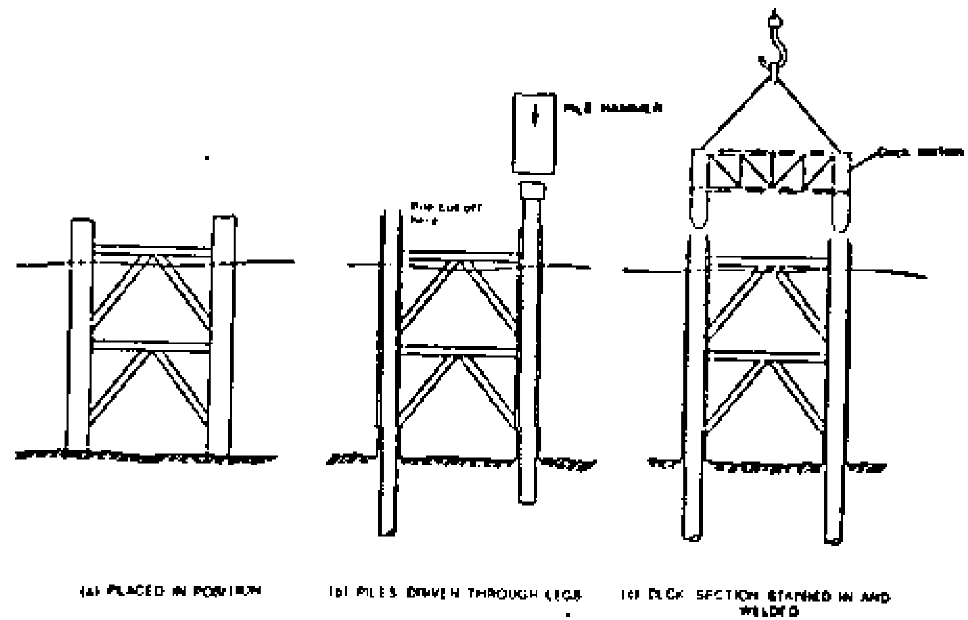


Figure 1.2 Principle of jacket or template

1.3.3.2 The tower

The tower is an extrapolation of the jacket to deep water. It is not only characterized by the use of pile clusters and skirt piles but the deck is now designed to be supported by the tower frame itself. Due to their size, towers are usually made self-buoyant either by enlarging several of the legs or by the use of a purpose-made buoyancy pontoon. Examples of the former type, which are sometimes called 'hybrids', have been installed on the Brent and Thistle fields

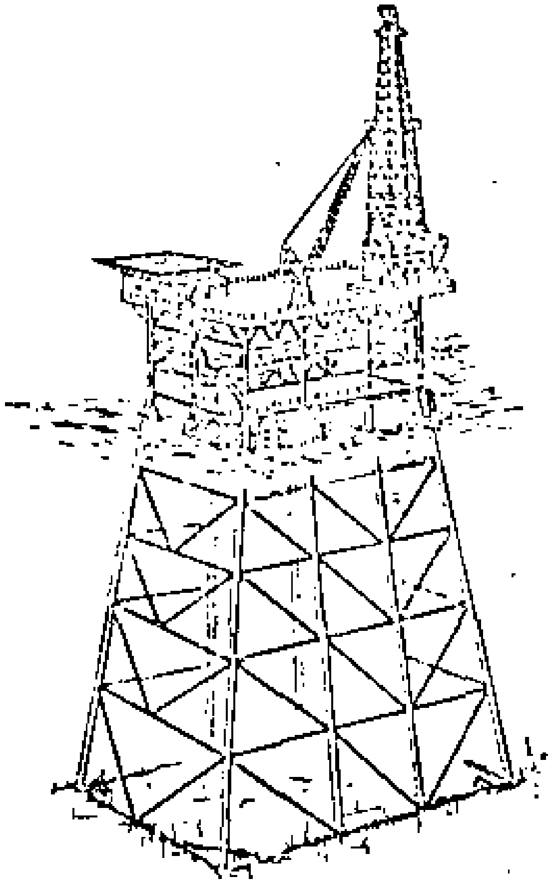


Figure 1.3 Eight pile jacket or template. (Reproduced by permission of Am. Soc. Civ. Eng.)

in the North Sea as shown in Figure 1.4. The Forties field towers used the supplemental buoyancy approach which has the advantage of reduced wave loads on the final structure but involves a more difficult installation operation.

The completed North Sea towers have been installed in depths up to 160 m and contain up to 40,000 tonnes of steel apiece, including deck and pilings. These do not however represent the largest framed offshore structures. The Exxon Corporation has recently installed a 260 m platform in southern California and Shell Oil Company are presently constructing a giant tower to be installed in more than 300 m of water in their Cognac field offshore Louisiana.

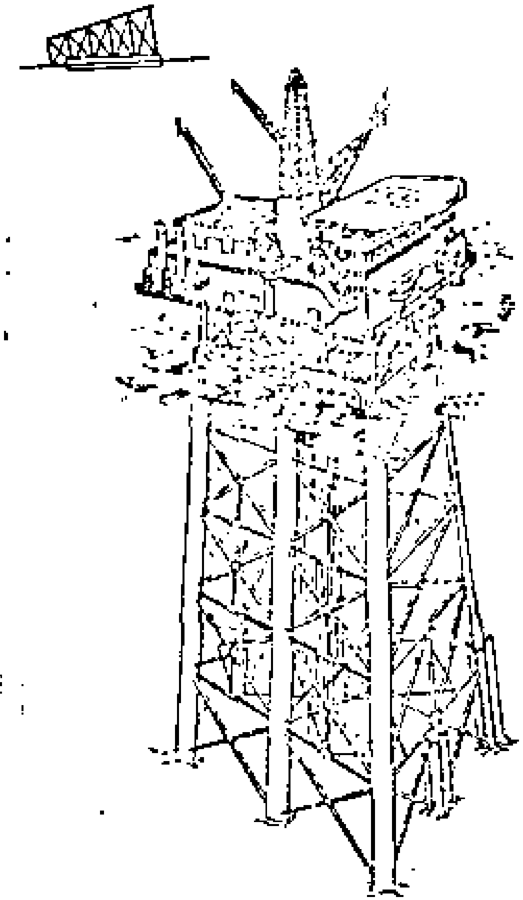


Figure 1.4 Hybrid tower with inbuilt buoyancy. (Reproduced by permission of Inst. of Civil Eng.)

1.3.3.3 Caissons

Not all offshore installations are on a heroic scale. Many oil and gas fields are quite small and have minor deck load requirements. In addition, there are always step-out wells, flares and other installations which require a small but reasonably stable platform above sea level. A traditional and relatively inexpensive concept is the so-called caisson. This consists of a single tapered pile driven to a sufficient depth below the mudline to enable cantilever action to develop. This type of structure has been used in water depths up to 60 m.[7]

1.3.3.4 Concrete gravity platforms

The pleistocene glaciation of the North Sea region created areas with heavily overconsolidated soils at or very close to the present mudline. These can support large loads at the soil surface and are hence well suited to gravity type foundations. A number of factors combined to create a suitable market for concrete gravity structures and several have already been installed. There are some differences between the designs built to date but the most common type comprises a large cellular base supporting three or four concrete towers which in turn support a steel deck. This type is shown in Figure 1.5(a) while 1.5(b) shows another type which is of more monolithic construction and hence much more rigid.

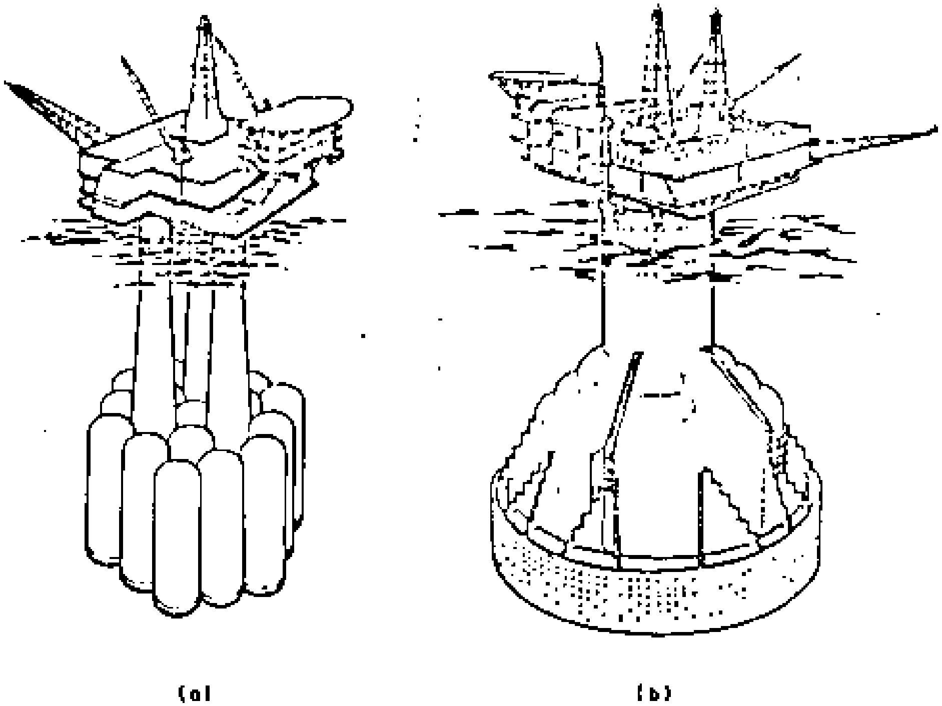


Figure 1.5 Concrete gravity platforms. (Reproduced by permission of Inst. Civil Eng.)

1.3.3.5 Steel gravity platforms and hybrids

The gravity structure limelight has undoubtedly been stolen by the concrete platform. Steel gravity platforms have been installed on the Loanga field offshore Nigeria where the presence of rock close to the mudline ruled out the possibility of tension piles (see Figure 1.6). Other steel gravity concepts are being actively marketed as are so-called concrete-steel 'hybrid platforms' which consist of a steel space frame supported on a large concrete gravity base. To the author's knowledge, no platforms of the latter type have as yet been ordered.

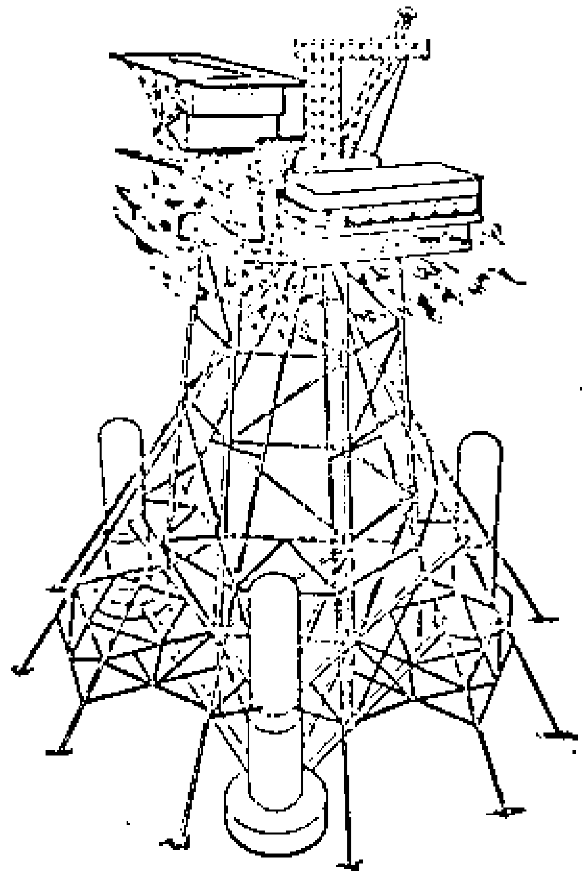


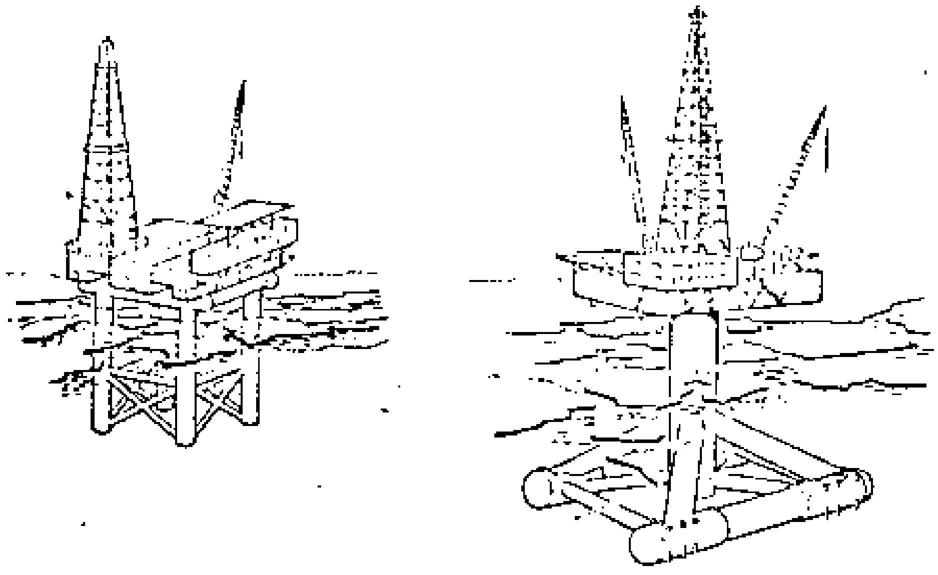
Figure 1.6 Steel gravity platform. (Reproduced by permission of Inst. Civil. Eng.)

1.3.3.6 Other fixed structures

There are many other fixed structures used offshore such as ballasted down concrete barges in the swamps of Louisiana, the piled oil storage Khazzans off Dubai[8] and the Ekofisk storage tank.[9] One could perhaps include the man-made islands in southern California and the Canadian Arctic. One of the most interesting classes of structure is that employed in Cook Inlet, Alaska; an area with large tides, high current velocities and floating ice. Figure 1.7 shows solutions which were adopted to meet the high horizontal ice loads in the Inlet.

1.3.3.7 Compliant structures

All structures deform under load. A compliant one, as its name implies, is designed to move so that the effects of the environmental load are mitigated. The analogy is a ship riding out a storm at anchor. As the scope of the anchor chain is reduced, the stiffness of the anchoring system increases. The ship



(a) (b)

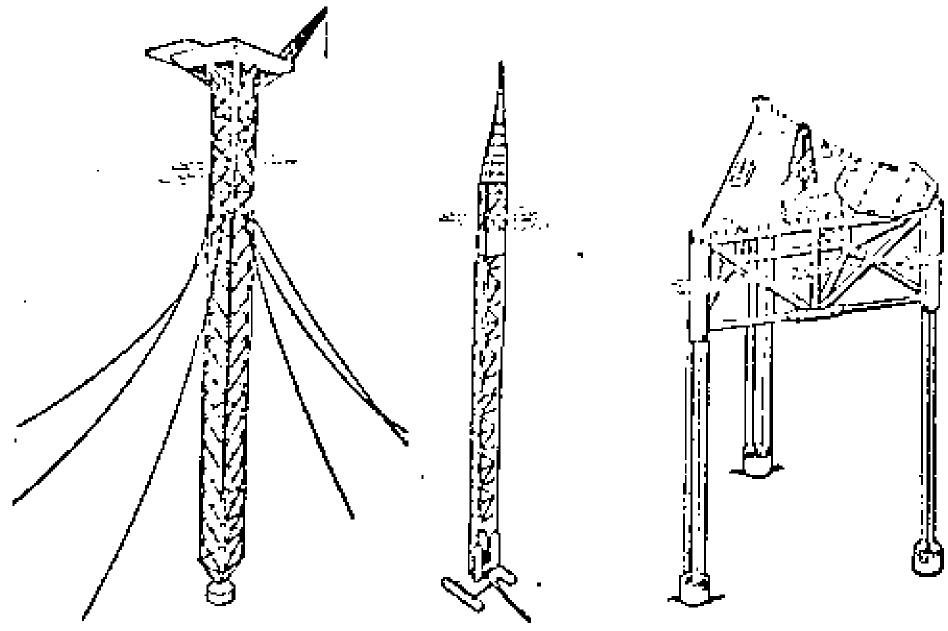
Figure 1.7 Ice-resistant structures in Cook Inlet, Alaska

movements are reduced but at the price of anchor forces increasing. The trade-off in a compliant structure is between excursion amplitude and restraining force. The first crude oil from the North Sea was produced from a compliant 'platform'. This is a modified semisubmersible drilling rig, permanently moored on location in the Argyll field.[10] Purpose made tethered or tension leg structures of various kinds are being developed and deployed as flares, single point moorings, and drilling and production platforms. Some examples are shown in Figure 1.8.

1.3.4 Factors governing form selection

Many factors must be considered in selecting the most desirable offshore hardware. Among the obvious ones anticipated by an experienced 'on-shore' engineer would be:

- capital cost
- cash call
- maintenance cost
- deliverability
- extreme environmental loads
- foundation conditions
- available construction infrastructure and labour
- materials availability



(a) GUYED TOWER (b) ARTICULATED FLARE (c) TENSION LEG PLATFORM

Figure 1.8 Compliant systems. (Reproduced by permission of Inst. Civil Eng.)

The offshore engineer would in addition be preoccupied with:

- accessibility (by sea and air)
- offshore construction weather window
- weather persistence characteristics
- availability of offshore plant (derrick barges)
- reliability

One of the biggest factors is the last, namely, reliability. It is not difficult to see why when one considers the consequences of the failure of a large offshore installation in human, financial and environmental terms.

1.4 DESIGN AND ANALYSIS

1.4.1 Selection of design criteria

Bea[11,12] described a system for the development of platform design criteria and showed how it has been applied to an installation area in the Gulf of Alaska with both earthquakes and waves. The system relies on many years experience, primarily in the Gulf of Mexico with one category of structure, namely, framed, piled steel towers or jackets. It would be highly desirable if this approach were used generally but it is probably fair comment that a rigorous criteria selection analysis of the kind described by Bea is only practised by a handful of companies.

The application of this method to new structural types is difficult in view of the absence of statistics on operational performance. It nevertheless represents the direction in which sound engineering practice must go. A major implication of the approach, is that the so-called 'design-wave' is not a physical event associated with a given annual probability of exceedence or return interval for the site. Rather it is a platform design criterion which is a function not only of the sea conditions but also typical platform resistances. The analyst must hence exercise discretion in using the design wave concept and interpreting the results.

1.4.3 Statutory requirements

These vary from country to country but the internationalism of the industry has resulted in a broad consensus on basic principles. In the U.K., the public interest is protected by the Offshore Installations (Construction and Survey) Regulations Act 1974 which imposes requirements for the certification of offshore installations. This is the only statutory requirement and the act specifies little of a truly technical nature. Advisory information is contained in the notes issued by the U.K. Department of Energy.[12] These are currently being revised and give advice on the selection of environmental criteria, design practice etc. Other bodies such as the American Petroleum Institute, FIP and the Certification Authorities, Lloyds Register, Det norske Veritas, Bureau Veritas and American Bureau of Shipping have issued design rules or guides. Some of these give a reasonably specific definition of the type of problem to be analysed, but there is considerable freedom in selecting the actual analysis method.

1.4.4 Analysis inputs to design decisions

Structures are built as a result of decisions made, not analyses being conducted. The role of analysis is to enable the decisions to be made with an adequate level of confidence. Design is a dynamic process and the degree of analytical refinement changes markedly from the initial feasibility stage through to the final design checks. The methods discussed in the following sections are generally more applicable to the more advanced stages of design. In the interests of a reasonably organized presentation, the analysis requirements will be discussed initially under the headings of the two major classes of structure, namely piled framed towers and concrete gravity structures. The problems of

aseismic design and ice loads are then discussed followed by fatigue and analysis of compliant systems.

1.5 ANALYSIS REQUIREMENTS FOR FRAMED, FIXED STRUCTURES

1.5.1 Fluid loadings

The basic deterministic approach to analysing wave loads on a framed structure comprising linear prismatic elements is to use the well known Morison equation.[14] This states that the fluid force is the vector sum of an inertial and a drag component. These are out of phase due to the phase shift between particle velocity and acceleration. For waves in which the ratio of characteristic structural diameter to wave length is less than 0.2, drag effects dominate. This is the case for most framed towers in fully arisen seas. The calculation of the fluid loads on the entire structure is relatively straightforward, if tedious. Assumptions are required concerning the inertial and drag coefficients C_m and C_D. The water particle kinematics are determined from the incident wave, provided its profile is assumed (e.g. Stokes Fifth) and its height and period are defined. The fluid load on the overall structure is obtained by summing the component forces on each prismatic element, making due allowance for shift in relation to wave crest and angle of attack. This is quite a straightforward mechanistic approach which has been used for many years. It requires no elaborate fluid flow analysis and the validity of the basic Morison equation has been checked many times in the laboratory in the context of frame structures.

Despite all this, there is sufficient doubt as to the accuracy of its application to complete structures for the oil industry to undertake a multimillion dollar, large scale test project in which an Ocean Test Structure[15] is being installed and instrumented to measure fluid loads. The reasons behind some of the suspected inaccuracies concern factors such as energy spreading and directionality characteristics of real seas as opposed to those which are assumed in analysis. In summary, the primary need right now is not for an analysis method which gives a more accurate measure of fluid flow around submerged prismatic members. It is rather a matter of being able to deal with the water particle kinematics in a random three-dimensional sea.

1.5.2 Static frame analysis

There is little that one needs to teach the offshore engineering world about the use of matrix methods for analysing space frames. Systems involving ordinary frame analysis with preprocessors to calculate wave loads have been marketed and used routinely for a number of years. As an industry, this helped to reduce the analysis and design costs to a very low level, so much so that some engineering managers found it difficult at first to adjust to the significantly higher costs of engineering on the major North Sea structures. A minor problem relates to the treatment of the foundation supports. The pile-soil combination exhibits non-linear stiffness characteristics.[16] This is usually allowed for by solving the problem iteratively using a set of non-linear springs at the mudline support points.

The vast majority of steel framed towers and jackets have been installed in less than 100 m of water. Their fundamental structural periods are low compared with the wave periods and dynamic amplification is minimal. Most of these analyses are therefore of a quasistatic type.

1.5.3 Dynamic analysis

Malhotra and Penzien[17] discussed the non-deterministic dynamic analysis of wave excited structures. While analysis capabilities are being developed on a number of fronts there are still severe problems. The coupled soil-structure-water system has associated non-linearities, due to the velocity squared damping from structure-fluid interaction and the force-deflection and damping non-linearities at the foundation level.

The random nature of the loading suggests the use of a stochastic approach combined with modal decomposition to reduce the number of freedoms. This requires the use of a linear transfer function which means eliminating the non-linearities described above. Malhotra and Penzien[17] suggested a method which enabled one to obtain a quasi-linear response operator. However, Tickell *et al.*[18] have shown recently that the use of a linearized spectral approach can lead to large errors when predicting extreme loads on drag-driven structures.

This problem can be eliminated by using a deterministic approach and solving the equations of motion by direct integration. The difficulty one faces is the large number of degrees of freedom and the obvious temptation to use modal analysis or some other form of condensation technique. Furthermore, there is still the need to make allowance for the random characteristics of the loading and such a deterministic approach requires to be interpreted in some statistical fashion in order to provide a meaningful representation of the problem.

Whichever method one adopts, there are two broad areas of uncertainty, the one relates to the modelling of the dynamic system and the other, the characterization of the load.

Ruhl[19] analysed the results of a measurement program on an instrumented platform in the Gulf of Mexico. Predictions of the dynamic response were made on both deterministic and spectral bases and resulted in errors between 13 and 25 per cent, despite good correlation between measured and predicted

structural periods. This implies that attention should be focused on the assumptions regarding loads. The work of Marshall[20,21] regarding directionality and spreading effects may hold the clue.

It is difficult to give a precise definition of the state of the art as practised. Informal enquiries by the author among oil companies and certification authorities in the U.K. and U.S.A. suggest that to date little has usually been done beyond the stage of a first eigenanalysis to determine modal response. This has usually been considered justifiable for installation water depths up to about 150 m since the frequency ratios have suggested low dynamic amplification for extreme events. (This is not the case for the low strain range, high cycle fatigue problems which are discussed in Section 1.8.)

In summary, methodologies for both deterministic and stochastic analysis have been developed, but not widely used. This situation is now changing with the move into deeper waters. Probably the greatest need is for the development of reliable means of simplifying the approaches but still giving meaningful design answers.

There is insufficient published information regarding parametric sensitivity of structural response to factors such as soil-pile stiffness, linearization of drag effects, etc. In short, the analysis methods exist but we lack experience in applying them.

It would be remiss of any discussion on dynamics to ignore factors such as wave slap and slam on horizontal members near the free surface or the hydroelastic oscillations associated with vortex shedding. These dynamic problems are of great importance in the design of individual members but their treatment at the present time is largely empirical.

1.5.4 Joint analysis

Failures in welded tubular frames occur predominantly at the nodes or joints of the structure. These are the most highly stressed regions and the nominal member stresses from the frame analysis do not reflect the true stresses which will affect factors such as fatigue strength and brittle fracture. A detailed evaluation of the stresses at the joints requires a separate analysis of each joint. In actual practice the large number of joints and the existence of a limited number of joint types has led to the use of 'stress concentration factors'. The nominal member axial and bending stresses are multiplied by these factors to give the estimated stresses at the so-called 'hot spots'. The determination of hot spot stresses such as the punching shears shown in Figure 1.9 and derivation of the stress concentration factors is ideally performed by the application of three-dimensional finite element analyses using isoparametric elements.[22] Marshall[20] has described the basic considerations with regard to tubular joint design.

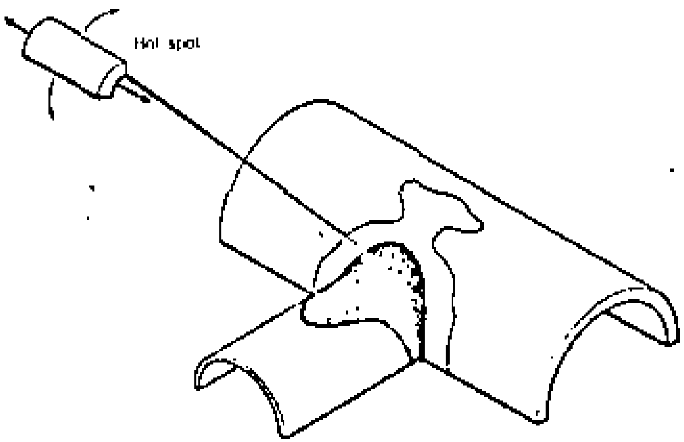


Figure 1.9 Hot spot stress analysis

1.5.5 Foundation problems

The state of the art of pile design for jacket structures was reviewed by McClelland.[23] Soil is a highly complicated material which exhibits among others, plastic, viscous, consolidation, hysteretic and cyclic degradation characteristics. Numerical models of soil behaviour have been used for a number of years. Unfortunately the state of the art of estimating soil deformability properties is so limited that most practical geotechnical analyses, where deformations are of interest, are virtually curve-fitting exercises. This is not meant as an unkind criticism of geotechnical engineers or numerical analysis methods. It is a fact of life that it is extremely difficult to predict soil deformations accurately even on the basis of carefully measured unit soil properties and refined analysis methods. However a combination of field measured full-scale behaviour and numerical analysis can lead to reliable backfigured parameters for use in subsequent analyses.

The three key problems of pile design are:

pile driveability
axial load transfer and stiffness
lateral and rotational pile head stiffness

Most pile driving analyses are based on the one-dimensional wave equation approach using a discretized model developed by Smith.[24] McClelland *et al.*[25] and Fox *et al.*,[26] have reported on its application to actual offshore foundation problems. The analysis is sensitive to a number of inputs involving ill-defined soil properties. The method is widely used but its application is treated with some scepticism in view of some of the assumptions which are required regarding soil parameters.

With regard to the other aspects, it is entirely appropriate to quote verbatim the conclusions drawn by McClelland in his 1974 state of the art lecture.[23]

1. *Axial load capacity.* 'We still have a less than adequate understanding of the soil mechanism that controls load transfer'.
2. *Laterally loaded piles.* 'Less complete information is available for establishing p-y (stiffness) curves for laterally loaded piles in sand.'
3. *Axial deformations.* 'Less fully developed methods are available for numerical definition of an equivalent axial spring'.
4. *Group behaviour.* 'The problem of movement prediction becomes more complicated and at present more uncertain when dealing with pile groups'.

These comments are based on current geotechnical engineering practice which relies extensively on numerical analysis techniques. The array of higher-order elements, dampers, non-linear, gap and slip-stick elements etc available should in theory be quite up to the task of modelling soil-structure interaction problems. It is evident that we have some way to go before the gap between principle and practice is bridged.

1.5.6 Construction and installation problems

The deployment and installation of platform structures has several interesting analysis problems. The first of these concerns the dynamic response during towout and upending or launching. Much of this work is done using model tests. However numerical analysis techniques have several advantages, not only in determining the probable behaviour during the critical stages of upending and sinking, but also in simulation exercises for launch operator training and in real time control systems.

The other aspect concerns pressure resistant design. The buoyancy legs of a large hybrid tower can be easily 5 m in diameter. It is evident that buckling analysis is very important considering the nature of the stiffened shells used, construction tolerances, complicated joint geometry and ambient pressures of 1.5 N/mm^2. The required buckling analysis of a 3-D structure is available in many of the large commercial finite element analysis packages.

1.5.7 Summary

The above brief presentation is summarized schematically in Figure 1.10. The picture is a varied one. At the level of overall frame analysis there is high confidence and everyday usage of numerical methods. Below the foundation level, methods have been developed to model soil-pile interaction but there is as yet insufficient confidence in their application. Fluid loadings on framed structures can be incorporated readily in a regular frame analysis package with the aid of a suitable preprocessor. However there are difficulties in defining a

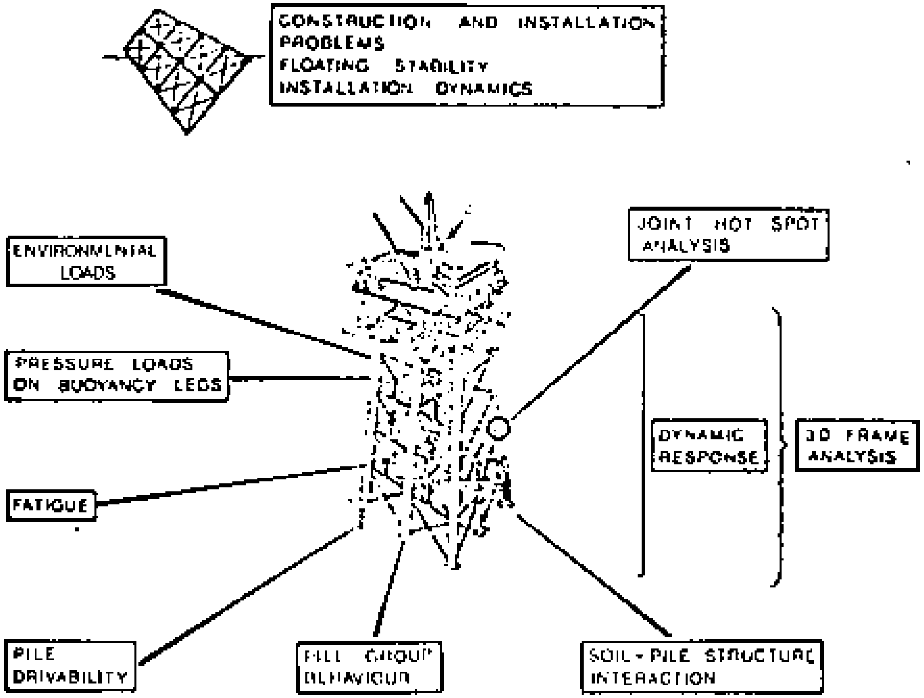


Figure 1.10 Current analytical problem areas for steel towers

suitable three-dimensional load input. Detailed stress and buckling analyses of 3-D problems are required and used. Dynamic analysis fundamentals have been developed but relatively few dynamic response analyses to predict extreme loads appear to have been carried out in practice.

1.6 ANALYSIS REQUIREMENTS FOR LARGE DISPLACEMENT FIXED STRUCTURES

Keulegan and Carpenter[27] showed that the ratio (N_{KC}) of water particle orbit width to characteristic structural dimension defines the relative importance of wave induced drag and inertial effects. As one might expect, the smaller this number becomes, the greater is the relative importance of inertial effects. Where the Keulegan-Carpenter number N_{KC} is less than 5 (equivalent to a diameter/wave length ratio of about 0·2) not only do inertial effects dominate, but the diffraction of the incident wave by the structure must be accounted for in calculating the fluid loads. As an example, for a low sea state with a characteristic zero crossing period of 7 seconds, diffraction becomes important for structural diameters of more than 15 m. This figure would increase to about

70 m for the 12 to 15 second waves associated with fully arisen seas in areas such as the North Sea.

The majority of structures in this category are constructed of reinforced concrete. This type of structure will therefore form the basis of the ensuing discussion though many of the comments, e.g. those relating to fluid loadings, are also relevant to other large displacement structures such as steel storage tanks or some steel gravity structures.

1.6.1 Fluid loadings

In a state of art appraisal of fluid loadings Hogben[28] outlined the relative importance of drag and inertial effects and the diffraction phenomenon's role in the determination of fluid loadings on large objects. MacCamy and Fuchs,[29] and Gran[30] proposed closed form solutions for diffraction around piles and submerged cylindrical tanks. The advent of the large gravity structures in the North Sea created a demand for a general analysis method permitting the calculation of diffraction effects and associated fluid loadings on arbitrarily shaped bodies. Some form of numerical procedure was obviously indicated.

The source distribution method proposed by Garrison *et al.*[31-33] has been very successful in filling this need. Though we do not as yet have adequate data to check its validity in the field, an extensive laboratory test programme conducted by Hogben and Standing[34] indicated high confidence in the method for regular long-crested waves. Finite element methods can also be used to predict fluid loads on fixed and floating structures but successful techniques have only recently been developed and have not yet had much impact in practice. It is the author's opinion that this is one area in which the powerful finite element method may be the less desirable alternative. The real power of the F.E. method is demonstrated in coping with continuum problems characterized by marked changes in the medium properties, e.g. anisotropy, inhomogeneity, complicated geometry, etc. In the present instance, the analyst is primarily interested in modelling the conditions at the boundaries while assuming a predictably smooth variation of the field variables within the fluid continuum. A boundary solution procedure such as the source distribution technique is ideally suited to this situation since one can usually comfortably neglect factors such as inhomogeneity of the fluid medium. The F.E. method however has to pay the penalty of modelling both the boundary and the continuum itself although banded matrix properties reduce the computational effort. A further disadvantage of the finite element method is the difficulty of simulating distant boundary conditions.

One of the most interesting recent developments is the use of solution techniques which combine the flexibility of the finite element method in discretizing the problem in the region of maximum interest with the efficiency of a boundary solution procedure to characterize the remote boundary conditions. Chen and Mei[14] were among the first to use this type of approach in solving a wave refraction-diffraction problem. Zienkiewicz[46,47] showed how the finite element method and boundary solution procedures could be effectively combined in solving a range of field problems. A related development for characterizing remote boundaries is the creation of so-called 'infinite elements' for evaluating fluid-structure interaction problems.[48] These approaches represent a very promising development for other applications also.

In this book recent developments in both techniques are presented (viz. Ch. 3–5) which promise to have an impact on future computation processes.

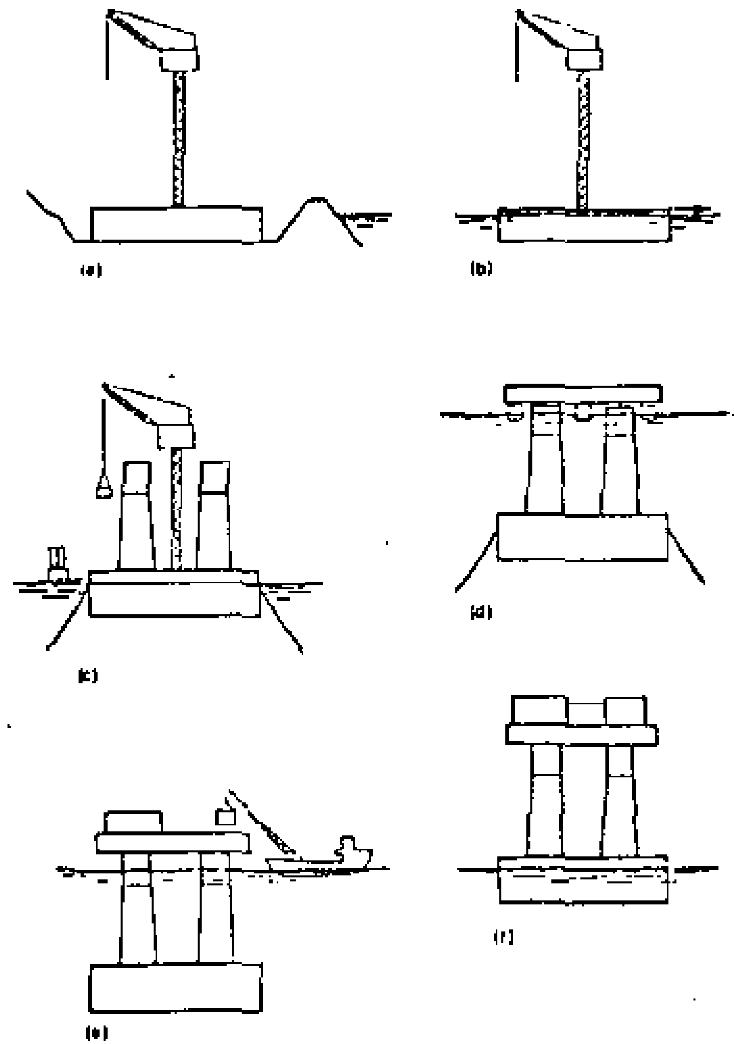
One of the major criticisms levelled in Section 1.5.1 dealing with framed structures concerned the inadequate modelling of the three-dimensional sea behaviour. Similar criticisms will be made of two-dimensional diffraction analyses, once we have obtained reliable field data. Even in the case of a two-dimensional approach, the wave force on submerged objects is quite sensitive to wave period.

There are no ready guidelines for selecting an appropriate period for the design extreme wave. If one adds to this the mitigating effects of energy spreading, it is not difficult to foresee a move towards stochastic analyses with directional and spreading effects accounted for. The diffraction analysis, whether by source distribution or finite element method will still be required however to generate the load transfer functions used in such a stochastic procedure.

1.6.2 Structural analysis

The popular technical press has been more than generous in its use of superlatives to describe the construction of the present generation of North Sea concrete platforms. While the problems of scale undeniably create some interesting challenges for the designer, the emphasis on their sheer size and cost has tended to mask some of the novel design difficulties posed by this new breed of structures. Unlike most land-based installations, many of these problems arise before the structure is finally commissioned to perform its planned function as a drilling and production facility.

Consider the typical construction scenario presented in Figure 1.11. The first stage consists of part construction of the base caisson in a dry dock. When construction has advanced sufficiently for the embryo structure to have adequate strength and floating stability, the dock is flooded and the structure towed to a protected deepwater construction site where it is moored. The remainder of the concrete structure is then completed, with both caisson walls and legs usually being slipformed. The prefabricated deck structure which may weigh up to 10,000 tonnes is then loaded onto the platform legs. This is



Key: (a) Construct part of base in cofferdam.
(b) Float out and moor.
(c) Complete concrete construction afloat.
(d) Float steel deck over ballasted-down structure.
(e) Load modules.
(f) Refloat to towing mode.

Figure 1.11 Typical construction scenario for concrete gravity platforms conveniently accomplished by ballasting down the concrete structure so that the deck can be floated into position over the legs. At this stage, the concrete caisson is only partly flooded and is thus subjected to very large net ambient pressures. Considering that the reserve buoyancy is very small at this juncture, it is evident that the deck loading operation represents a critical stage in the construction. Deballasting of the structure enables the deck to be lifted off its delivery pontoons and the structure is then raised to an adequate level for loading of the modules of topside equipment.

The structure complete with its large payload is then refloated to the planned towing draft and the tow to location carried out when suitable weather conditions are forecast. On reaching its installation location, the structure is ballasted until touchdown is effected. Damage due to impact on touchdown is avoided by a slow descent and the use of downstand dowels to arrest structural oscillations during the final stage of descent. Virtually all of the structures to date have been equipped with steel or concrete skirts. These are driven to the required penetration by further ballasting. The water filled voids under the structure may be grouted before final flooding of the structure is carried out.

Once it is in operation, the storage of hot crude or the presence of the hot well conductors and marine risers inside the structure will result in differential heating of the structure and associated thermally induced loads. Thus in addition to the much vaunted need to resist a 100 year wave of the order of 30 m the structure must be designed to be satisfactory in respect of other characteristics such as:

- watertightness
- resistance to high pressures
- floating stability
- towout draft limitations
- payload carrying capacity
- placement on an unprepared site
- resistance to thermal loads

This is by no means an exhaustive list but it suffices to indicate that the designer's options are constrained in several respects at all stages from construction through operation. It is clear therefore that while the concept of a concrete gravity structure is simple, its behaviour will not be. An exhaustive discussion of these problems is clearly outside the scope of this presentation but certain problem areas are worth highlighting.

1.6.2.1 Pressure loads

During the critical deck mounting operation, parts of the caisson are subjected to differential pressures typically in the range 1·3 to 1·6 N/mm^2 or at least three hundred times the design distributed live load in a typical building. Two

basic structural systems have been used for the majority of structures to date. These are shown diagrammatically in Figure 1.12. In the first of these, the caisson consists of an assembly of cylindrical cells as shown in Figure 1.12(a). The alternative system shown in Figure 1.12(b) comprises a cellular box with straight walls which act as a system of struts to support a pressure resistant skin.

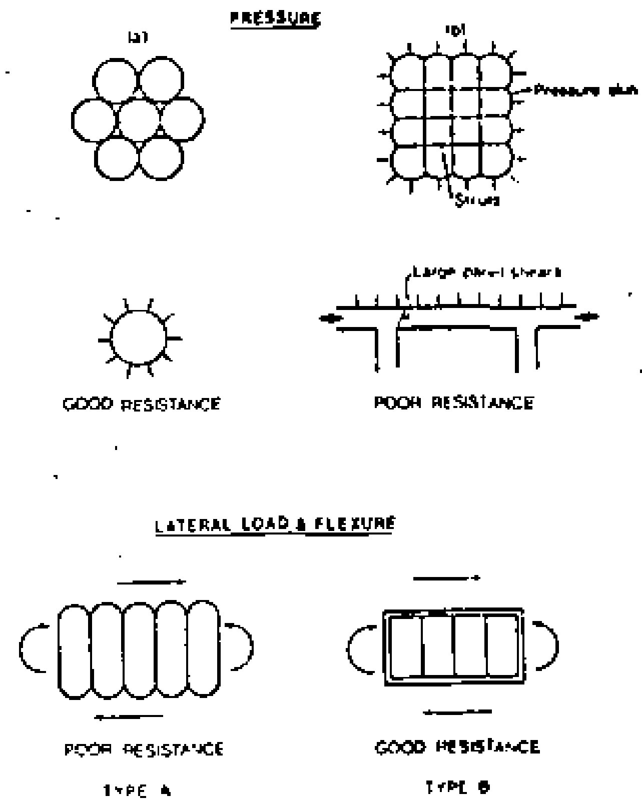


Figure 1.12 Typical caisson structural systems

In both cases, the ambient pressure is accepted by cylindrical surfaces which ensure that the pressure loads are resisted by direct membrane compression, at least in plan. When one looks at the method for capping top and bottom of each of these systems, a different picture emerges. The first system can be conveniently capped with hemispherical domes which will permit the simple membrane action of the concrete to be retained. In some cases, a larger radius of curvature dome has been used to permit the elimination of the top shutter during construction. While this has construction advantages, it introduces large thrusts and associated bending near the springing points. The closing of the square cellular base has been most frequently accomplished by the means of flat slabs although in certain instances polyhedral capping surfaces have been used.

The first question to be considered is that of overall stability. The thickness to diameter ratios of the typical concrete cylinders shown in Figure 1.12(a) are usually high enough to ensure thick shell type behaviour and a fairly small risk of elastic instability. Similarly, for the rectangular configuration shown in Figure 1.12(b), cell sizes can usually be selected which will ensure that individual cell walls are unlikely to give buckling problems. However, for this type of system, it is possible that overall instability may be associated with the behaviour of the complete cellular structure. This can obviously be analysed using an appropriate buckling analysis and there is nothing in the boundary conditions or element configurations which appear to be particularly exacting.

The biggest problem appears to be associated more with the question of the triaxial stress state and the exceedence of the appropriate rupture criterion for reinforced concrete. This has been described as 'implosion' by Haynes,[39] who has carried out a series of experimental investigations on the failure of concrete pressure vessels under high ambient pressures. Although no general theory exists to date, the evidence available would suggest that the problem lies not so much in determining the appropriate stress conditions as in the selection of the correct failure criterion. It still remains to be proved how important factors such as strength loss of the concrete due to the build-up of pore pressures are in influencing the resistance to high pressures.

Since the boundary conditions are so simple, the overall analysis of pressure loads is usually quite trivial and the only difficult analytical aspect relates to the effect of the restraints imposed by adjacent parts on the structure. For example, the influence of a fixed connection between the cylinders in Figure 1.12(a) on shear stress and bending has to be investigated by means of a stiffness analysis. The suite of elements available in most of the major analysis packages is quite adequate for the purposes of this type of investigation. A slight level of sophistication may be introduced by the investigation of the effect of creep when considering that some of these pressure loads may be acting for many months. As the stress distribution will remain sensibly constant, the problem is clearly one of finding the right constitutive law rather than of conducting a detailed time-dependent analysis to determine deformations.

By far the biggest problem with regard to pressure loads is one of design rather than analysis, in that there are few guidelines to assist the engineer in designing for the very large panel shears and in particular, the allowance which should be made for the assistance given by axial prestress. The British Code CP110 reduces the allowable shear stress in slabs to half of that in beams, without stating why this is the case although it is suspected that this has to do with the non-uniform distribution of shear forces around the perimeter of

non-circular plates. The ACI Code 318/71 allows the designer some freedom to use the influence of the axial prestress but it is still a far from satisfactory situation facing the designer. Clearly, there is considerable scope for investigating the behaviour in shear of the heavily loaded thick slabs.

1.6.2.2 Thermal loads

The problems with thermally induced loads in nuclear pressure vessels have been studied for many years and it is tempting to believe that a direct transplant of technology would suit the problems of thermally induced loads in concrete platforms. The pressure vessel consists essentially of a single structural element which, apart from start up conditions is associated with a steady state heat flow regime. The gradient of temperature and pressure is outward and apart from the problems created by local penetrations, the thermal regime is fairly uniform throughout. This is fundamentally not the case for a concrete gravity platform, particularly where there is storage of hot crude. In the latter event, parts of the structure are heated while others are quite cold. A hot/cold fluid interface exists in one or more parts of the structure and its position moves with time. Load cycling is a characteristic feature, despite operational attempts to maintain a sensibly constant thermal regime.

Clearly, the problem is a fairly complicated one in several respects. In the first instance, it is necessary to establish a typical range of thermal regimes, including transient states to be designed for. Coupled convective and conductive heat transfer complicates the problem of analysis and most practising engineers have thus far been prepared to make the convenient assumption that steady state conditions are obtained quite smoothly and rapidly and that problems of thermal shock need not be assessed. Analysis techniques for carrying out such investigations were developed by Hsu and Nickell[40] and applied to a class of initial-boundary-value problems that are commonly referred to as diffusion-convection problems. The author is unaware of any published results based on the use of methods such as these to investigate typical thermal regimes in concrete storage vessels.

Even given a sensibly constant thermal regime within the structure, the thermoviscoelastic nature of concrete results in several time dependent phenomena of significance to the designer. An analysis of typical loads associated with assumed elastic behaviour of the structure shows local loads at an unacceptably high level. Richmond[41] has shown how creep can assist to provide a marked reduction in the thermoelastic stresses. On the other hand, he has also shown that particular combinations of prestress and thermal loading can result in higher stresses when the thermal differential is removed than are incurred under the initial elastic regime. Most important of all, creep effects in redundant structures can cause locked-in stresses after the removal of the initial load regime. Reversal of the load will then create a far worse condition than occurred on first loading. The initial evidence would suggest therefore that as in the case of many other structures, creep has an ameliorating effect on stresses unless cycling of the load occurs.

1.6.2.3 Environmental loads

Section 1.6.1 described the type of diffraction analysis which can be used to predict the distribution of pressure over the structure resulting from wave loading. Pressure variation over the structure is quite smooth and the analysis of structural load distribution under environmental forces is relatively straightforward insofar as fluid loadings are concerned. The question of seismic loads is considered later.

1.6.2.4 Foundation loads

The local distribution of forces applied by the foundation to the structure is less reliably predicted than the loads caused by environmental factors such as waves and currents. Starting at the time of installation, the structure is installed on a totally unprepared site which has its own topographic and stratigraphic characteristics. Not only is it difficult to sample these adequately in advance, but it is also a fairly demanding exercise to emplace a structure with a displacement of over half a million tonnes in exactly the planned location.

Bjerrum[42] described some of the problems to be anticipated. Apart from the obviously expected moments, torsions and shears induced by non-uniform resistances over the site, local pressures of the order of 2·0 N/mm^2 can be applied to the underside of the structure by high spots. Such pressures, when superimposed on the already very high hydrostatic differential pressures can create an almost intolerable situation for the structural designer. Very often, the problem is turned around and the question becomes one of designing a structure-foundation interface such that these difficulties do not occur.

The entire philosophy of foundation design is different from that of a typically landbased structure due to the uncertainties regarding the site properties as well as the fact of placing the structure on an unprepared bed. The geotechnical engineer must therefore provide the structural designer with a range of terrain models which will embrace the expected range of conditions anticipated at the installation location. These can then be used to generate support conditions for analysis of the forces in the structure resulting from installation as well as the response to environmental loads.

Both the short and long term reactions must be considered and some of the special influences due to effects such as cyclic loading must be considered as discussed in a later section.

1.6.2.5 Other structural analyses

There are a number of other problems as one might anticipate with regard to the design of the structure. Among the current preoccupations of the industry is

the question of fatigue. This relates not only to the problems in the concrete part of the structure where tension crack propagation, fatigue of the reinforcement, bond degradation etc are less than perfectly understood, but also with regard to the design of the deck. Most of the decks which have been used to date have fully fixed moment connections at the top of the concrete legs. Under the action of wave loads it is evident therefore that cycling of the stresses in a steel deck will occur, requiring an investigation of the fatigue problems associated with these stress reversals. The question of high cycle low amplitude fatigue is discussed in a later section.

1.6.3 Geotechnical analyses for gravity platforms

Excessive conservatism in the design of a gravity structure foundation is very costly. If one adds to this the problems associated with installing a complete structure on an unprepared site with somewhat uncertain foundation conditions, it is clear that the geotechnical engineer's role in the design of the gravity structure assumes a greater relative importance than it does for most conventional landbased projects.

1.6.3.1 Installation problems

Among the most interesting of the geotechnical analyses required for the successful installation of a gravity platform is the prediction of skirt penetration resistance. At first sight, this would appear to be a relatively straightforward matter of applying limit analysis techniques such as are employed in conventional bearing capacity problems. An examination of the theories for predicting deep foundation bearing capacity by Vesic[43] showed an order of magnitude difference between the various theories for predicting bearing capacities in sand. Work by Kerisel[44] had indicated practical upper limits on resistance which do not agree with classical strength theories and penetration tests results from North Sea site investigations showed far higher penetration resistance than could reasonably be expected from the properties of typical soils.[42] Current practice tends to follow the pile design type of approach with due adjustment to allow for an upper limit on penetration resistance as revealed by Kerisel. The problem is not simply resolved as in the case of bearing capacity analyses in which one takes a conservative approach and picks a low value. In the present circumstances, the platform cannot be left sitting on partially penetrated skirts and a high premium is therefore placed on obtaining a reliable estimate of the maximum resistance likely to be encountered. The present state of affairs is unsatisfactory in analysis terms and there is obviously scope for a general investigation of the problem. Considering the inhomogeneity of typical installation sites, and variation in skirt profiles, a general numerical analysis technique probably based on a Lagrangian formulation with a general dilatant constitutive law could perhaps provide the answer.

1.6.3.2 Stability analyses

For the extreme environmental load case, the force resultant at the mudline is characterized by much larger values of obliquity and eccentricity than are typical for the majority of foundation problems. It is therefore necessary to use a generalized bearing capacity theory which takes full account of the three-dimensional nature of the problem. The most general theory of bearing capacity for such situations was developed by Hansen[12] and his procedure has been fairly widely used, despite its obvious drawbacks. Among the most serious of these is the assumption of homogeneous foundation conditions, a condition which is unlikely at best for most gravity structure sites. Accordingly, other limit analysis techniques such as the method of slices are being increasingly used. Vaughan *et al.*[46] applied finite element techniques to the analysis of the stability of large gravity structures, taking into account the variation in shear strength with depth. A primary drawback with this approach is that for reasons of economy, the analysis is usually limited to the assumption of plane strain conditions and the results therefore are only likely to be representative of the conditions near the plane of rotation. While they are useful in identifying the initiation of yielding and the development of failure modes, the prediction of overall bearing capacity is likely to be somewhat suspect for the truly three-dimensional nature of the problem. It is the author's view that a combination of such analysis methods together with model testing approaches such as those used by Rowe *et al.*[47] will provide the most useful means of attacking the problem of predicting both bearing capacity and deformation of the foundation.

1.6.3.3 Hydraulic instability

Henkel[4] identified the role of waves in inducing bottom instability and it is generally recognized that any foundation stability analysis must take into account the effective change in surcharge caused by the transient pressure wave. However, this phenomenon and the obvious question of scour around the structure are not the only hydraulic factors to be investigated in the design. The presence of virtually incompressible ambient fluid during the application of overturning forces to the base of the structure results in the development of a pressure differential between the underside of the structure and the surrounding free water. In addition to the obvious question of eliminating potential piping problems around skirts, it is in some cases necessary to evaluate the rate of dissipation of this pressure differential in order to check the validity of the assumptions with regard to the overturning environmental loads. For example, it is fairly common practice in predicting the wave loads on a gravity structure to assume a so-called 'closed gap' under the structure. This assumption is only valid if the water pressures under the structure are not dictated by the hydraulic boundary conditions outside the skirts. The investigation of this requires one to look at the response of the foundation as a consolidation phenomenon. Approximate analyses carried out by Chan[48] using a simplified finite difference model indicated that this is a reasonable assumption for typical skirt configurations and soil conditions in the North Sea.

1.6.3.4 Cyclic degradation effects

The questions of liquefaction and cyclic softening of clays have received considerable attention during recent years, particularly the latter in view of the predominantly clay profiles underlying the gravity structures installed to date. (Note: The Ekofisk Tank[9] is the major exception.) A joint industry project on the repeated loading on clay has been carried out and the results published by Andersen *et al.*[49] The programme showed quite clearly that cyclic loading effects could result in a significant change in both stiffness and shear strength of normally and overconsolidated clays. Despite the substantial nature of the investigation, and the advances which have been made in understanding the effects of cyclic loading on soil specimens, no generally accepted methodology has yet been developed for conducting both stability and deformation analyses which incorporate the findings of this programme.

1.6.4 Dynamic response to waves

The prediction of the dynamic response of a gravity platform to environmental load transients such as waves or earthquakes represents a very challenging set of demands largely because of the coupling between the fluid-structure-soil systems. General methods have been proposed by Eatock-Taylor[50] and Moan *et al.*[51] for the dynamic response of structures to wave inputs. The application of such methods is still very much in its infancy and there are many fundamental questions to be answered, from the validity of two versus three-dimensional models to the selection of realistic sea state spectra. The response of coupled dynamic systems forms a major part of the proposed discussion for this conference. The following paragraphs are intended to provide a brief introduction to some of the factors involved and their relative importance as seen in the context of the present state of the art of actual design.

1.6.4.1 Structure-water coupling

Both finite element and source distribution techniques have already been used to investigate dynamic coupling between submerged structures and their ambient fluids. Liaw and Chopra[52] used a finite element characterization of both structure and fluid to assess the effects for a typical submerged intake tower responding to seismic excitations. A source distribution technique was used by Garrison and Berklite[31] to assess the dynamic response of other bluff bodies. The work of these investigators has shown that in most cases, the dynamic system can be adequately characterized by using the so-called 'added mass' approach to allow for the participation of the ambient fluid in the response. Added mass factors are relatively insensitive to frequency except near the free surface where gravity waves generated by the structural motions attenuate the added mass effect, but there are no definitive guidelines yet as to how this and other forms of hydraulic damping should be incorporated in a dynamic response model. Among the other factors still to be thoroughly explored is the degree of coupling between the added mass contributions in the different modes of motion.

1.6.4.2 Foundation-structure coupling
The aseismic design of nuclear reactors has resulted in an intensive study of the influence of foundation-structure coupling in relation to earthquakes. Despite the difference in the frequency contents of earthquake versus wave excitations, the basic physical phenomena are still similar. A full evaluation of the methods for analysing these effects was carried out by the American Society of Civil Engineers and the results are due to be published shortly (see ASCE 1976).[52] The key conclusion from this work is that the soil-structure interaction effects have a very significant influence on the response of both structure and foundation. Most of the experience which has been gained has been in relation to nuclear power plants and much of the work currently being done on developing suitable structure-foundation models for gravity platforms is based directly on its findings.

1.6.4.3 Selection of analysis model
Provided that remote boundary conditions can be adequately characterized, it is possible to idealize both structure and foundation using a finite element model. The primary advantage of this approach is its suitability for inhomogeneous foundation conditions and non-linear response of both structure and foundation. Unfortunately, it has some severe disadvantages, not only in terms of overall cost but also because the technique frequently requires one to use large numbers of freedoms to discretize the foundation at the expense of a less accurate characterization of the structure itself. Consequently, there is a strong tendency towards the use of closed form impedance functions to define the foundation support conditions. Standard solutions are available for half-space conditions,[53] but this is considered a doubtful assumption for structures on the scale of a typical gravity platform where a significant change in soil stiffness with depth is likely to occur.

A two stage procedure is sometimes used, the first comprising a dynamic finite element analysis of the foundation in order to determine the frequency dependent foundation impedances, followed by the use of these in a dynamic response analysis of the structure itself.

1.6.4.4 Deterministic analyses for waves
Relatively few response history analyses of the dynamic response of a gravity structure to a wave train have been published. There are several reasons for this, only one of which is the cost. The only attempts of which the author is aware have been based on the direct integration approach which has several theoretical difficulties associated with it. In the first instance, there is no simple way of incorporating frequency dependent foundation behaviour in a time domain analysis. It can be argued with some justification that the foundation impedance is relatively insensitive to the range of frequencies involved in wave excitations, but this is only likely to be true in the lowest modes. The second and more compelling reason is the fact that the forcing function itself is dependent on the frequency content and amplitudes of the wave train and therefore it is not possible to generate a simple correlation between forcing function intensity at various parts of the structure and sea surface elevation with time.

The author and his colleagues have formulated an approach[55] which may be used to obtain a time history response to waves, incorporating frequency dependent effects in both foundation and wave input. This is based on the use of discrete Fourier Transforms in a similar fashion to the approach used for the analysis of earthquake response problems.[54]

1.6.4.5 Stochastic analyses
Even if one is successful in carrying out a response history analysis of the kind mentioned above, the validity of the wave train itself is bound to be suspect. In the first instance, considering the response to extreme events, there is an obvious lack of wave train data and one is forced to generate a synthetic record or to scale up from measured wave trains corresponding to a higher probability event. Secondly, the random nature of the sea implies that a number of such analyses would have to be carried out in order to obtain an adequate statistical interpretation of the problem. It is not surprising therefore that most investigators appear to favour the use of a stochastic approach where the sea state is characterized by a power density spectrum. This approach is also subject to limitations, the first and most obvious being the use of a linear transfer function in order to obtain the response spectrum. Fortunately, concrete gravity structures are better favoured in this respect than steel templates since the wave driving on a gravity structure is predominantly inertial and the response operator can be treated as linear with some justification. Also, the foundation response is sensibly linear for the usual range of foundation safety factors for the 100-year event. There are inevitable arguments as to the suitability of various standard wave input spectra for these analyses and attention to this is given in several chapters of this book.

1.6.4.6 Designing for dynamic effect
As stated earlier, the analysis of dynamic response of gravity structures is at an

early stage of development and it is not surprising therefore that there are as yet no generally agreed design criteria to be used in relation to dynamic response calculations. Several sources of uncertainty have been mentioned above, probably the most important of these is the foundation compliance. As in the case of piled structures this is of fundamental importance and moreover is difficult to predict in advance. It should thus be treated in a parametric fashion.[51]

1.6.5 Review of analysis requirements

The above areas of current analysis interest are summarized in Figure 1.13. Although the list is not an exhaustive one, it contains a considerable variety of problem types ranging from three-dimensional stress analysis, through steady and transient flow problems to viscoelastic and elastoplastic deformations in structure and foundation. The engineering analyst would be hard pressed to find an equivalent set of challenges in any other single project.

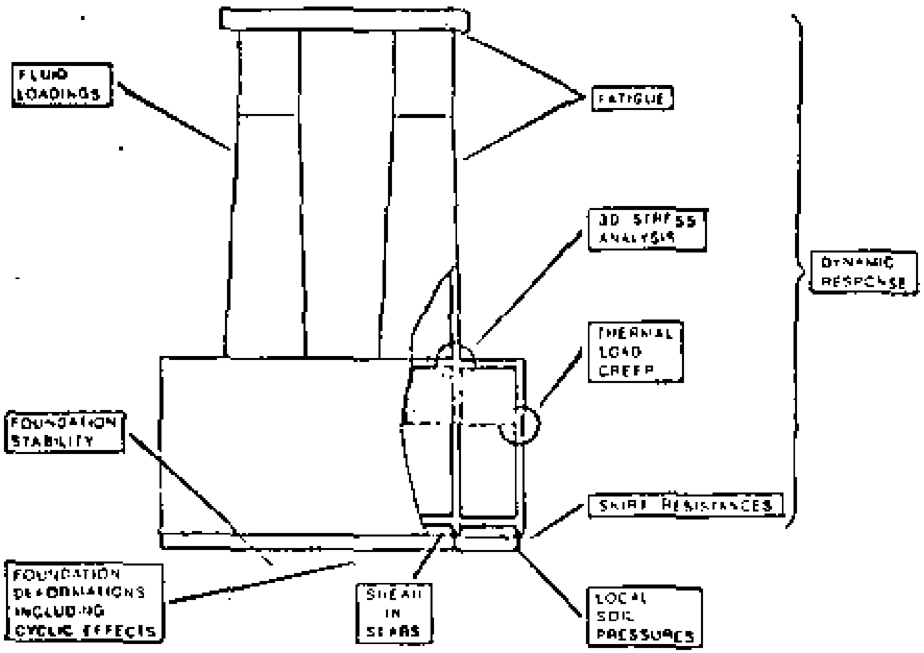


Figure 1.13 Current analytical problem areas for gravity platforms

1.7 OTHER ENVIRONMENTAL LOADS

The introduction to this chapter identified several forms of environmental loading which are significant in the development of a safe design for an offshore

structure. Among these are winds, currents, floating ice and earthquakes, but the first two are usually of secondary importance. It is only relatively recently that the question of aseismic design offshore has attracted much interest; however the development of fields offshore southern California and New Zealand and the lease sale earlier this year in the Gulf of Alaska have resulted in an apparent gearing up by the industry to tackle these problems. Another problem area which has attracted some interest is that of loads due to floating ice. These two issues will be briefly considered.

1.7.1 Seismic loads

Evidence of an increased awareness of the seismic design problem is the fact that the most recent issue of the American Petroleum Institute's API RP2A[56] (Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms) in January 1976 introduced a substantial section on earthquake design. The available methodology for conducting seismic analyses of jacket or tower type structures was reviewed by Penzien,[57,58] and their application to the design of platforms for regions such as the Gulf of Alaska was summarized by Bea.[15] The structure-water coupling and the associated non-linear drag term introduces an obvious limitation when the analysis approach requires the assumption of linearity. Penzien proposed a stochastic technique in which a quasi linearization procedure is used to overcome this. However, this is not the only source of difficulty since the foundation interaction problems are also severe as had been pointed out by Parmalee *et al.*[54] One result of the investigations conducted by Parmalee, Penzien, Bea and others over the past few years has been an increased awareness of the need to understand the soil-structure interaction problems as the key to aseismic design and a major research effort is presently being undertaken by the industry in this regard.

The development of earthquake resistant designs for gravity platforms has received much less attention than is the case for tower structures. Penzien and Tseng[60] developed an analysis procedure based on the use of discrete Fourier Transforms similar to that used by Veletsos.[54] The writer and his colleagues[61] showed that the loads predicted for large magnitude earthquakes by means of this approach can be significantly larger than those due to a 30 m wave. This work has once again demonstrated the fundamental importance of soil-structure interaction effects. Although we are able to borrow extensively from the methodology developed for nuclear reactor design, the mass and stiffness characteristics of a typical gravity platform differ from those of a nuclear power plant and much remains to be done in understanding the importance of these differences. For example, Figure 1.14 shows the seismic response of a typical gravity platform to a natural earthquake (TAFT accelerogram). The structure-foundation system has a fundamental period of about 4·5 seconds and the

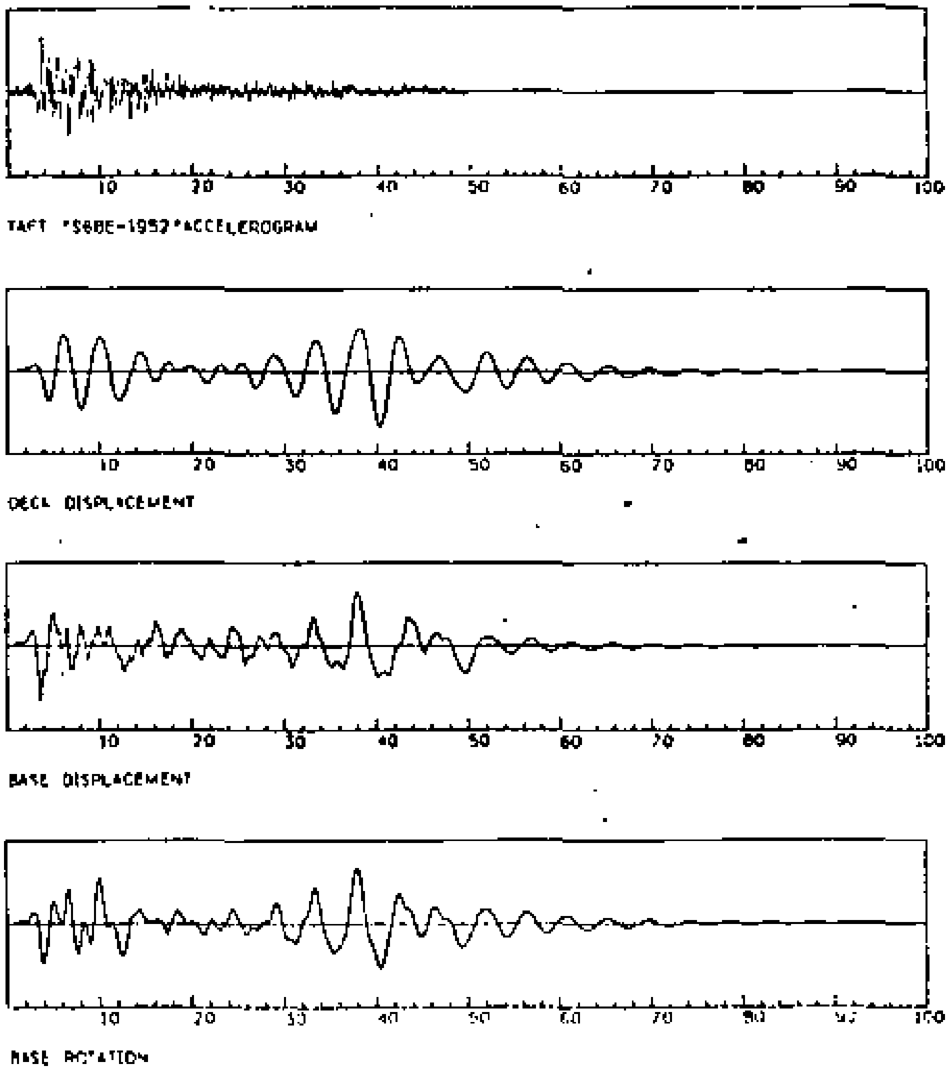


Figure 1.14 Seismic response of gravity platform

response at deck level at a late stage in the duration of the record clearly demonstrates the importance of the low amplitude, long period components in the earthquake accelerogram. In making this point, it is worth remembering that the biggest uncertainty in earthquake engineering is the earthquake itself.[62] Other investigations are in hand to evaluate the non-linear response of such systems to extreme magnitude events in order to assess their survivability characteristics and develop appropriate design criteria. The writer is confident

that it is a matter of time before gravity platforms are constructed for earthquake regions and that the investigation of the dynamic response problems will attract a lot of attention in the near future.

1.7.2 Loads from floating ice

Fixed platforms designed to cope with floating ice have been installed and successfully operated in Cook Inlet, Alaska for about a decade, but apart from some man made islands in the Canadian Arctic, there are as yet no offshore platforms in a true Arctic region such as the Beaufort Sea. Work has been in progress for some time on the development of suitable design methods for such installations and America, Russia and Japan are developing extensive ice engineering facilities for the purpose. Almost nothing has been published on the use of numerical methods although some proprietary studies have been made.

The emphasis for fixed structures must be on horizontal loads. Most attempts to predict these have been based on model tests but the time is ripe for developing numerical techniques and calibrating these with the model test results. There are the inevitable problems regarding input data since the mechanical properties of ice are not well understood. In addition to brittle failure characteristics sea ice exhibits marked creep behaviour and is far from being homogeneous and isotropic. If one adds the obviously three-dimensional nature of the problem, there are clearly plenty of challenges facing the numerical analyst. It would seem that much of the work in the short term will be of a conceptual nature.

1.8 FATIGUE

The prediction of the fatigue life of an offshore structure, or more precisely the fatigue life of a particular joint in the structure, is rapidly becoming one of the most crucial areas of investigation in offshore engineering. This is not because the problem of fatigue has only recently been recognized, but rather because it has until recently been primarily associated with low cost structures such as the caisson described by Hong and Brooks.[67] In a review of the problems of tubular joint design, Marshall[71] stated that at that time, no fixed template type platforms had collapsed as a result of fatigue failure, even though there had been problems with waterline braces. The primary reason for this increased interest is the focus on high cycle, low strain amplitude fatigue where experience of cumulative damage problems has been rather limited. The importance of fatigue analysis in the design of the current generation of steel offshore platforms for the North Sea was recently reviewed by Williams and Rinne.[63]

A basic fatigue analysis consists of three elements, namely, the determination of the relationship between 'hot spot' stress and wave amplitude, the prediction of the number of load cycles at each strain or stress amplitude and the substitution of the stress amplitude-load cycle data in some cumulative damage criterion to determine the fatigue life. Virtually all of the published work on fatigue has been based on the well known Palmgren-Miner cumulative fatigue damage law.[64,65] This linear damage criterion underlies the so-called design codes which have been published by authorities such as Lloyds, the British Standards Institution and the American Welding Society. Credibility of the linearity assumption was defended by Marshall[20] as being justifiable within the current range of design uncertainties, but this assumption still continues to attract the criticism of fatigue researchers such as Kirk,[66] who maintain that the linear damage theory results in a significant overestimate of fatigue life.

The major differences of opinion relating to fatigue design however seem to originate in the matter of treating the wave loads, and two basic schools appear to have emerged. The first of these is basically a deterministic approach, as suggested by Maddox,[67] who opted for a deterministic, rather than a stochastic treatment of the dynamic amplification effects in view of the non-linear drag problems which were discussed earlier. Maddox used modal analysis and a time history integration of the equations of motion in order to determine stress responses at the hot spots and a statistical treatment of the long term wave data in order to assess fatigue life in terms of the Palmgren-Miner rule. Other investigators such as Vugts and Kinra,[68] took a probabilistic approach to the fatigue analysis problem and considered that the linearity assumption could be justified since the low amplitude, short period waves associated with the fatigue problem implied predominantly inertial, rather than drag driving of the structures. Williams and Rinne[63] compared both deterministic and spectral methods considering both quasistatic and dynamic analyses. Not surprisingly, they pointed out that one of the biggest problems associated with either method is the selection of appropriate input data regarding sea state and wave force calculation.

Among the factors emerging as being of primary importance are the directionality and energy spreading characteristics of the sea. Some of the implications of these factors were discussed by McDowell and Holmes,[69] and Marshall[21] who developed a dynamic and fatigue analysis procedure using directional spectra and applied this to a 1000 ft platform currently being constructed for the Gulf of Mexico. This work has shown that the three-dimensional nature of the sea must be included in any fatigue analysis in order to obtain meaningful results.

1.9 OTHER STRUCTURAL TYPES

Based on our experience to date, the cost of a fixed platform would appear to increase exponentially with the water depth on location. It is inevitable

therefore that the offshore engineer should be seeking new structural systems to meet the demands of the frontier areas. An obvious solution is to provide systems which can function entirely on the seabed, such as the wet or dry subsea completion systems which have been developed and are being actively marketed at present. The only systems of this kind which have been installed to date are being used in association with other more conventional oil field hardware and there are strong grounds for believing that it will be some considerable time before a totally self-contained system can be developed which will eliminate the need for a permanently emplaced surface vessel or platform. There is an obvious interest in developing so-called compliant systems which represent a reasonable compromise between the need to maintain such a link and the cost of providing it by means of a rigid structure. Some of the design problems associated with compliant systems were reviewed by Godfrey[70] who pointed out that there are severe design disadvantages for certain types of compliant structure. Nevertheless, the industry is confident that such systems have a future and large scale test programmes have been carried out on the guyed tower and the so-called tension leg structure shown in Figure 1.8 and several articulated flares have been installed. One of the main reasons behind carrying out such test programmes is the difficulty of obtaining realistic predictions of the motions of such systems in different sea states. The pursuit of suitable compliant systems will inevitably lead the analyst into some intriguing non-linear dynamic regimes.

1.10 CONCLUSION

This paper has attempted to provide an overview of the technical problems which are of primary interest to the civil engineer operating in the offshore environment at present. The range of problems is so large that the treatment has necessarily been superficial. However, it is hoped that it has served to illustrate those areas in which numerical analysis techniques are currently being applied in practice.

Several common threads run through most of these problem areas, one of the most obvious being the coupled nature of the problems. Not only does this make the problems themselves more interesting, but it has the added advantage that it demands the closest collaboration between engineers working in the different fields of structural, soil and fluid mechanics. The offshore engineer must address problems in all of these areas simultaneously if reasonable solutions are to be found.

A second obvious factor is the difficulty of characterizing the random environmental forces, whether these are due to winds, ice, earthquakes or waves. In conclusion, it is appropriate to quote directly from the excellent text by Blair Kinsman[71] on wind waves. 'It is very easy to forget the chaotic state of the sea when one is engaged in theoretical work. Our means are seldom commensurate with our ends, and the only way to make progress is to simplify and regularize. When one has given a great deal of effort and thought to a problem, one has an emotional investment in any results obtained. It is very difficult for any man to evaluate justly the distance his work lies from 'physical reality', or even the distance it lies from the problem he would like to have solved. Most of us need to be reminded. Therefore, go wave watching. If you will watch waves with a seeing eye, you will never confuse the regularity of any simplified approximation, like the sinusoid, with ocean waves as they really are'.

Acknowledgement

The author is indebted to Ove Arup & Partners for permission to publish this survey.

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DIVISION DE EDUCACION CONTINUA
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CURSO:

INGENIERIA OCEANICA

CRITERIOS DE DISEÑO DE PLATAFORMAS MARINAS

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MARZO, 1982.

I N D I C E

1. INTRODUCCION

En este trabajo se describen los aspectos principales del diseño estructural de las plataformas para explotación de hidrocarburos fuera de la costa. Se mencionan inicialmente los tipos más comunes de plataformas (de acero y de concreto) y se describen brevemente los procedimientos de construcción, transporte e instalación; se señalan los problemas que presenta cada tipo de estructura, así como ventajas e inconvenientes de las plataformas de acero y de concreto. La atención se concentra en las estructuras para plataformas de profundidades intermedias, de 30 a 100 m.

Se presentan los criterios que se siguen para la selección de la forma estructural más apropiada y para la determinación de las solicitaciones de diseño; se describen los procedimientos de análisis y dimensionamiento más comunes, haciendo referencia a la práctica recomendada por los organismos más reconocidos internacionalmente.

Las primeras perforaciones petroleras en la plataforma continental se efectuaron en la década de los treinta. En la actualidad 20% de la extracción petrolífera mundial procede de las regiones marítimas, siendo el porcentaje más elevado aún en lo que se refiere al gas natural.

La búsqueda de los yacimientos marinos sigue concentrándose en las plataformas continentales cuya superficie hasta la profundidad de 200 m se equipara a la del continente africano.

En el borde continental se intuyen grandes depósitos de hidrocarburos lo cual plantea un desafío a la tecnología moderna. El gran avance tecnológico obtenido en la última década con el desarrollo de de la explotación de yacimientos del Mar del Norte y Golfo de México a profundidades superiores a los 150 m no habrá de ser sino un eslabón con el devenir de la ingeniería del futuro.

La investigación, la experiencia y la práctica en el diseño y la construcción de plataformas de acero y concreto, han venido conduciendo a un entendimiento más profundo del comportamiento estructural de ambos tipos de plataformas, sin embargo el mejor conocimiento de las solicitaciones. La creciente importancia dada a los aspectos de seguridad y protección ambiental y los accidentes ocurridos en los últimos años han dado lugar a que la filosofía de diseño se encuentre actualmente en plena evolución.

El objetivo que se persigue en el diseño de plataformas marinas, al igual que en cualquier otra estructura, es lograr que en las condiciones normales de operación el funcionamiento no se vea afectado por un comportamiento estructural inadecuado (deflexiones o vibraciones excesivas, daños locales, etc.) y que además se tenga una seguridad apropiada contra la falla, aún ante situaciones excepcionales de solicitación. El grado de seguridad que debe buscarse contra cada tipo de falla depende de las consecuencias de la misma y del costo que implica incrementar la resistencia de la estructura. Se trata por tanto de un problema de optimización en el que deben balancearse la seguridad y la economía.

Para realizar el proceso de optimización señalado se requiere del conocimiento de las características de las acciones que pueden afectar la estructura en sus etapas de construcción, transporte, instalación y sobre todo durante su vida útil. Dado el alto grado de aleatoriedad de muchas de las acciones, especialmente de las de tipo ambiental, se hace indispensable un tratamiento probabilístico de las mismas, basado en los datos estadísticos disponibles y en modelos probabilísticos del problema físico. Se requiere de la determinación de los efectos que estas acciones inducen en la estructura, considerando en muchos casos su naturaleza dinámica y repetitiva. Finalmente es necesario revisar que las propiedades mecánicas y geométricas de los elementos estructurales sean tales que les permitan resistir dichos efectos sin que se presenten fallas o comportamientos indeseables.

Por la complejidad de los efectos de muchas acciones para las que hay que diseñar las plataformas marinas y por la importancia de estas estructuras, se hace necesario recurrir a procedimientos de análi lisis y diseño altamente refinados. Lo anterior, junto con la necesidad de realizar un gran número de iteraciones para evaluar diversas alternativas de diseño, implica necesariamente el empleo de computadoras para realizar gran parte de las operaciones de análisis y diseño.

El extraordinario auxilio de las computadoras permite liberar al diseñador de la necesidad de la ejecución de cálculos tediosos y repetitivos, para dedicar su atención a los problemas fundamentales que requieren de su experiencia y buen juicio ingenieril. Merece la pena llamar la atención sobre los peligros que presenta el empleo indiscriminado de programas de cómputo cuando no está asociado a una evaluación cuidadosa de los datos y de los resultados y a un entendimiento completo de las operaciones que se realizan en dichos programas.

El diseño de plataformas y de estructuras marinas en general enfrenta al ingeniero con problemas complejos y sobre los cuales queda aún bastante por investigar, que hacen de ésta una de las ramas más interesantes dentro de la ingeniería estructural. Como ejemplos de estos problemas pueden mencionarse los efectos de oleaje, el análisis sísmico incluyendo la presión hidrodinámica y la interacción con el suelo, los problemas de fatiga y concentraciones de esfuerzos, el diseño de pilotes especialmente en sus condiciones de hincado y ante efectos sísmicos, la interacción de los conductores y las tuberías con las plataformas y las condiciones de carga durante las fases de fabricación, transporte y montaje.

TIPOS DE PLATAFORMAS

Diversos tipos de plataformas han sido propuestos, sin embargo la casi totalidad de las construidas hasta la fecha pueden agruparse en dos tipos principales:

a) Plataformas de acero cuya parte característica es una subestructura (jacket) que proporciona la rigidez ante las cargas laterales y sirve de guía para la hinca de pilotes que transmiten las cargas al terreno y soportan una superestructura a base de marcos de acero.

b) Plataformas de concreto (*) que equilibran las cargas laterales por el solo efecto de gravedad y que transmiten las cargas al subsuelo a través de una base muy amplia sobre la que se desplantan las columnas que soportan la superestructura de concreto.

Las características principales de los dos tipos se describen a continuación haciendo una comparación de sus ventajas respectivas.

2.1 Plataformas de acero.

La subestructura es un marco espacial triangulado formado por elementos tubulares alineados generalmente en dos entreejes en una dirección y cuatro en la transversal. Las columnas de esta subestructura son tubos de gran diámetro que sirven de guía para la hinca de los pilotes, también de acero, que sobresalen de la subestructura para recibir la superestructura. De esta forma la subestructura sirve de plantilla, facilitando grandemente la colocación precisa de la plataforma. Esta subestructura es un sistema estructural muy eficiente para resistir las fuerzas de oleaje, ya que ofrece poca área expuesta y la forma circular de sus secciones hace mínimos los empujes de las olas; resulta además muy rígida ante cargas laterales. El sistema transmite directamente las cargas verticales al terreno por medio de los pilotes, los que además absorben los momen-

* de gravedad.

tos de volteo, y cortantes debidos a las cargas horizontales.

La superestructura típica está formada por dos niveles estructurados a base de vigas soldadas de alma llena con el fin de contar con el mayor espacio para la instalación de tuberías y equipo. Las dimensiones típicas de una plataforma de perforación de 12 pozos son de 25 X 40 m. El sistema constructivo implica la fabricación en tierra de la subestructura y la cubierta completas, las que son transportadas en barcazas y montadas en el lugar de instalación.

2.2 Plataformas de concreto.

Estas plataformas están constituidas por una base reticular tipo cajón, muy rígida, normalmente de concreto presforzado y postensado, la cual funciona como sistema de flotación durante el transporte y como cimentación una vez instalada la plataforma. Las columnas son de sección circular hueca con diámetro exterior constante y espesor que se reduce con la altura; también son de concreto presforzado. La cubierta es en general una losa de concreto soportada en una retícula de vigas de concreto presforzado, pero puede ser también de acero, similar a la de las plataformas anteriores.

El procedimiento constructivo implica la construcción en dique seco de la base de cimentación y, en plataformas para profundidades no muy grandes, también de las columnas y de la cubierta; la unidad completa con gran parte del equipo ya instalado es flotada y remolcada al lugar de instalación, donde se hunde mediante lastrado hasta su posición definitiva.

2.3 Evaluación comparativa de las plataformas de acero y de concreto.

Muchos factores influyen en la elección del tipo de plataforma más adecuado, algunos de carácter técnico y otros económico.

Un aspecto básico lo constituye el hecho que las plataformas de acero trasmiten su carga al terreno por medio de pilotes que pueden desplantarse a la profundidad necesaria para evitar los estratos más débiles del subsuelo marino; por el contrario, las plataformas de concreto se desplantan directamente sobre el fondo y su factibilidad puede estar limitada por la excesiva deformabilidad o la escasa resistencia de los estratos superficiales; pueden también presentarse problemas de deslizamiento lateral, de socavación o de licuación de arenas por movimientos telúricos.

Sin embargo, si se efectúa un diseño cuidadoso y se toman precauciones para limitar los movimientos después de la colocación de la plataforma, es factible desplantar plataformas de concreto aún sobre estratos apreciables de suelos muy compresibles.

Aparte del problema anterior pueden anotarse las ventajas siguientes de las plataformas de concreto sobre las de acero:

1. Se construyen con materiales más fácilmente accesibles y con una tecnología menos especializada.
2. Su transporte e instalación requiere menos equipo especializado y es en general más rápido, ya que pueden transportarse casi totalmente equipadas.
3. No requieren de pilotes, lo cual elimina una componente importante del costo de las plataformas de acero.
4. El costo de su mantenimiento es considerablemente inferior, ya que prácticamente no se requiere protección contra la corrosión.

Las ventajas anteriores son suficientes para que las plataformas de concreto compitan muy favorablemente con las de acero, en países como México. Adicionalmente pueden mencionarse las ventajas siguientes:

5. Pueden emplearse para almacenar gran cantidad de hidrocarburos.

6. Son recuperables, ya que terminada su función original, pueden reflotarse con relativa facilidad y llevarse a un sitio diferente.
7. Se reducen los riesgos asociados a incendios y presentan menos problemas de vibraciones.

Como inconvenientes principales de las plataformas de concreto, pueden citarse:

1. Los problemas relativos a asentamientos después de la instalación, los que pueden provocar deformaciones inadmisibles en los conductores.
2. La necesidad de efectuar trabajos de protección contra la socavación de la base.
3. El costo de la preparación del dique seco y de la planta de prefabricación de concreto es muy significativo si sólo se va a construir una plataforma del mismo tipo.
4. El tiempo de fabricación inicial es mayor, aunque se compensa por el menor tiempo de instalación.
5. Por la diversidad de los materiales que intervienen y por el proceso constructivo mismo, el control de calidad es más complicado.

3.- PROCEDIMIENTOS DE CONSTRUCCION, TRANSPORTE E INSTALACION DE PLATAFORMAS.

Plataformas de Acero

a) Subestructura convencional

- Dentro de este tipo de plataformas, existe un número considerable de variantes que dependen de la produndidad del lecho marino y de las funciones de la plataforma, ya que éstas pueden destinarse a perforación, producción, inyección de agua, enlace, medición, vivienda u otra función y dependen también del equipo disponible para su fabricación e instalación y del lugar donde se fabriquen.

El caso más usual de plataformas de acero consta de una subestructura de 8 patas, tal como se describió anteriormente. Dicha plataforma se fabrica armando sobre el suelo los cuatro marcos transversales que la constituyen posteriormente se levantan los dos marcos interiores mediante grúas de orugas para apoyarse sobre vigas de deslizamiento construídas de concreto y revestidas en la superficie por placa de acero.

- Los marcos interiores se contraventean y las grúas con que se levantaron, se utilizan para ayudar a colocar los miembros estructurales que constituirán parte de los marcos longitudinales y del sistema de arriostramiento definitivo de la plataforma.
- Soldados los elementos de la sección central de la plataforma se levantan los marcos transversales extremos con las mismas grúas, y se procede en forma similar a contraventearlos mientras se suelda el resto de los miembros de la subestructura.

- Las dos columnas de la subestructura que descansan sobre las trabes de deslizamiento están provistas de madera en toda su longitud impregnada de grasa en la superficie de contacto para facilitar su deslizamiento sobre la placa de acero, cuando se carga la subestructura en la barcaza.
- Las barcazas cuentan con dos trabes giratorias o de lanzamiento en uno de sus extremos, las cuales quedan alineadas y hacen contacto con las trabes de concreto sobre las que desliza la subestructura. La fuerza necesaria para arrastrar la subestructura hasta su posición definitiva sobre la barcaza la proporcionan uno o dos malacates instalados en el extremo opuesto a las trabes giratorias de la misma barcaza. Una vez que la subestructura se ha cargado, se fija soldando miembros provisionales para evitar accidentes durante su transporte.
- La barcaza es arrastrada por remolcadores hasta su posición de lanzamiento, mismo que se ejecuta utilizando los malacates con que se cargó la subestructura, esta es forzada a deslizar hacia afuera hasta que coincide su centro de gravedad con el de rotación de las trabes giratorias y de allí en adelante, por peso propio la subestructura gira y se desliza simultaneamente hasta abandonar totalmente la barcaza en un minuto aproximadamente.
- En el taller de fabricación se sueldan a la subestructura las orejas de arrastre y las placas de izaje dejando previstos, antes del lanzamiento, pasadores, grilletes y cables de izaje, para girar la estructura a su posición definitiva, una vez dentro del agua. Todas las subestructuras cuentan normalmente con un sistema de inundación que facilita esta operación, ya que las ocho patas están selladas para favorecer la flotación.

La subestructura se apoya en el lecho marino sobre bases de madera o acero localizadas justamente en el extremo inferior de las patas y diseñadas precisamente con ese fin. Los embarcaderos y defensas, así como los protectores de ductos y conductores se sueldan a la subestructura en el patio de fabricación cuando no estorban para cargarla a la barcaza, pues de otra manera, se diseñan para ajustarse una vez colocada la estructura sobre la barcaza o bien, una vez instalada en su posición definitiva.

La subestructura (Jacket) constituye la base de trabajo y guía para los pilotes.

Pilotes

Una vez localizada la subestructura en su posición definitiva, se procede al hincado de los pilotes que la fijan al fondo marino. Los pilotes están constituídos por 3 4 ó más segmentos para facilitar su manejo por una parte, y por otra, por requerimientos de resistencia. El proceso de hincado en una plataforma convencional de 8 patas dura aproximadamente 2 ó 3 semanas.

Superestructura

Los procedimientos de fabricación son también muy variados y dependen prácticamente de los mismos factores que se señalaron en el caso de las subestructuras. Es común que la cubierta principal se fabrique sobre el suelo y se levante mediante grúas para instalarse sobre las columnas de la superestructura.

Una vez terminada su fabricación, la superestructura también se a-rrastra sobre vigas de lanzamiento, similares a las descritas, para cargarse sobre una barcaza.

Plataformas de Concreto

La construcción de las plataformas de concreto varía respecto al tamaño, diseño y contratista. En el caso de bajas profundidades se lleva a cabo en un dique seco, en el cual se construye el cajón de cimentación simultáneamente se construye la cubierta de manera de que pueda deslizarse sobre rieles por encima del cajón de cimentación, llevando instalado el equipo correspondiente.
Movilizada la cubierta sobre el cajón de cimentación, se construyen las columnas, y apoyándose en ellas, se levanta la cubierta para remover los rieles sobre los que deslizó originalmente.
Enseguida, se inunda el dique seco y mediante tanques temporales de flotación si se requiere, se flota la estructura para remolcarse hasta su posición definitiva; una vez allí, se sumerge la cimentación, mediante lastre, hasta apoyarse en el lecho marino, permaneciendo la cubierta flotando sobre la superficie del mar.
La siguiente operación consiste en izar la cubierta apoyándose en las columnas, sobre la superficie del mar y se lastra la cubierta con el fin de producir una precarga adicional sobre el terreno. Cuando se ha logrado la consolidación deseada, se remueve el lastre de la cubierta y se iza ésta hasta su posición definitiva terminando la instalación.

Su transporte se efectúa utilizando varios remolcadores que alcanzan una velocidad de 3 ó 4 nudos.
Si una vez apoyada sobre el lecho marino se observan hundimientos no uniformes, el cajón de cimentación puede nivelarse, lastrándose o bien mediante la inyección de aire comprimido.
Una vez en su sitio definitivo, la inmersión e instalación de una plataforma de concreto es un proceso que se lleva a cabo en unas cuantas horas.

4. CARGAS Y CRITERIOS DE DISEÑO

4.1 Tipos de carga.

Las cargas que se consideran en el análisis de plataformas marinas se clasifican en cinco categorías: cargas muertas, cargas vivas o de funcionamiento, cargas debidas a las deformaciones de las estructuras, cargas ambientales y cargas accidentales.

Dentro de las cargas muertas se considera el peso propio de la estructura, el del balasto o lastre permanente, el del equipo fijo y la presión hidrostática que obra sobre los elementos estructurales, una vez instalada la plataforma "in situ".

La carga viva está constituída por el material almacenado sobre la plataforma, el equipo móvil y los líquidos contenidos en tuberías y recipientes, así como por el peso de grúas y helicópteros; también se incluyen las cargas dinámicas producidas por la vibración de máquinas y los empujes y jalones de barcazas.

Las cargas por deformaciones están asociadas a los efectos del presfuerzo, de la temperatura, de las deformaciones diferidas con el tiempo (creep), de las contracciones y los hundimientos diferenciales en la estructura.

Las cargas ambientales son las debidas a viento, oleaje, corrientes marinas, empuje de hielo, nieve, sismos y, en general, a acciones ambientales similares.

Finalmente, se consideran como cargas accidentales, las debidas a colisiones de barcazas, a explosiones, a fuego, a caída de objetos, y a aquellas cargas debidas a situaciones extraordinarias.

Las intensidades y características de las cargas de diseño se fijan con base en las normas emitidas por organismos especializados; entre las más empleadas se encuentran las del American Petroleum Institute (API) y Det Norske Veritas (DNV).

Hay que notar que para algunos miembros no resultan críticas las condiciones de carga de la plataforma en operación, sino las que se presentan en las diferentes etapas de las fabricación, transporte e instalación; hay que considerar por tanto también las cargas que se presentan debido a estas maniobras y a las condiciones ambientales que puedan ocurrir en ese lapso.

Las cargas muertas se obtienen en forma directa a partir de las dimensiones y propiedades de los materiales. Las cargas vivas se basan en las especificaciones de los fabricantes de equipo e instalaciones y en el análisis de las diferentes situaciones de operación. La parte más delicada es la determinación de las cargas ambientales. Los reglamentos citados recomiendan que las estructuras se diseñen para poder soportar fuerzas excepcionales que se deban a fenómenos que tengan un período de recurrencia de 100 años, o sea del orden de cuatro veces la vida esperada de la plataforma; sin embargo dejan libertad al propietario de fijar otros períodos de recurrencia, si esto se justifica con base en estudios costo-beneficio.

Entre las cargas ambientales suelen resultar más críticas las debidas al efecto combinado del oleaje, el viento y las corrientes que se presentan durante grandes tormentas. Los datos de diseño a este respecto deben basarse en un estudio realizado para el sitio en cuestión por especialistas en oceanografía y meteorología, quienes a partir de la información estadística disponible para el sitio y para zonas de condiciones meteorológicas semejantes, y basados en modelos matemáticos de los problemas físicos, realizan análisis probabilísticos para determinar los parámetros de diseño para el período de recurrencia prescrito; proporcionando así altura, longitud y período de la ola máxima, velocidades de viento sostenido y de ráfaga y velocidades de las corrientes para diferentes profundidades. Para su uso en la revisión de problemas de fatiga deben indicar también el número de veces que se exceden anualmente determinadas alturas de olas.

El diseño por efecto de oleaje se realiza normalmente idealizando los efectos dinámicos de la ola por empujes estáticos equivalentes, aplicando normalmente la fórmula de Morison asociada a la suposición que la ola es del tipo de Stokes de quinto orden; este análisis resulta muy laborioso por tener que considerar diferentes direcciones y posiciones de la ola a fin de determinar las condiciones más críticas para cada elemento.

Los efectos sísmicos pueden regir el diseño en algunas regiones y especialmente para las estructuras de concreto debido a su mayor masa. Nuevamente se requiere de un estudio de microrregionalización sísmica por parte de un especialista, que debe tomar en cuenta las características geotectónicas de la región, la información estadística acerca de los sismos generados en las zonas de influencia y de las intensidades sísmicas registradas en el sitio; para con ello realizar un estudio probabilístico que le permita estimar las intensidades de diseño. De particular importancia resulta para las plataformas considerar la amplificación que los movimientos sísmicos pueden presentar debido a los estratos superficiales muy compresibles que existan.

Los efectos sísmicos en la estructura se determinan a través de un análisis modal o de uno paso a paso. Resulta indispensable en estos análisis considerar la interacción de la estructura con el subsuelo y con el agua. Se requiere además considerar adecuadamente, los amortiguamientos y ductilidades que pueden esperarse.

4.2 Combinaciones de cargas.

La diversidad de cargas que hay que considerar en el diseño da lugar a que se tenga que revisar un gran número de posibles combinaciones críticas. La tabla 1 presenta las combinaciones para una situación típica.

Desde luego, las cargas muertas deben intervenir en todas las combinaciones de carga con su intensidad máxima. Las cargas vivas intervienen también en todas las combinaciones, sin embargo cuando se superponen a las cargas ambientales máximas se emplea un valor reducido de las cargas vivas dado que en estas condiciones se interrumpen las operaciones de las plataformas.

Para las cargas ambientales se considera por una parte una tormenta de operación correspondiente a un período de recurrencia pequeño, para la cual no debe alterarse el funcionamiento de la plataforma y que debe superponerse a las cargas vivas máximas. Para la tormenta de diseño (de 100 años) hay que revisar que no ocurra el colapso de la estructura.

El efecto de sismo no se superpone a los de viento y oleaje y para ello los reglamentos también recomiendan considerar un sismo de operación y uno excepcional con los mismos criterios que se siguen para las tormentas.

.3 Criterios de diseño y factores de seguridad.

Las plataformas de acero suelen diseñarse con criterios de esfuerzos admisibles. El API adopta las recomendaciones generales para diseño de estructuras metálicas del American Institute for Steel Construction (AISC). En estos criterios de diseño el análisis se realiza suponiendo un comportamiento elástico de la estructura, se determinan los esfuerzos que se inducen en las secciones y se comparan con valores admisibles que son una fracción del esfuerzo máximo resistente del material. Para combinaciones de cargas que incluyen efectos ambientales excepcionales se admite un incremento de 33% en los esfuerzos admisibles.

Al diseñar los elementos estructurales contra colapso hidrostático, se considera un factor de seguridad adicional de 1.4

y el mismo factor se aplica para revisar las piezas por cortante por penetración.

Respecto a la penetración de pilotes se considera un factor de seguridad de 2, cuando se trata de cargas de operación; y de 1.5, cuando las cargas se deben a tormenta de 100 años o a sismo; además, los pilotes se revisan por tensión suponiendo una carga viva reducida como factor de seguridad adicional.

Para las plataformas de concreto se emplean generalmente criterios de diseño por resistencia última o de estados límite; el American Concrete Institute (ACI) ha editado recientemente unas recomendaciones para plataformas marinas de concreto en las cuales se establecen una gama de factores de carga por los que hay que afectar las diferentes cargas según las combinaciones de que se trate. Los efectos de estas cargas factorizadas deben compararse con la resistencia última de la estructura, afectada ésta de un factor de seguridad relativo a las incertidumbres de la resistencia.

La tendencia actual es hacia la adopción de métodos de diseño por resistencia o estados límite con preferencia a los de esfuerzos admisibles.

PROBLEMAS ESPECIALES DE DISEÑO

Además del proceso general de análisis y diseño descrito anteriormente existen problemas particulares que requieren una revisión detallada y que son peculiares del sistema estructural adoptado para la plataforma. A continuación se tratan someramente los principales problemas de este tipo en las plataformas de acero y de concreto.

5.1 Plataformas de acero.

a) Diseño de los pilotes. Los pilotes quedan libres dentro de la funda de la subestructura y solo hacen contacto con ella en algunos puntos de contraventeo en los que se colocan placas de relleno. Ameritan atención especial la revisión de esfuerzos durante el hincado y la interacción suelo-pilotes-estructura ante las diferentes combinaciones de carga.

El costo de los pilotes y de su hincado representa una porción muy significativa del costo de la plataforma, por tanto se requiere un análisis cuidadoso para determinar los espesores óptimos de los tubos y el tipo de martillo más apropiado para su hincado. A través de la aplicación de la ecuación de onda unidimensional y a partir del conocimiento de las propiedades del subsuelo se pueden determinar los esfuerzos y la velocidad de penetración del pilote considerado como un voladizo sujeto a su propio peso y a la acción dinámica del martillo.

Para el estudio del estado de esfuerzos en el pilote ante las cargas verticales y horizontales se suele sustituir el suelo por sistemas de resortes que proporcionan condiciones equivalentes de restricciones. Las restricciones a la rotación que imponen las patas de la subestructura inducen momentos flexionantes importantes en los pilotes al considerar el efecto de las cargas laterales. Las zonas críticas suelen ser a nivel del fondo marino y unos 10 m bajo el mismo.

b) Conexiones. Las intersecciones entre los elementos tubulares que forman la subestructura plantean problemas geométricos para los cortes de las piezas y problemas de concentración de esfuerzos en las soldaduras. Las concentraciones de esfuerzos se estudian a través de análisis por elementos finitos con malla muy cerrada; para los principales tipos de conexión el comportamiento se ha comprobado a través de ensayes a escala natural. El análisis indica que las concentraciones en las conexiones producen esfuerzos máximos en la soldadura que son varias veces superiores al esfuerzo nominal en el elemento. Especialmente críticas son las conexiones entre elementos diagonales pequeños y columnas de gran diámetro. En las conexiones es necesario revisar la posibilidad de falla por punzonamiento de un tubo dentro del otro. Para ello el API proporciona recomendaciones detalladas.

c) Fatiga. El oleaje induce en la subestructura ciclos de carga que pueden llevar a la falla los elementos por fatiga a niveles de esfuerzos inferiores a los admisibles. Estos problemas son particularmente críticos en las conexiones y se vuelven más importantes a medida que aumenta la profundidad de la plataforma. El análisis de estos efectos es muy laborioso ya que requiere conocer la historia de esfuerzos en los diferentes elementos durante la vida útil de la plataforma. Es necesario determinar el número de repeticiones que pueden esperarse para diversos niveles de esfuerzos y conocer para los materiales empleados las curvas que relacionan los diferentes niveles de esfuerzos con el número de ciclos que el material es capaz de resistir antes de la falla. La superposición de los efectos de los diferentes niveles de esfuerzos suele hacerse con relaciones de interacción sencillas como la de Palmgren-Miner.

d) Cargas durante la construcción. Durante su transporte en la barcaza la subestructura puede experimentar fuerzas importantes debidas al oleaje las cuales pueden ser críticas en los elementos que en la posición definitiva no van a estar sujetos a efectos importantes por este concepto. Se requiere un análisis de esta condición de carga considerando un oleaje de intensidad no muy elevada, ya que para este transporte se busca siempre aprovechar periodos en que pueden esperarse buenas condiciones meteorológicas.

Al botar la estructura desde la barcaza se presentan las condiciones más severas cuando su posición es la que corresponde al inicio de la rotación de las vigas giratorias y en el instante del lanzamiento; puede ser crítico el punzonamiento de las conexiones que se apoyan directamente sobre la barcaza. Es necesario revisar la capacidad de flotación de la subestructura dentro del agua y las fuerzas que se presentan en el viaje de la misma para colocarla en su posición final.

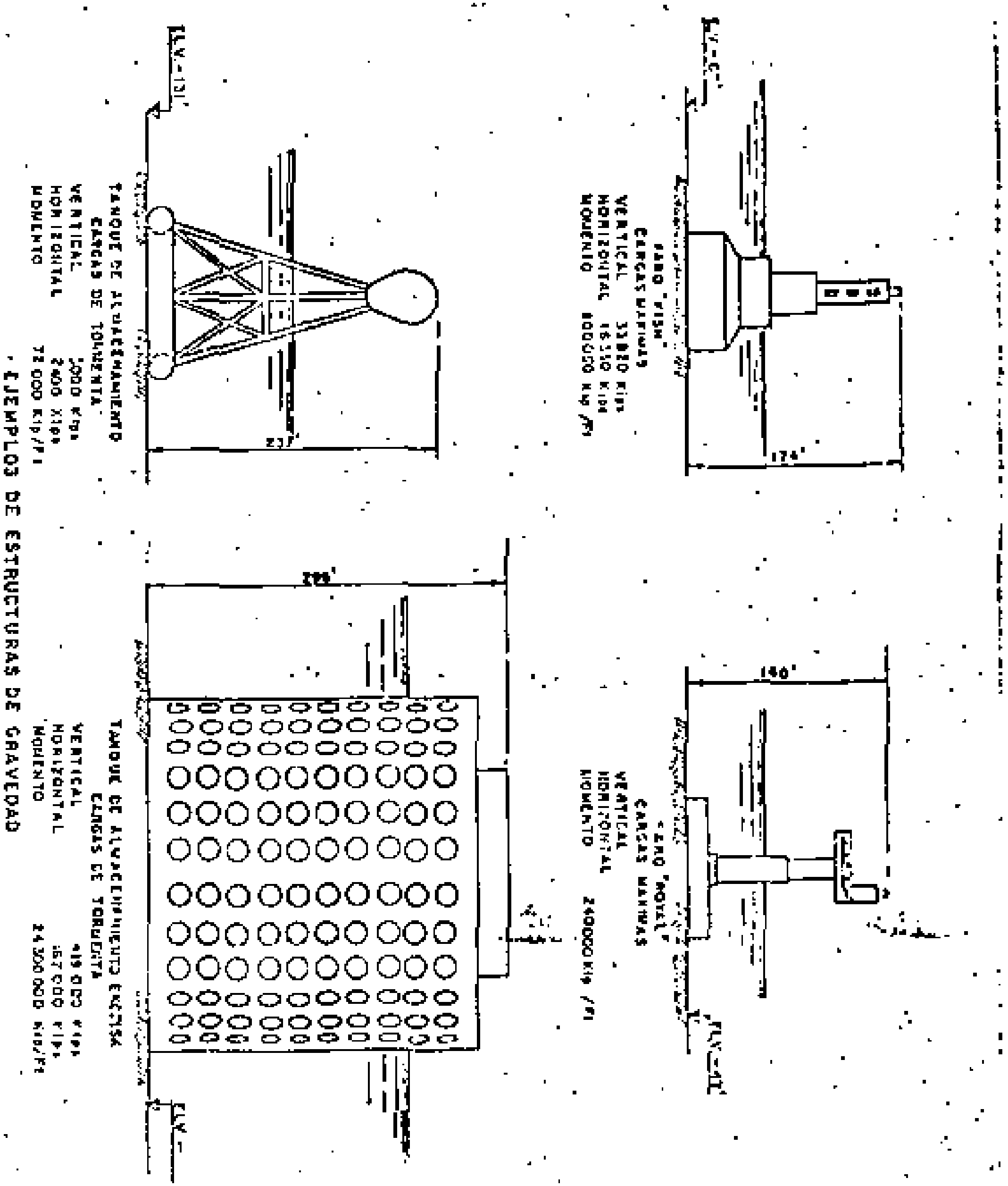
5.2 Estructuras de concreto.

a) Análisis de asentamientos. Probablemente el aspecto más crítico de estas plataformas de gravedad apoyadas en suelos compresibles es el cálculo de los asentamientos y de su variación con el tiempo y el diseño de las políticas de carga y precarga para minimizar los asentamientos que puedan ocurrir una vez instalada la plataforma y los conductores. La calidad y cantidad de los resultados de los sondeos y de las pruebas de laboratorio que permiten determinar con buena aproximación las propiedades del subsuelo marino, es un factor fundamental para estas determinaciones.

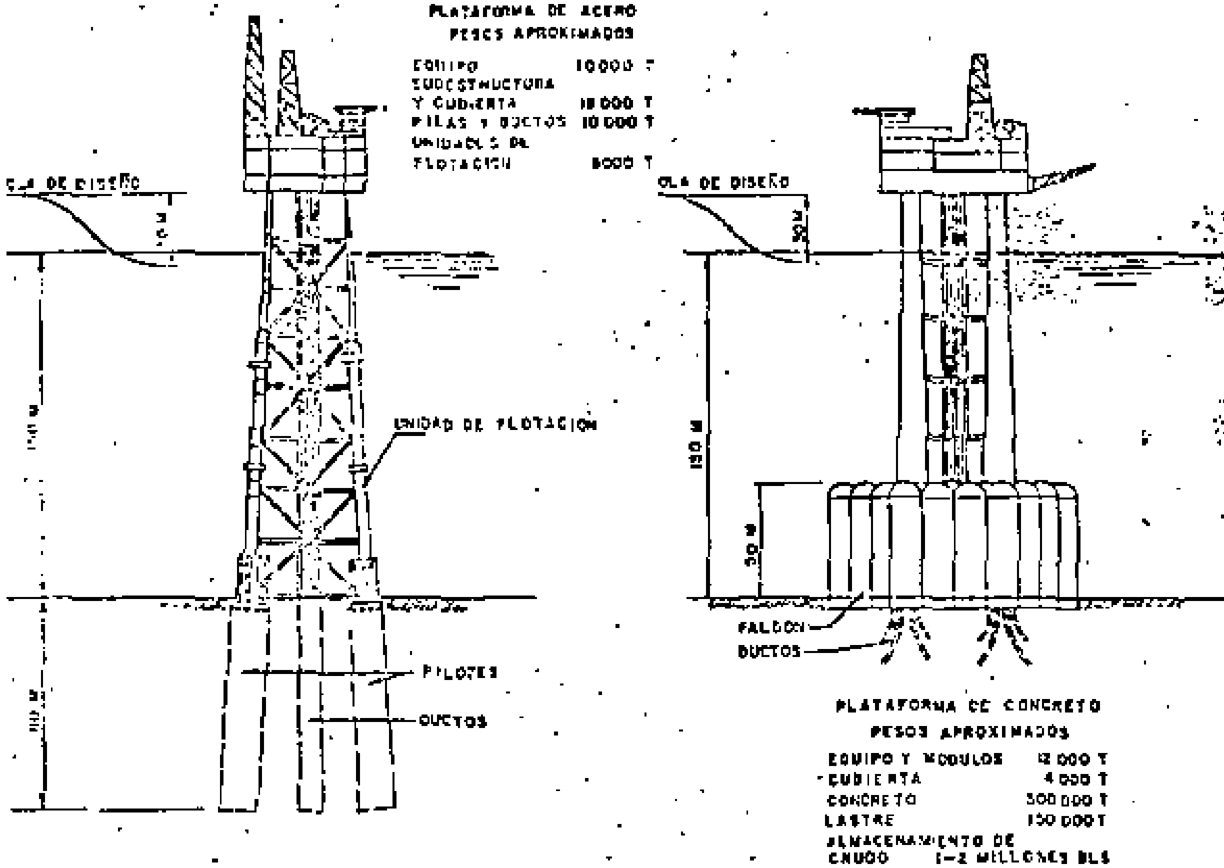
b) Flotación. Las plataformas de este tipo deben funcionar como embarcaciones durante su transporte; esto requiere un análisis cuidadoso de las condiciones de flotación y de los efectos hidrodinámicos durante el transporte. Se requiere para ello una estimación muy precisa de los pesos de las diferentes partes y un análisis detallado del lastre necesario para el hundimiento a su posición definitiva.

c) Elección de materiales. La durabilidad del concreto y la protección que éste ofrezca contra la corrosión del refuerzo dependen de una cuidadosa selección de los agregados, de la dosificación y de la colocación. Estos aspectos deben especificarse en detalle. Las recomendaciones ACI para plataformas dan criterios generales sobre este concepto.

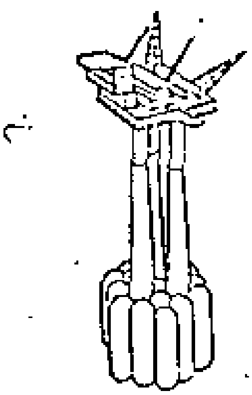
d) Detalles para ductilidad en zonas sísmicas. Debido a que las fuerzas sísmicas pueden ser críticas en este tipo de plataformas se requiere un especial cuidado en el refuerzo y en su detalle especialmente en las conexiones para asegurar un comportamiento dúctil antes de la falla.



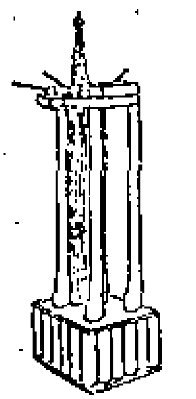
EJEMPLOS DE ESTRUCTURAS DE GRAVEDAD



COMPARACION DE PLATAFORMAS DE ACERO Y CONCRETO
(Caso típico Mar del Norte).

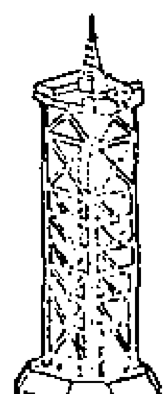


PLATAFORMA DE GRAVEDAD
"CONDEEP"

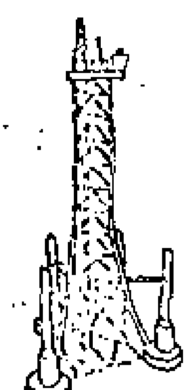


PLATAFORMA DE GRAVEDAD
"SEA TANK CO."

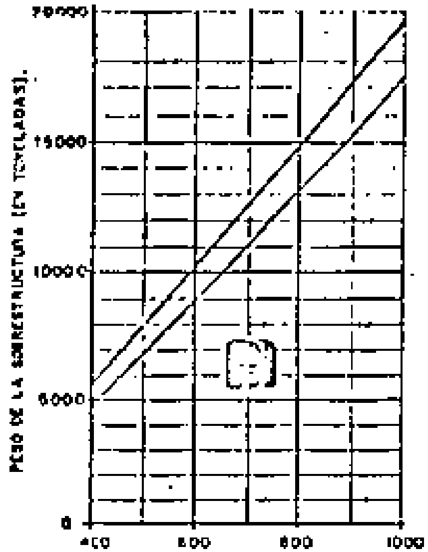
PLATAFORMAS DE GRAVEDAD



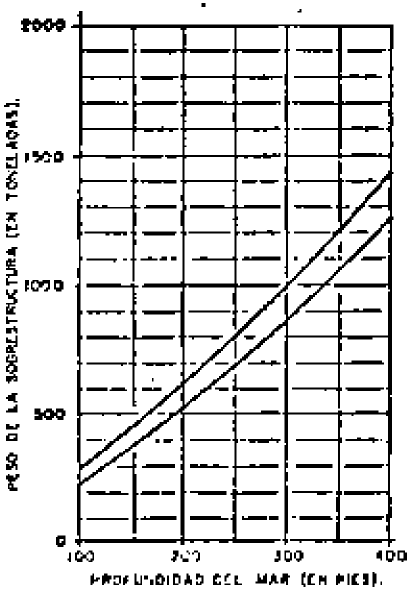
PLATAFORMA DE GRAVEDAD
DISEÑO HIBRIDO



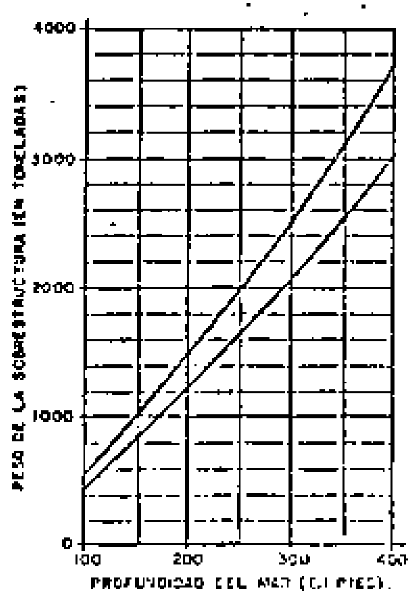
PLATAFORMA DE GRAVEDAD
DISEÑO EN ACERO



PLATAFORMAS DE PERFORACION Y PRODUCCION DE 2 TORRES DE PERFORACION

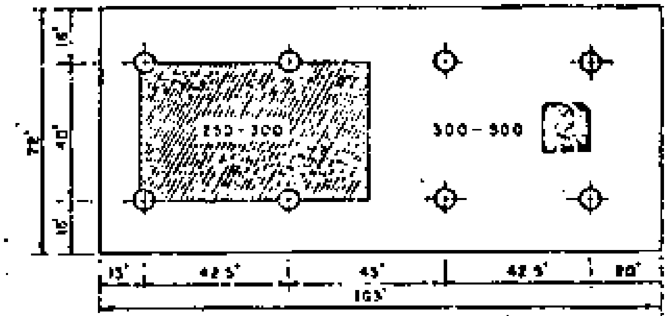


PLATAFORMAS DE PERFORACION Y PRODUCCION LIGERA.



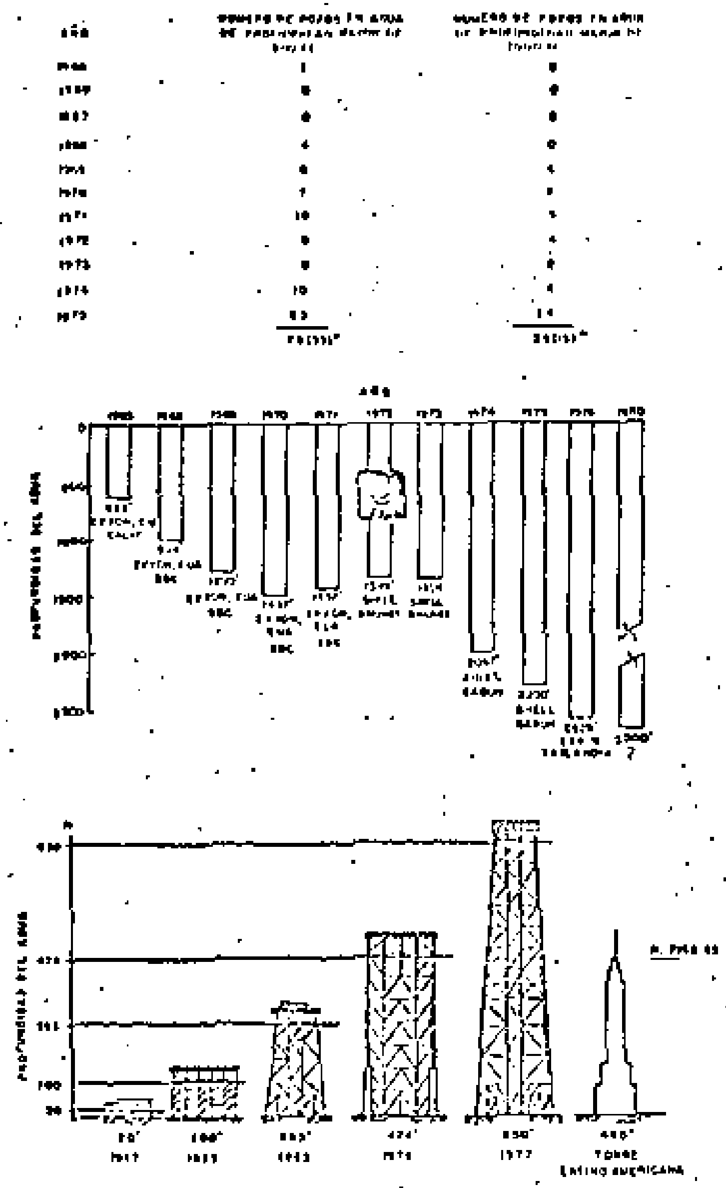
PLATAFORMAS DE PERFORACION Y PRODUCCION DE UNA SOLA TORRE

CARGAS VIVAS UNIFORMES (EN Lb/pie²) EN PLATAFORMAS DE PRODUCCION.



PLATAFORMAS DE CONCRETO (DATOS GENERALES)

SITIO	PROFUNDIDAD DEL MAR (EN METROS)	ALTURA MAXIMA DE OLA (EN METROS)	FUERZA HORIZONTAL MAXIMA (EN TONS)	MOMENTO DE VOLTEO MAXIMO (EN TONS-m)
EKOFISK	70	24	78 600	3.33×10^6
FRIGG CDP1	100	29	69 130	3.33×10^6
FRIGG MP2	97	29	69 150	3.13×10^6
NINIAN CENTRAL	139	31.2	102 900	4.10×10^6



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DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

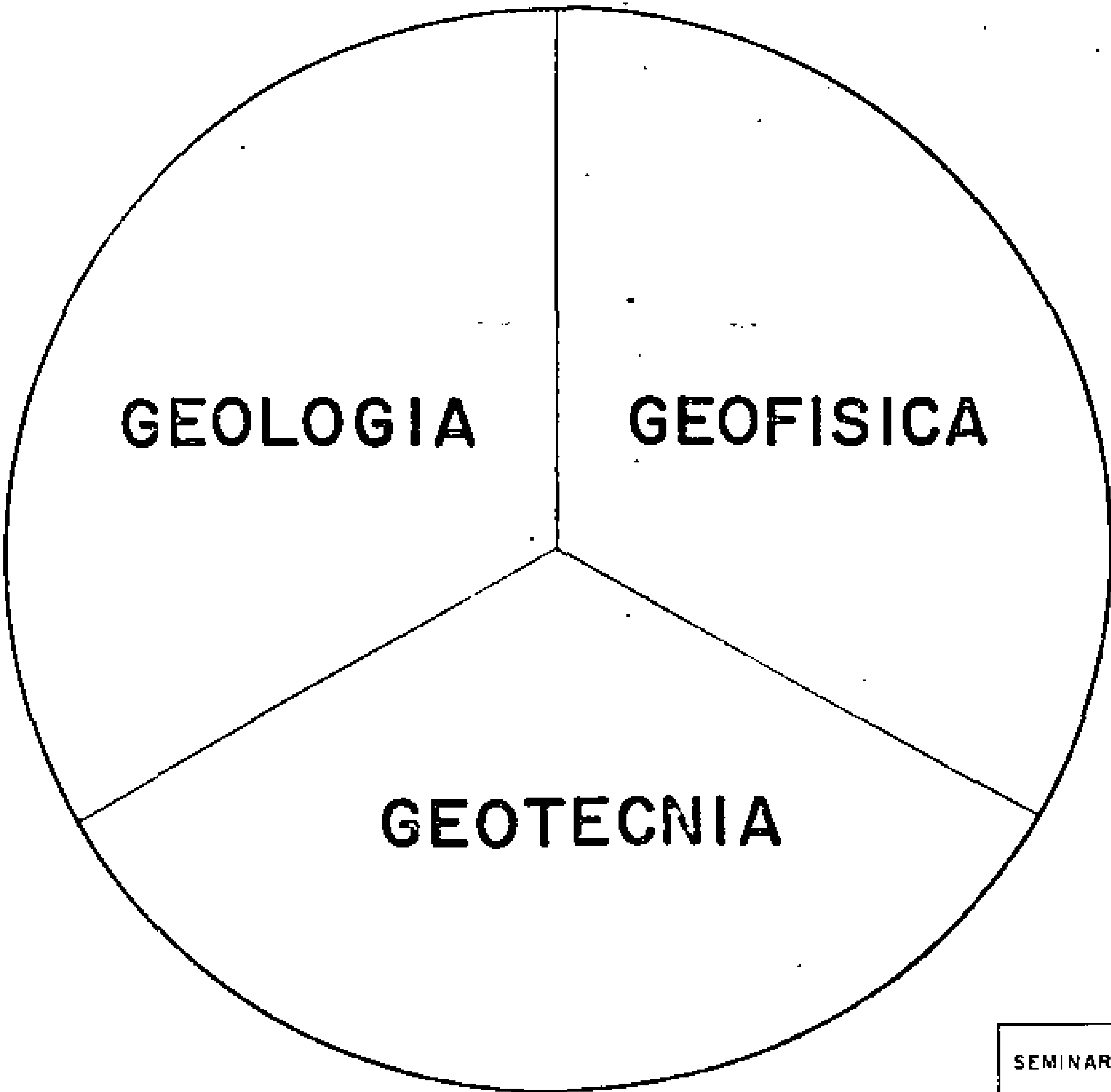
SEMINARIO INTERNACIONAL DE INGENIERIA OCEANICA

AMBIENTE MARINO

ING. J. J. HANELL

MARZO 1982

Palacio de Minería Calle de Tacuba 5 primer piso Deleg. Cuauhtemoc 06000 México, D.F. Tel.: 521-40-20 Apdo. Postal M-2285


GEOLOGIA
GEOFISICA
GEOTECNIA

Exploración del Fondo Marino

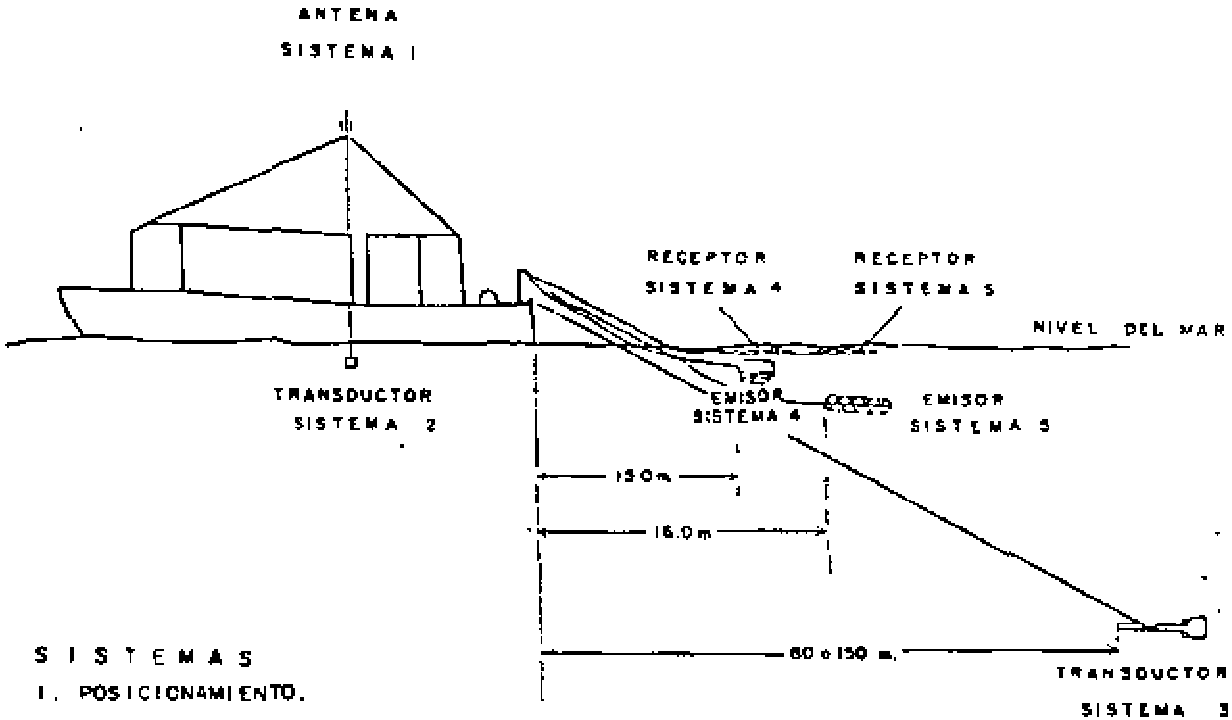
- Métodos Geofísicos Marinos de alta resolución Acústica.
- Métodos de exploración con obtención de Muestras.
- Métodos de exploración mediante mediciones in situ.

Geofísica Marina de Alta Resolución Acústica.

Objetivos a Investigar y Equipos Empleados

OBJETIVO	EQUIPO
Tirante de Agua	Ecosondas (echo sounder)
Fondo Marino	a) Sonar de barrido lateral (side-scan sonar)
	b) Cámaras de Televisión y ó Fotografías
Subsuelo	Perfilador (boomer / sparkers): Somero; Profundo

FIGURA Nº 28

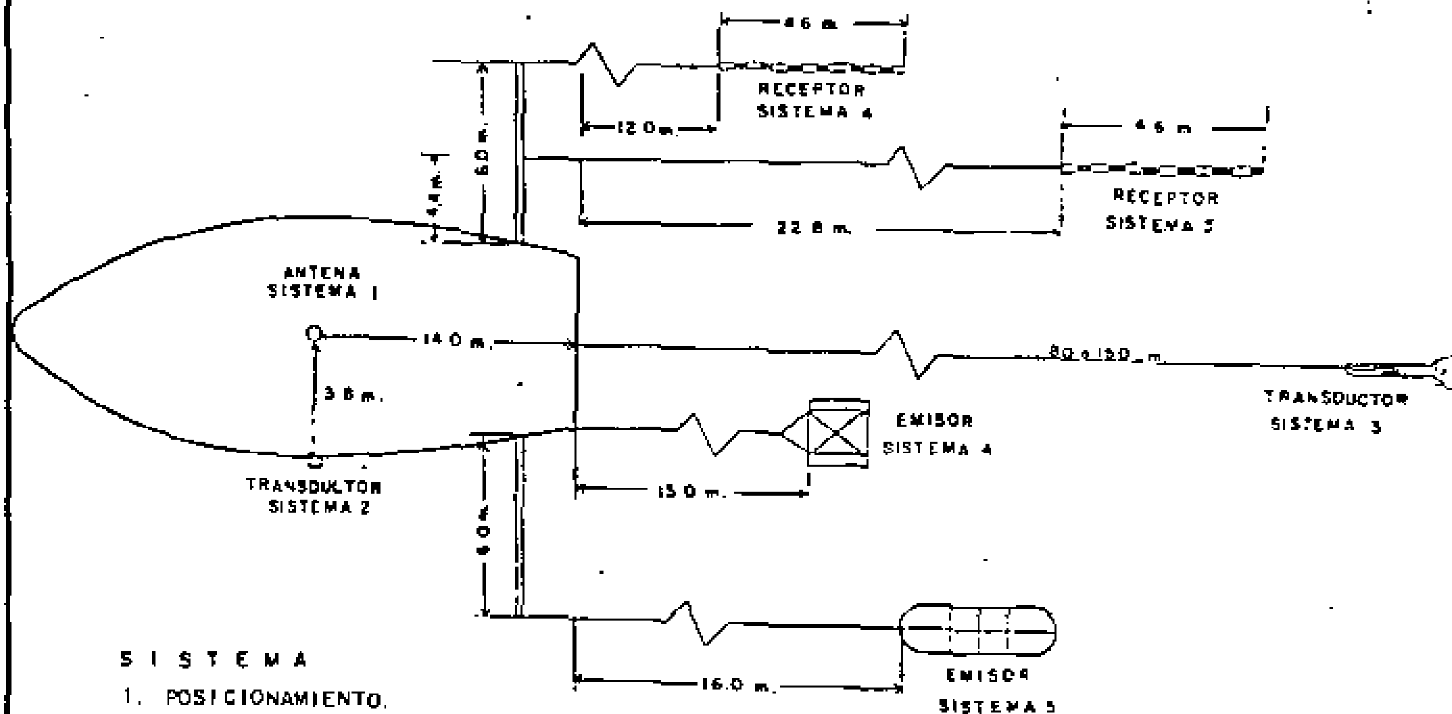


SISTEMAS

1. POSICIONAMIENTO.
2. MEDIDOR DE PROFUNDIDAD.
3. SONAR DE BARRIDO LATERAL.
4. PERFILADOR SOMERO
5. PERFILADOR PROFUNDO.

DISPOSICION ESQUEMATICA EN PERFIL DE LOS EQUIPOS EN EL AGUA.

FIGURA Nº 2 A

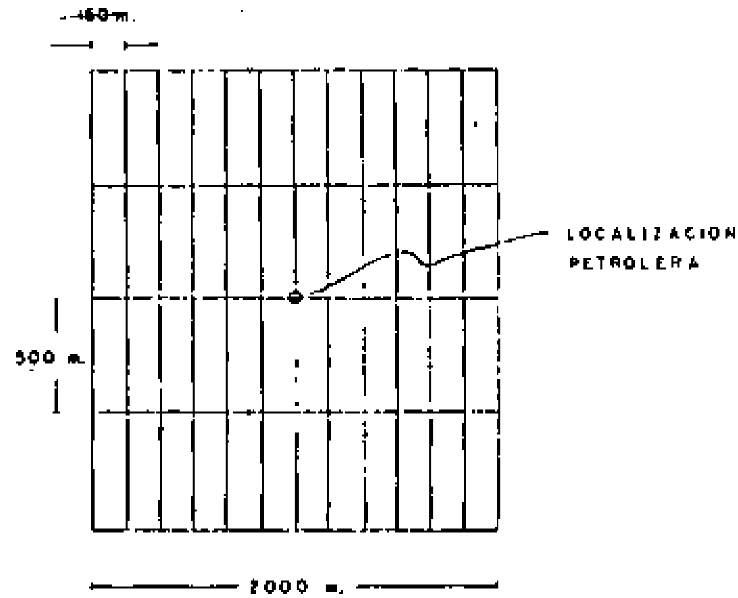


S I S T E M A

1. POSICIONAMIENTO.
2. MEDIDOR DE PROFUNDIDAD.
3. SONAR DE BARRIDO LATERAL
4. PERFILADOR SOMERO
5. PERFILADOR PROFUNDO.

DISPOSICION ESQUEMATICA EN PLANTA DE LOS EQUIPOS EN EL AGUA.

(6)



RETICULA DE LEVANTAMIENTO DE UNA LOCALIZACION PETROLERA

FIGURA Nº 1

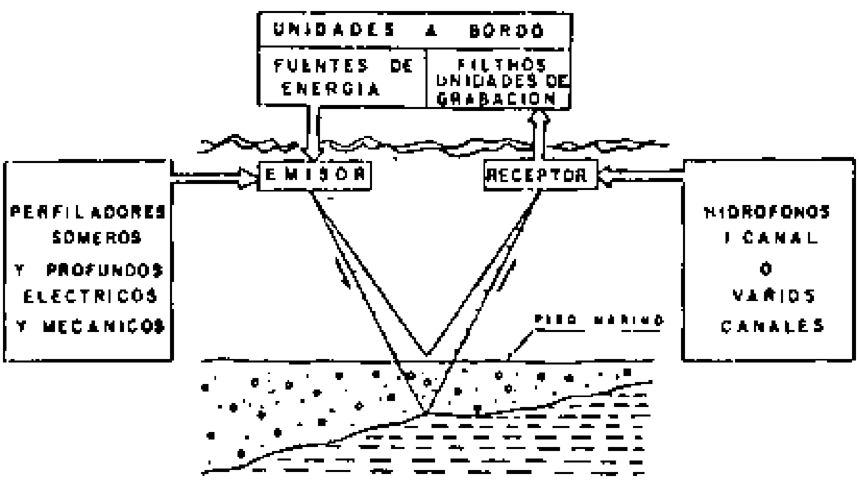
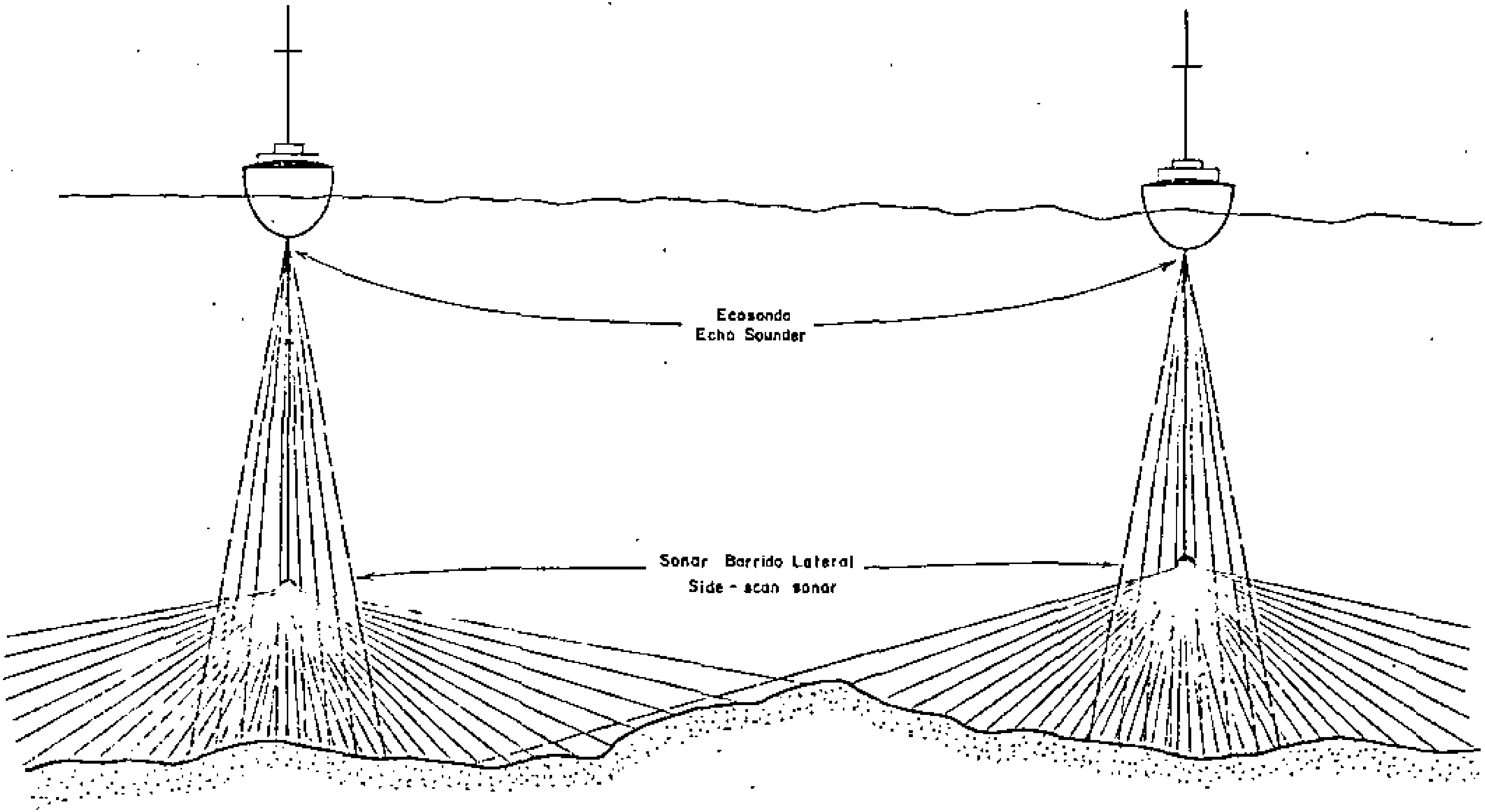


DIAGRAMA ILUSTRATIVO DEL FUNCIONAMIENTO DE LOS PERFILADORES

FIGURA Nº 1 A



SEMINARIO INTERNACIONAL
DE
INGENIERIA OCEANICA

Accidentes Geológicos:

- Variaciones rápidas en la Estratigrafía
- Cauces antiguos abiertos o rellenos (paleocanales)
- Arrecifes de Coral
- Roca
- Fallas y Plegamientos
- Gas
  - biogenético
  - petrogenético
- Material y ó Estructuras Artificiales
- Inestabilidad del fondo Marino

Estratigrafia y Geologia Superficiales en la Bahia de Campeche

Neurauter W.T. (1), Antoine W.J. (1), Roden R.A. (1),
Gutierrez E.C. (2), Fahlquist A.D. (3).

1. Decca Survey Systems, Inc., Bryan Texas, EUA.
2. Proyectos Marinos, S. C., México
3. Texas A & M University, College Station, Texas, EUA.

SYNOPSIS A joint venture including Proyectos Marinos S.C., Decca Survey Systems, Inc., and Fugro Gulf, Inc. produced a high resolution seismic-coring-borehole program to delineate the shallow geology over portions of the Bay of Campeche. The primary objective was to outline the areal distribution of potential geohazards to drilling, production, and pipeline activities and to define the shallow stratigraphy of the region. The delineated anomalies considered to represent potential geohazards in the Bay of Campeche include reefs, faults, shallow gas (low shear strength zones), relatively deep pressured gas zones, and channel cut and fill features. By defining the locations and the depths to these anomalies, it is possible to plan the most effective pipeline routes and to select the safest sites for the emplacement of bottom structures for drilling and production operations.

INTRODUCCION

En el articulo "El Arte y Ciencia de la Interpretación de Datos Sísmicos de Alta Resolución" presentado ya en este Simposio se mostró un número de estructuras geológicas delineadas acústicamente que pudieran representar riesgos potenciales para las operaciones en alta mar. Varias de esas estructuras se observaron en el área de la Bahía de Campeche. Proyectos Marinos realizó aproximadamente 3,000 km de exploración geofísica de alta resolución en la Bahía de Campeche (Fig. 1), contando para ello con la asesoría técnica de Decca Survey Systems y Fugro Gulf, Inc. Esta exploración sísmica sirvió de base para la evaluación de los sitios propuestos para la instalación de plataformas así como en las rutas para el tendido de tuberías.

Estos datos complementados con numerosos perfiles estratigráficos superficiales, permiten comprender la composición y estructura de la geología marina somera de la parte oriental de la Bahía de Campeche. A pesar de que una discusión detallada de la geología de esta zona está fuera del alcance de este escrito, se presenta una descripción general de la estratigrafía de la región explorada y se enumeran e ilustran las diferentes estructuras anómalas delineadas por esos datos sísmicos.

EQUIPO

Los sistemas sísmicos de alta resolución incluyeron: 1) un perfilador profundo de baja frecuencia (60-186 Hz) de 3 filamentos y 4.2 kilojoules de potencia con un hidrófono receptor de canal sencillo, 2) un perfilador profundo de alta frecuencia (300 - 1,000 Hz) de 40 filamentos y 400 joules de potencia, también con un hidrófono receptor de canal sencillo, 3) un perfilador somero Edo-Western de 7 KHZ, 4) un sonar de barrido lateral que proporciona un trazo de registro doble para la información del sonar y para la del perfilador somero, y 5) una ecosonda de precisión Atlas.

GEOLOGIA REGIONAL

La localización del área de estudio que nos ocupa se ubica en la margen oriental de la Bahía de Campeche sobre la Plataforma Continental de Yucatán.

La Bahía de Campeche es la extensión marina correspondiente a la porción ístmica de sedimentación del sureste de México, formada por una región de domos salinos que son una prolongación mar adentro de la Cuenca Salina del Golfo de México. Esta zona salina emerge hacia el noreste con los domos y lomeríos de Sigsbee en la Cuenca profunda de la parte central del Golfo de México.

La historia geológica de la Bahía de Campeche y de la Región Istmica del Golfo corresponde generalmente a la última orogénesis del Paleozoico, en la cual se formó un sistema montañoso hacia la parte sur de esta área. Esta formación montañosa fue seguida por deslizamientos hacia la cuenca debidos a un sistema de fallas originados a lo largo de la costa. Durante el Jurásico se depositaron sales y otras evaporitas a través de toda la zona ístmica del Golfo. En el Cretácico se depositaron tanto carbonatos como clásticos térreos, predominando los primeros hacia el este. La sección terciaria es muy potente, con más de 9 km de sedimentación, y está formada principalmente por sedimentos areno-arcillosos (Murray, 1961).

La región bajo estudio está situada en la zona de transición entre los rellenos clásticos de la Bahía de Campeche y los carbonatos del Banco de Campeche. A pesar de que los depósitos térreos dominan la litología superficial del área, existen sin embargo arrecifes de coral y montículos de carbonatos en muchas zonas.

ESTRATIGRAFIA SUPERFICIAL

La mayoría de los sitios explorados se localizan hacia la costa frente a Ciudad del Carmen, y en ellos los sedimentos superficiales están formados por aproximadamente 10 m de arenas densas que sobreyacen a suelos arcillosos de consistencia firme a dura. Mar adentro, los sedimentos superficiales se componen de arcillas con alto contenido de agua y cuya consistencia varía de muy blanda a blanda. Hacia el norte de la localización Finic 1 (Fig. 1) estas arcillas subreyacen a una arena carbonatada fina a media, de compacidad densa. Este estrato de arena es regional según se pudo identificar en numerosos sondeos geotécnicos, además de haberse correlacionado con los datos de la exploración sísmica a lo largo de toda el área. El espesor de las arcillas superficiales es de más de 25 m en la parte surpeste y se reduce hacia el norte y este, siendo de unos 6 m en la localización denominada Balam 1 y desapareciendo en Cech 1 dando lugar a que la arena subsiguiente aparezca superficialmente hacia el borde de la Plataforma Continental.

La presencia de este estrato de arena es importante en la evaluación de los sitios para el emplazamiento de las plataformas móviles de perforación, debido a que los remanentes de las estructuras formadas por los arrecifes de coral esparcidos aleatoriamente por toda esta área se desarrollaron a lo largo de esta superficie. También se encontró que en algunos sondeos existen más de 10 m de material carbonatado cementado, asociado aparentemente con los crecimientos de coral. Hacia el norte del área Ku y Cech 1, donde las arcillas reducen su espesor, se encontraron remanentes de estos arrecifes de coral expuestos sobre el lecho marino.

A continuación de la arena carbonatada se tiene una secuencia alternada de unidades de arena y arcilla. Los horizontes reflectores sísmicos a través de las áreas de Abkatún, Cantarell y Ku han sido correlacionados con las fronteras superiores de los estratos de arena observados en los perfiles estratigráficos de los sondeos geotécnicos. La estratigrafía de toda esta región se ilustra sísmicamente en las figs. 2 y 3.

ARRECIFES DE CORAL

Como se mencionó anteriormente, los remanentes de arrecifes de coral han sido observados a lo largo del estrato de arena carbonatada a través de la mayor parte del área explorada. Con base en la distribución limitada de los datos disponibles, se puede decir que la extensión general en planta de estas formaciones está delimitada de la latitud correspondiente al área de Abkatún hacia el norte.

Estas estructuras están distribuidas aleatoriamente y forman bancos y montículos de 3 a 5 m de elevación sobre el estrato de arena (Fig. 2). En la parte norte del área Ku, donde las arcillas superficiales reducen su espesor a menos de 3 m, estos montículos de coral llegan a sobresalir algunos metros del lecho marino (Fig. 4). Se piensa que el origen de estos montículos tuvo lugar en un medio ambiente similar al que presenta hoy en día el Banco de Campeche, interrumpiéndose posteriormente estos crecimientos de coral debido al flujo de arcillas provenientes del sur.

La instalación de estructuras en el lecho marino se ve entorpecida por la existencia de estos montículos de coral debido a:

1) presentan un nivel desigual para la colocación de las plataformas autoelevables móviles de perforación.

2) durante la cimentación de plataformas se hace difícil el hincado de pilotes a través de estos materiales cementados.

3) hacen que el zanjado para la instalación de tuberías sea difícil y costoso, además de ocasionar pandeos en las mismas cuando éstas se llegan a encontrar suspendidas entre montículos sucesivos de coral.

FALLAS Y PLEGAMIENTOS

Existen fallas y plegamientos en varias de las áreas exploradas en donde se nota que las fallas con movimientos más recientes son las más cercanas al borde de la Plataforma Continental (Fig. 5), en las que se han desplazado los estratos que se encuentran inmediatamente bajo el lecho marino, y en algunos casos desplazando incluso al mismo lecho marino. Los desplazamientos totales ocasionados por estas fallas varían en un rango que va de menos de 1 m hasta aproximadamente unos 50 m. Hacia la costa las fallas se hacen progresivamente más profundas (Fig. 6) afectando pocas veces a los estratos que se localizan en los primeros 75 m superficiales. El rumbo de estas fallas es generalmente con dirección norte-sur.

Los sistemas de fallas son la respuesta a movimientos salinos profundos. Los domos salinos parecen encontrarse a menor profundidad conforme se aleja uno de la costa. Este fenómeno es también observado en las costas de Texas y Louisiana.

La mayoría de las fallas no han presentado actividad alguna en épocas geológicas recientes. Sin embargo, la posible migración de gas a lo largo de los planos de falla presentan un riesgo potencial para los trabajos de perforación debido a la posibilidad de:

1) encontrar inesperadamente a poca profundidad gas presurizado.

2) que exista una acumulación de gas justo bajo una plataforma, ocasionando la inestabilidad del subsuelo debido a su rápido decremento de la resistencia al cortante del mismo.

ACUMULACIONES DE GAS

La detección de gas en los sedimentos superficiales se logra de varias maneras en los registros sísmicos, esto es: por zonas sin señal acústica, por reflexiones débiles o confusas, por reflexiones distorsionadas o caóticas, por zonas de reflexión muy marcada o por arqueo en los horizontes reflectores.

El origen de la mayor parte del gas encontrado en el área bajo estudio es aparentemente de tipo biogénico, formado por la descomposición de materia orgánica dentro de los sedimentos superficiales no consolidados. El gas de origen petrogénico se localiza a mayor profundidad y es difícil de determinar con los medios actuales de muestreo geoquímico, a pesar de que en muchos casos estas acumulaciones profundas se detectan por salidas de gas delineadas acústicamente.

Las mayores zonas con gas superficial se localizan en el interior de la Plataforma Continental entre los 19°00' y la línea costera, es decir, a lo largo de unos 45 km; en la sección costera de una de las rutas de tubería exploradas se obtuvieron reflexiones típicas de acumulaciones de gas justo bajo el lecho marino (Fig. 7); esto se debió al contenido de gas en forma de burbujas en el agua intersticial de las arcillas, las cuales actuaron como puntos individuales que dieron origen a nuevas señales sísmicas.

La presencia de gas en los sedimentos marinos hace que disminuya la resistencia de los suelos, de donde se debe tener especial cuidado de este efecto dado que puede darse lugar a una posible condición de inestabilidad en el lecho marino. Se tiene también que si se encuentra gas en grandes cantidades esto puede deformar la señal sísmica a tal grado que las estructuras más profundas pueden llegar a ser cubiertas en los registros, aún empleando fuentes de sonido de menor frecuencia.

Las acumulaciones más profundas de gas, evidentes por sus reflexiones de alta amplitud o "zonas de reflexión muy marcada" (Fig. 5), pueden indicar la existencia de presiones mayores a la hidrostática. Por lo tanto, se debe tener cuidado de la profundidad a la que se encuentran estas anomalías y si se efectúa perforación alguna a través de ellas se deberán preparar lodos de perforación con mayor peso al normalmente empleado. También se tiene que los horizontes reflectores deformados o discontinuos a lo largo de los planos de falla pueden indicar la posibilidad de que exista migración de gas a lo largo de los planos de falla.

Otro tipo de indicios de gas se observa en forma de reflexiones sísmicas a partir de burbujas de gas filtrándose a través de la columna de agua asociadas con depresiones y montículos en el lecho marino que varían de 1 a 3 m de desnivel. Excepcionalmente se encontró una gran depresión (Fig. 8) cerca del área de Abkatún, cuyas dimensiones fueron 135 m de diámetro y más de 24 m de profundidad. Aparentemente esta anomalía parece haber sido ocasionada por un escape violento de gas (blow-out), similar a los encontrados en las costas de Texas y Louisiana. Puede también observarse en el registro del sonar de barrido lateral de ese mismo sitio que existe un escape de gas asociado a este cráter.

PALEOCANALES Y SUPERFICIES EROSIONADAS

En el Pleistoceno ocurrió un descenso del nivel del mar que dió lugar a que corrientes y ríos atravesaran las zonas que hoy en día constituyen las plataformas continentales de la Tierra. Los remanentes de estos antiguos sistemas de drenaje son estructuras de corte y relleno denominados paleocanales (Fig. 9) con una topografía sub-superficial irregular. La distribución de estos paleocanales está principalmente concentrada en las áreas que se localizan al oriente y al sur de la zona explorada.

La mayoría de los paleocanales son de pequeñas dimensiones y cuyo trazo es difícil de seguir a lo largo de grandes distancias. A 15 km mar adentro de la Terminal Marítima de Dos Bocas se observó una de estas superficies erosionales (Fig. 10), la cual fue formada probablemente por oleaje o por el paso de corrientes, siendo necesarios estudios más amplios para confirmar esta interpretación.

La variación de la resistencia al cortante de los depósitos de relleno de estas estructuras así como los horizontes de cantos y gravas encontrados dentro de las mismas presentan problemas técnicos para la instalación de las plataformas y durante las operaciones de perforación. De igual manera, la topografía sub-superficial de tales formaciones pueden significar una desventaja para las instalaciones de las plataformas autoelevables móviles de perforación.

RESUMEN Y CONCLUSIONES

La interpretación de los datos sísmicos de alta resolución, conjuntamente con la información obtenida de los sondeos geotécnicos con recuperación de muestras, ha probado ser un método eficiente para la evaluación de las condiciones geológicas superficiales en la Bahía de Campeche.

El propósito de la investigación descrita en este estudio ha sido el delinear los principales riesgos geológicos potenciales para la instalación de plataformas de perforación y producción así como también para el tendido de tuberías, además de mostrar a grandes rasgos las zonas anómalas que pueden ser detectadas por este tipo de exploración indirecta, lo que constituye una ayuda auxiliar durante las etapas de diseño de las estructuras y de planeación de las operaciones de perforación para mayor seguridad y economía.

Específicamente, en este escrito se ha definido la estratigrafía superficial y las estructuras geológicas del área explorada.

1) ESTRATIGRAFIA

Existe una arena superficial a partir del lecho marino en la región playera y llega a ser cubierta por una capa de arcilla de consistencia muy blanda a blanda cuyo espesor aumenta mar adentro. El contenido de carbonatos del estrato de arena aumenta conforme se aleja de la costa y es subyacido a su vez por una secuencia alternada de estratos de arena y arcilla.

2) ARRECIFES DE CORAL

A partir del área de Abkatún hacia el norte prevalecen estructuras formadas por arrecifes de coral ya sea bajo el lecho marino o a través de él, las cuales se encuentran asociadas frecuentemente con áreas localizadas de carbonatos cementados. De la habilidad para delinear estas anomalías se hace posible la efectividad para determinar las rutas más ventajosas para el tendido de tuberías así como las localizaciones más adecuadas en la ubicación de plataformas.

3) FALLAS

En general, las fallas delineadas tienden a ser más profundas hacia la línea de costa, mientras que los desplazamientos ocurridos cerca del borde de la Plataforma Continental llegan a desplazar al lecho marino en algunos sitios. Estas fallas son aparentemente un resultado de movimientos salinos profundos, presentando algunas de ellas desplazamientos significativos.

Es importante considerar que los emplazamientos sobre el lecho marino se ubiquen lejos de las fallas que han dado lugar a grandes desplazamientos y de todos aquellos accidentes geológicos que desplazan a los sedimentos superficiales, ya que deberán esperarse futuros movimientos verticales a lo largo de esos planos de falla.

4) ACUMULACIONES DE GAS

Por los métodos de exploración sísmica es posible delinear las anomalías ocasionadas por acumulaciones de gas, tanto superficiales como relativamente profundas.

La importancia de las acumulaciones someras de gas se debe a la disminución que éstas ocasionan en la resistencia al cortante de los sedimentos, originada por la existencia de burbujas de gas en los espacios de agua intersticial. Las anomalías relacionadas con las acumulaciones profundas de gas pueden presentar zonas a presiones mayores que la hidrostática, particularmente cuando son definidas por reflexiones de alta amplitud y horizontes arqueados. Por ello, en las operaciones de perforación a través de este tipo de anomalías profundas se requieren de lodos de mayor densidad.

5) PALEOCANALES Y SUPERFICIES EROSIONADAS

Estas huellas erosionales ya rellenas y cubiertas se encuentran principalmente en las áreas al este y sur de la zona estudiada. Durante la utilización de plataformas autoelevables móviles de perforación es importante definir las fronteras exactas de los paleocanales, ya que la resistencia al corte del material de relleno de estas estructuras pueden variar dentro de rangos muy amplios.

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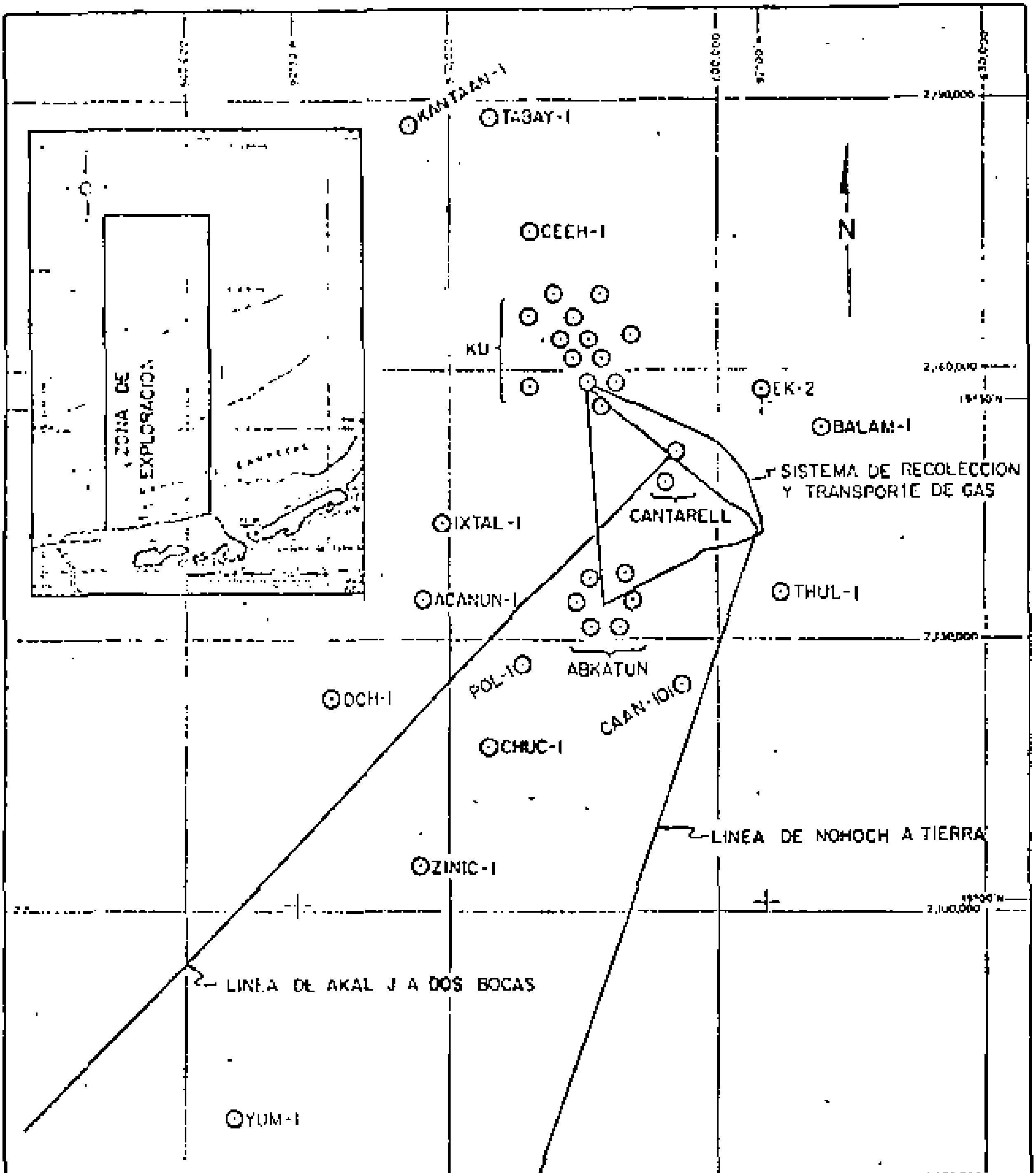

FIGURA 1

AREAS DE EXPLORACION GEOFISICA
BAHIA DE CAMPECHE

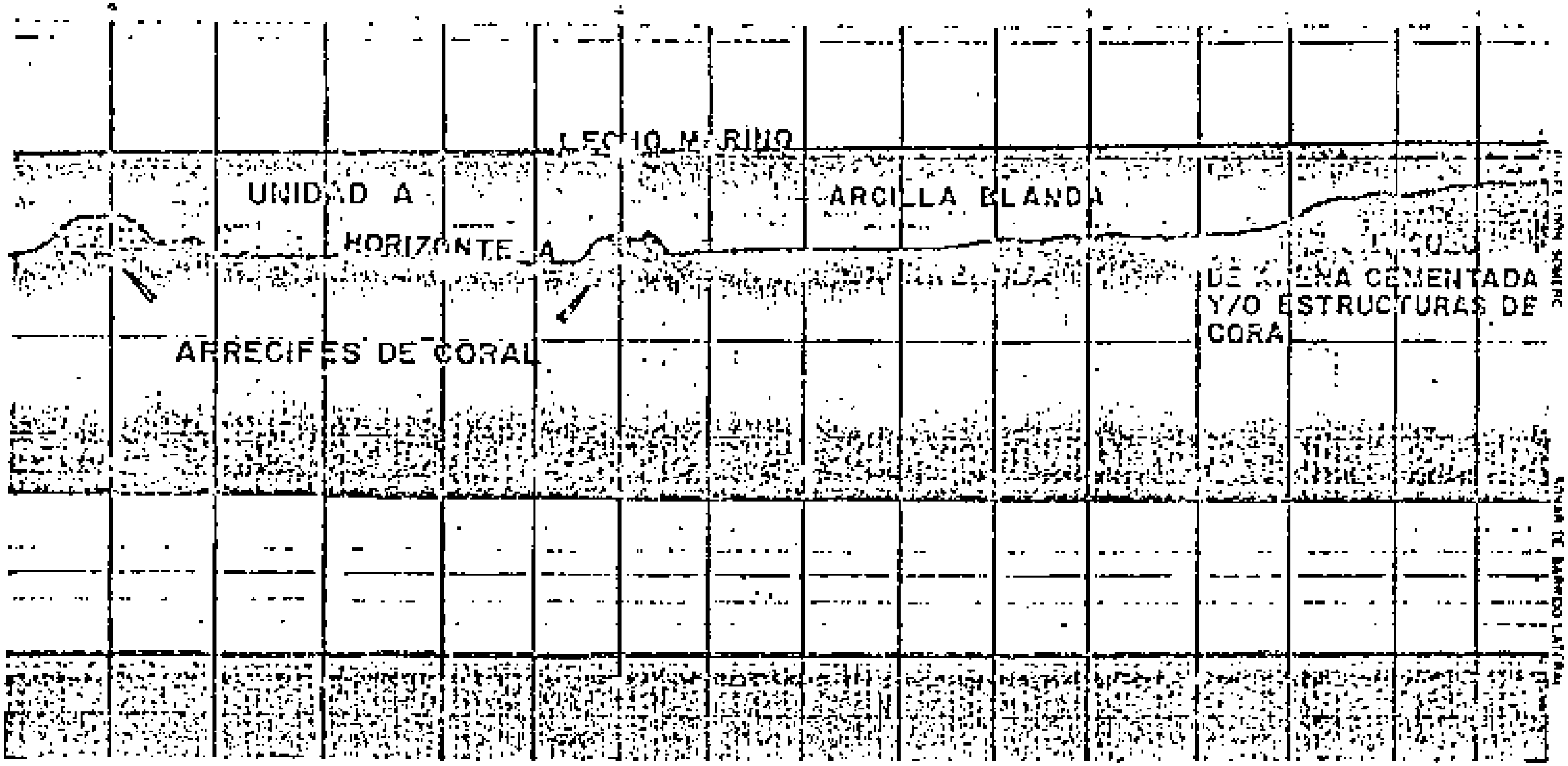

FIGURA 2. REGISTRO DEL PERFILADOR SOMERO MOSTRANDO LAS UNIDADES DE SEDIMENTACION SUPERFICIAL CON FORMACIONES DE CORAL SOBRE EL HORIZONTE DE ARENA.

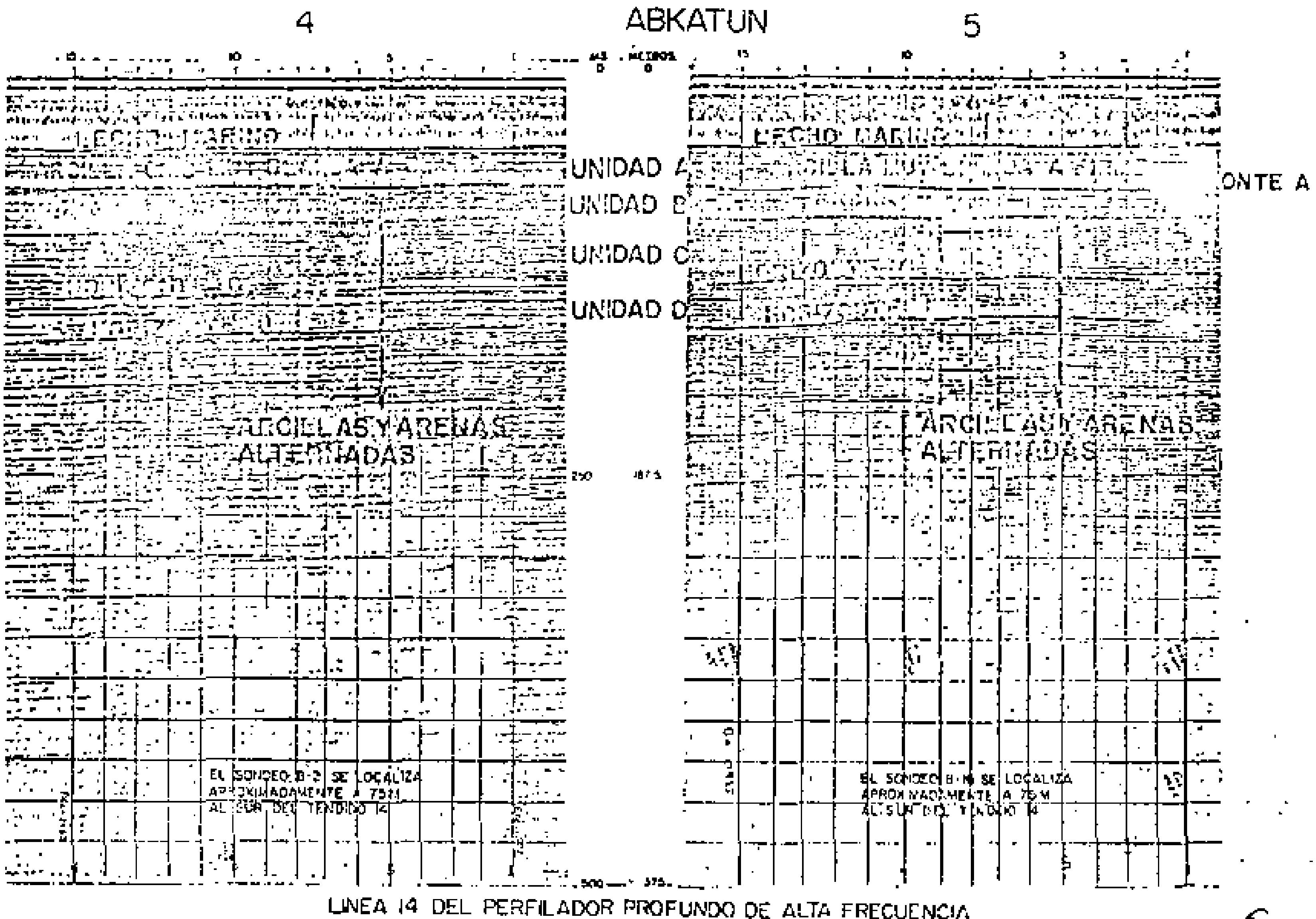

FIGURA 3. REGISTROS DEL PERFILADOR PROFUNDO DE ALTA FRECUENCIA DELINEANDO LA ESTRATIGRAFIA SUPERFICIAL.

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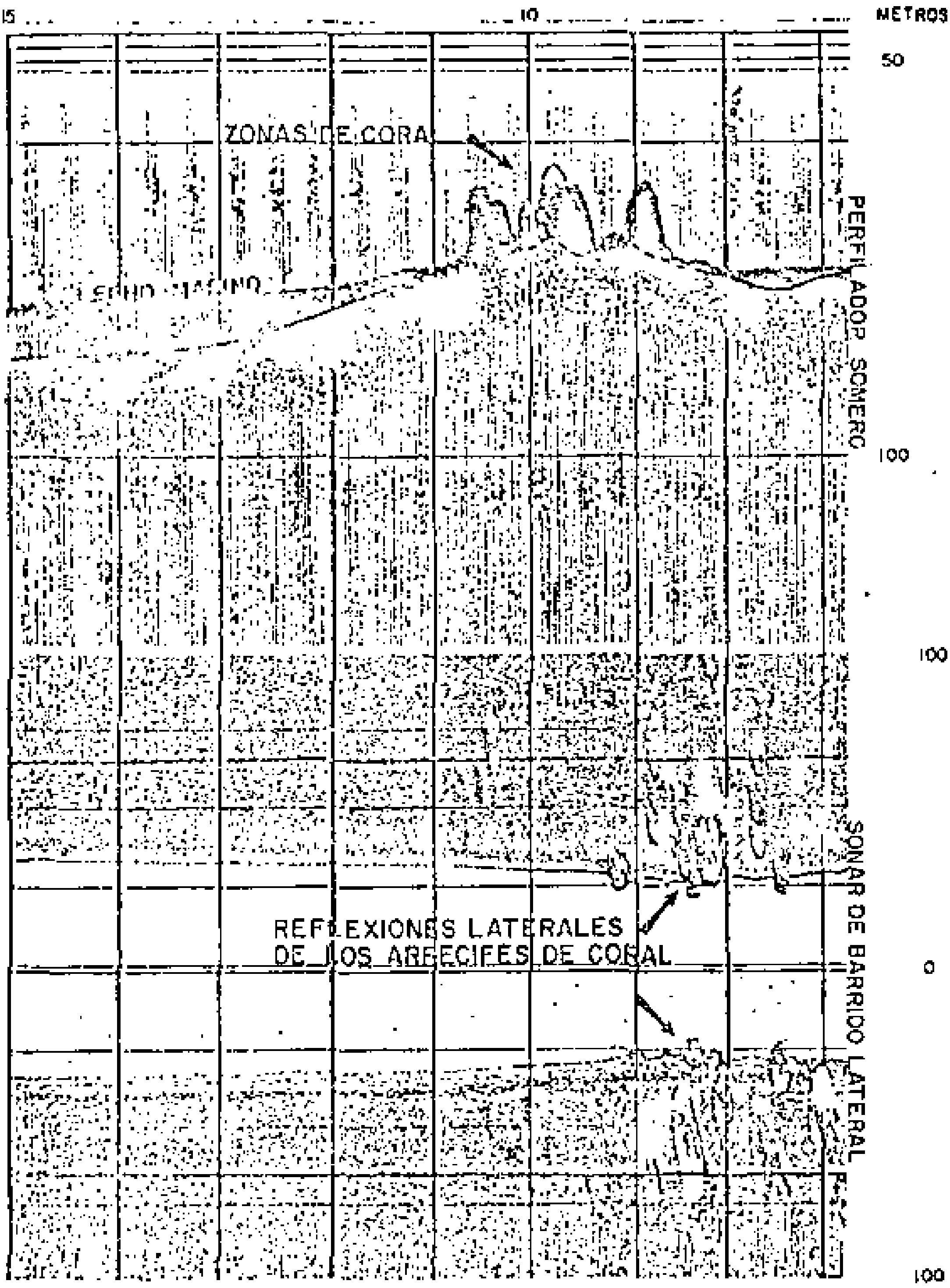


FIGURA 4. MONTICULOS DE CORAL SOBRESALIENDO DEL LECHO MARINO SEGUN MUESTRAN LOS REGISTROS DEL PERFILADOR SOMERO Y DEL SONAR DE BARRIDO LATERAL.

(27)

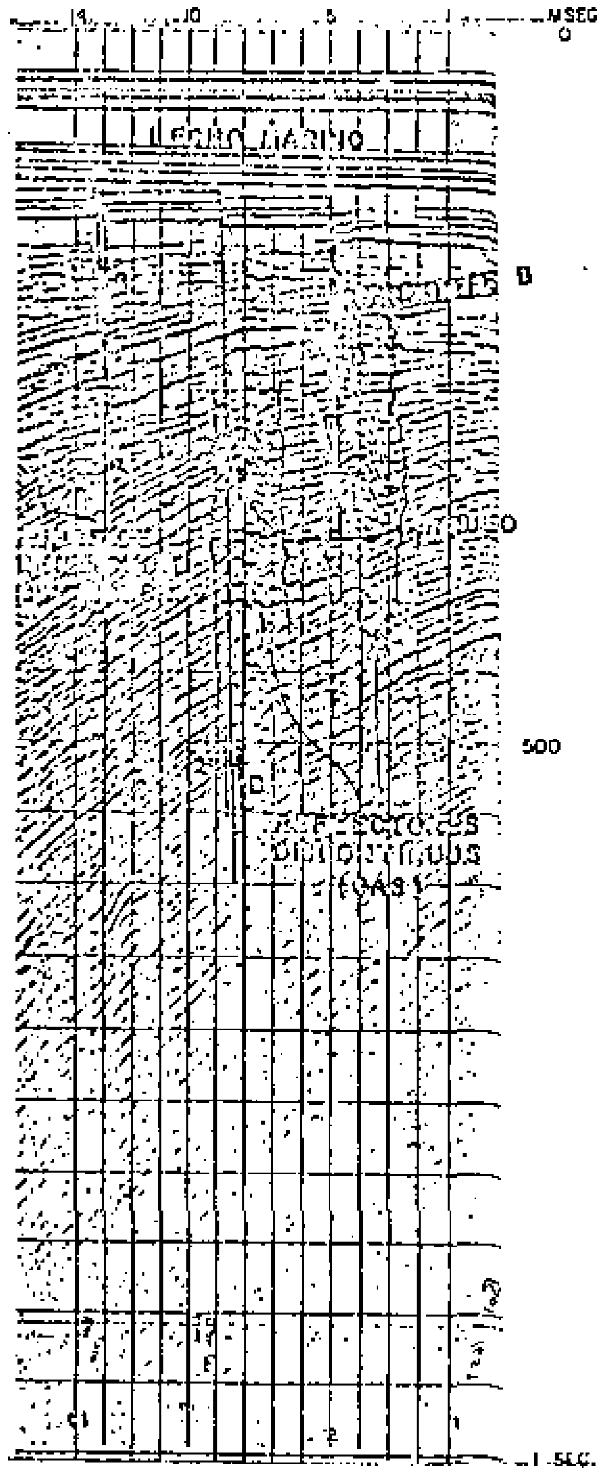


FIGURA 5. SISTEMA DE FALLAS EN LA ORILLA CONTINENTAL CON HORIZONTES REFLECTORES DE ALTA AMPLITUD DETECTADOS POR EL PERFILADOR PROFUNDO DE BAJA FRECUENCIA.

(18)

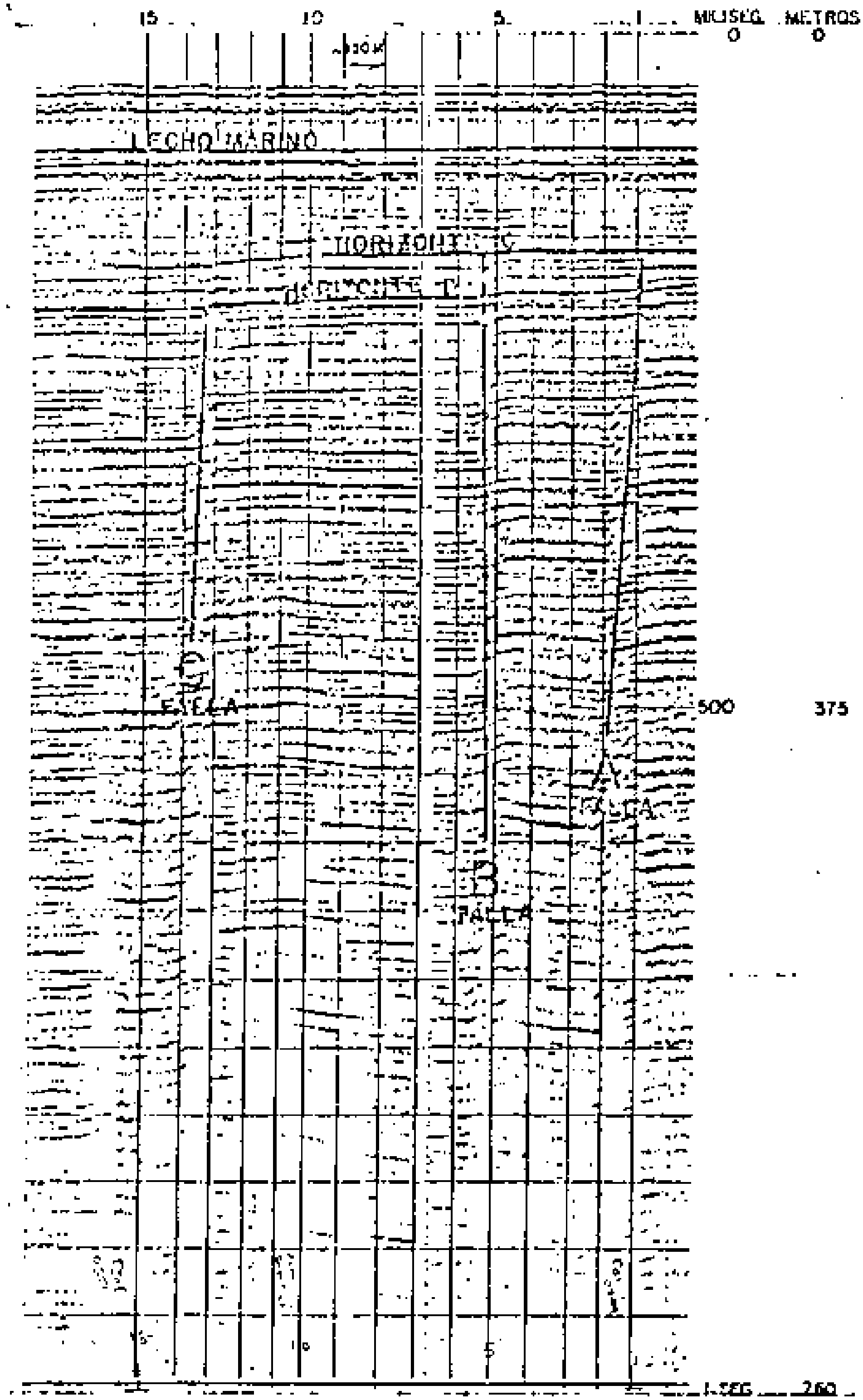


FIGURA 6. REGISTRO DEL PERFILADOR PROFUNDO DE BAJA FRECUENCIA MOSTRANDO UN SISTEMA DE FALLAS PROFUNDAS.

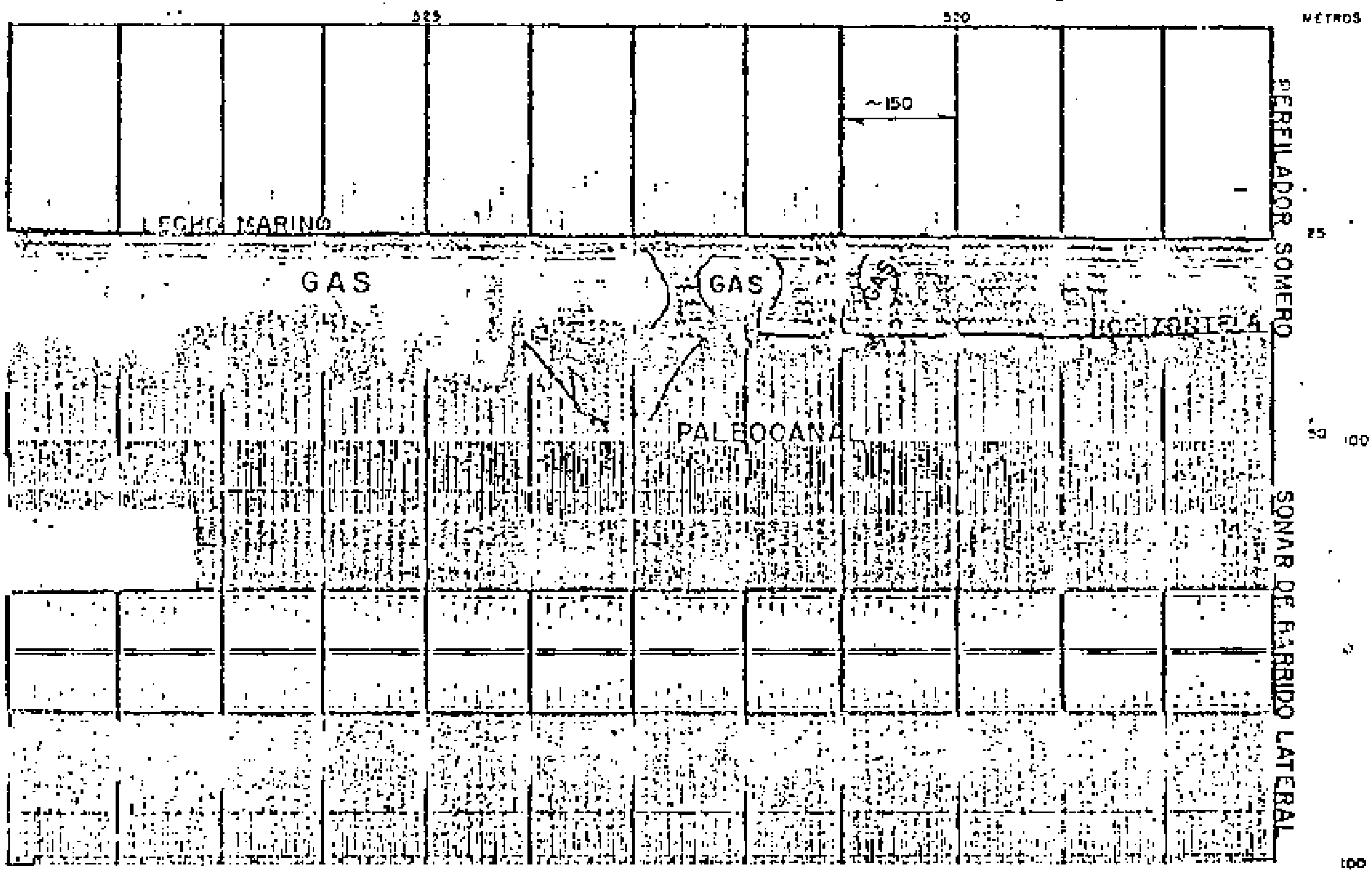
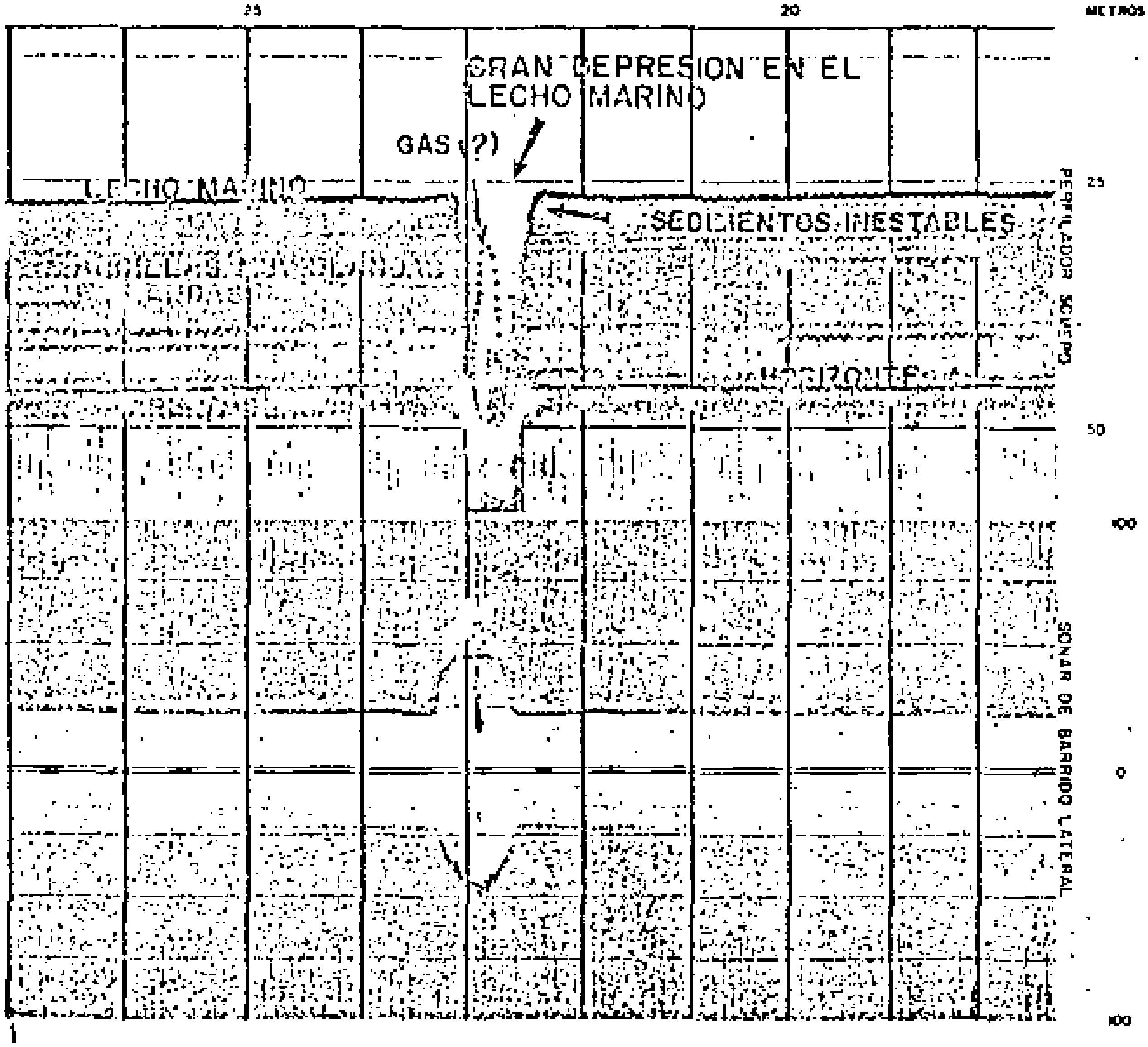


FIGURA 7. ACUMULACIONES SUPERFICIALES DE GAS DETECTADAS POR EL PERFILADOR SOMERO.



LINEA DERECHA DE ABKATUN IA A ABKATUN 5 DEL PERFILADOR SOMERO

FIGURA 8. GRAN DEPRESION EN EL LECHO MARINO ASOCIADA CON ESCAPE DE GAS SEGUN MUESTRAN LOS REGISTROS DEL PERFILADOR SOMERO Y DEL SONAR DE BARRIDO LATERAL.

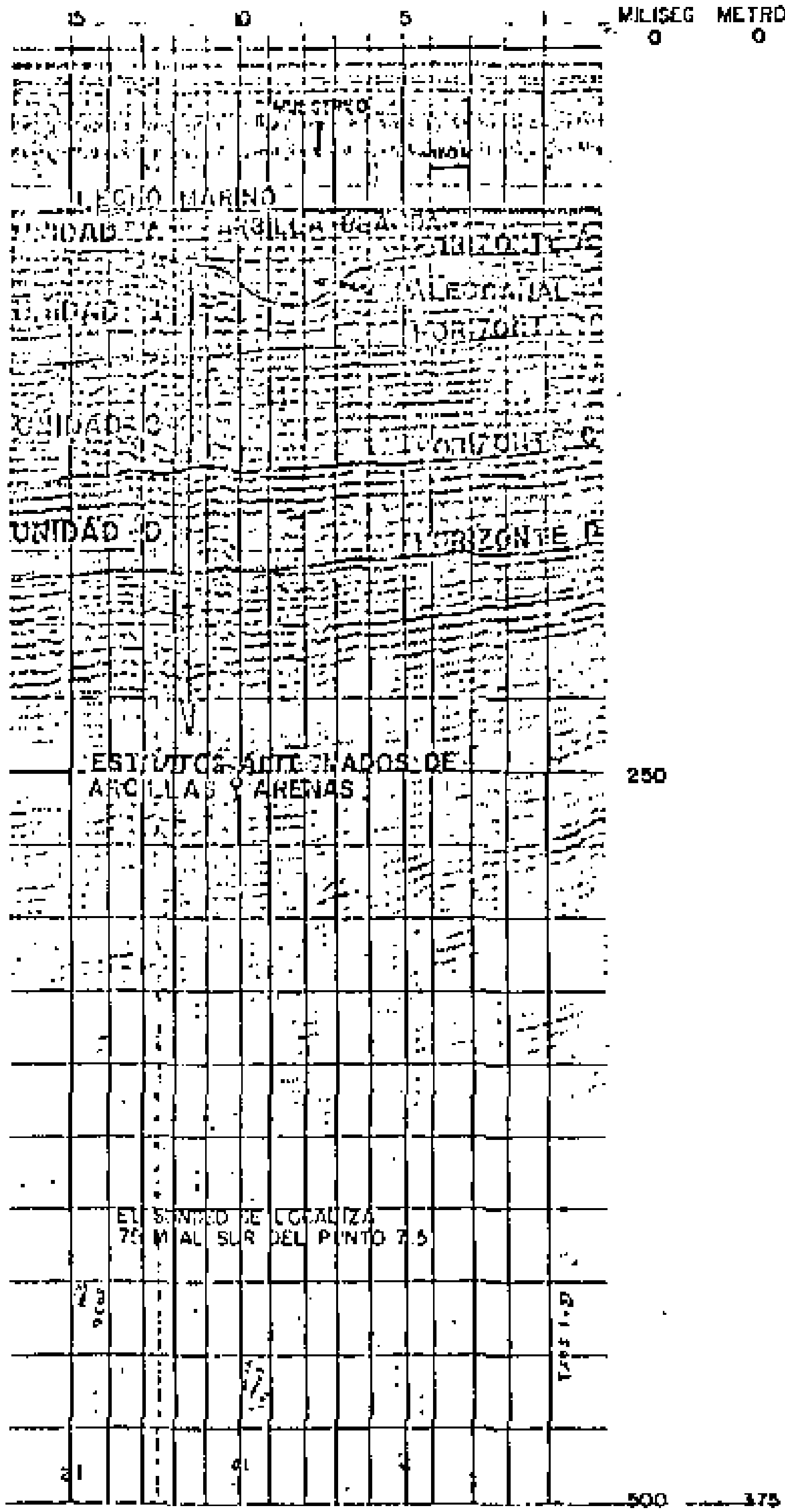


FIGURA 9. REGISTRO DEL PERFILADOR PROFUNDO DE ALTA FRECUENCIA ILUSTRANDO UNA ESTRUCTURA TIPICA DE CORTE Y RELLENO (PALEOCANAL)

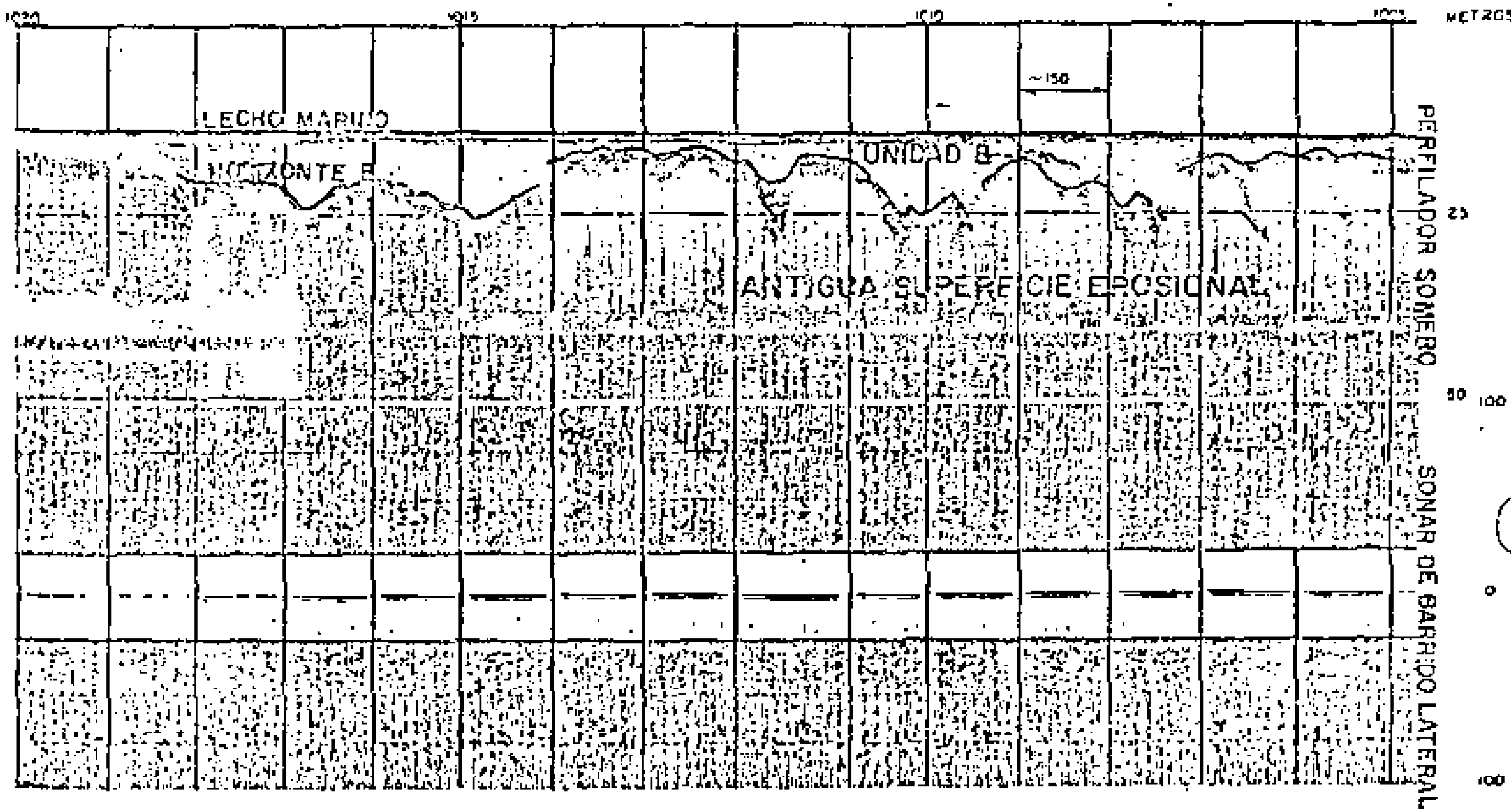


FIGURA 10. REGISTRO DEL PERFILADOR SOMERO
MOSTRANDO UN ANTIGUO HORIZONTE
SUPERFICIAL EROSIONADO.

(22)

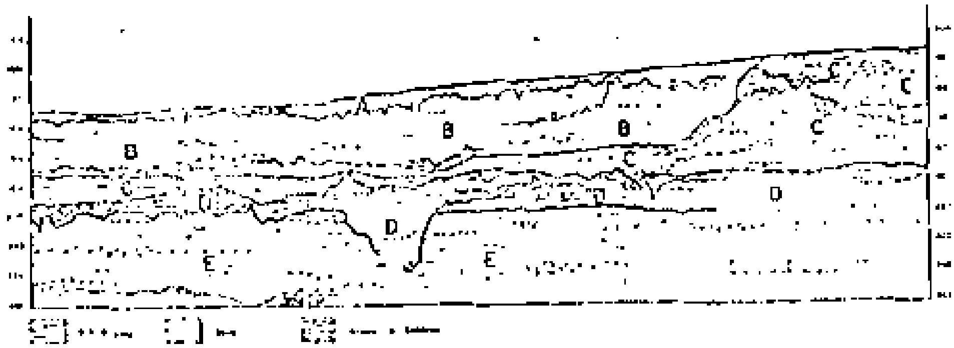


Figure 7: Interpreted sparker section from the Forties area using data from boreholes and regional seismic mapping (after McCave *et al.* 1977). Major seismic reflectors are listed 1–7. Formation A — late glacial and Holocene silty clay. Formations B and C – glacio-marine and glacial (?till) deposits. Formation D — channel deposits. Formation E — marine clays.

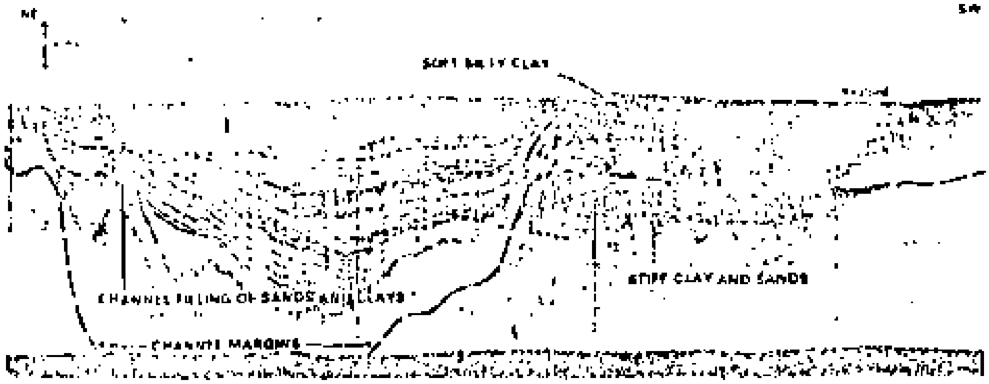


Figure 8: Filled channel close to seabed. The channel was probably cut and partially filled subglacially. Note evidence of several stages of filling and variations in seismic texture. Position 57°55'N, 1°25'E.

Devil's Hole) have remained open on the seabed. As with the lower channels, their distribution is complex; they may exhibit considerable relief and may contain a variety of sediment types deposited during several stages of filling. The last filling episode within some channels appears to have resulted in the deposition of soft silty clays. These sediments are also seen further north (between 58°N and 60°N), below about 110m water depth, where more than 30m of late glacial and Holocene normally consolidated silty clays are developed.

North of 60°N the succession is less well understood. Several phases of both seabed and buried channelling are present but much older Pleistocene deposits are found in the top 100m of the succession[12] and the complex of glacially associated sediments appears to be thinner. This means that an even more complicated sequence of events has affected the sediments close to the seabed.

Still farther north, between 61° and 62°N, seismic evidence suggests that material removed during early phases of Pleistocene erosion has been dumped over the edge of the continental shelf to form a series of offlapping envelopes which have built the edge of the shelf out beyond the original Tertiary margin.

Unlike the dominantly clay sequences in the North Sea the Quaternary in the west is more sandy and commonly has a higher stone content reflecting the dominance of tills and proximal glacio-marine sediments. Soft late glacial and Holocene clays are restricted to protected sites like St. Magnus Bay, where up to 40m of soft clays are present. In general however Holocene sedimentation is limited and appears to have been largely confined to the early Holocene. Over much of the northern North Sea micropalaeontological evidence indicates that shallow water arctic faunas and floras are present within a metre or so of the seabed and therefore it seems likely that there has been little net sedimentation over the last 5,000 years.

GEOLOGICAL HAZARDS

Using data from the regional geological survey a number of potential geological hazards have been identified (figure 9). The term hazard is an emotive one and is defined here to include geological features or conditions which may adversely affect installations on or in the seabed. Some hazards are therefore easily recognised and steps can be taken

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to remove or minimise the risk. Other hazards are more difficult to deal with because their mode of formation in unknown or their occurrence or frequency of occurrence cannot be predicted. Some of the more common potential hazards in northern British waters are described below.

1. Rapid variations in sediment properties
Evidence from the geological surveys has already shown that sediments near the seabed in northern British waters have been deposited under a variety of conditions and may have been subjected to a complex post depositional history (figure 7). The fabric of sediments deposited from or under glaciers shows a wide variation in the degree of sorting and grain orientation.[13,15] This level of sorting, which can vary abruptly within a few centimetres as a result of depositional conditions or post-depositional disturbance, may therefore radically affect static soil properties or the behaviour of the soil under dynamic loading. Evidence of time breaks and erosion surfaces are common in the geological succession and mean that the sediments below disconformities may have been considerably affected by post-depositional events before sedimentation

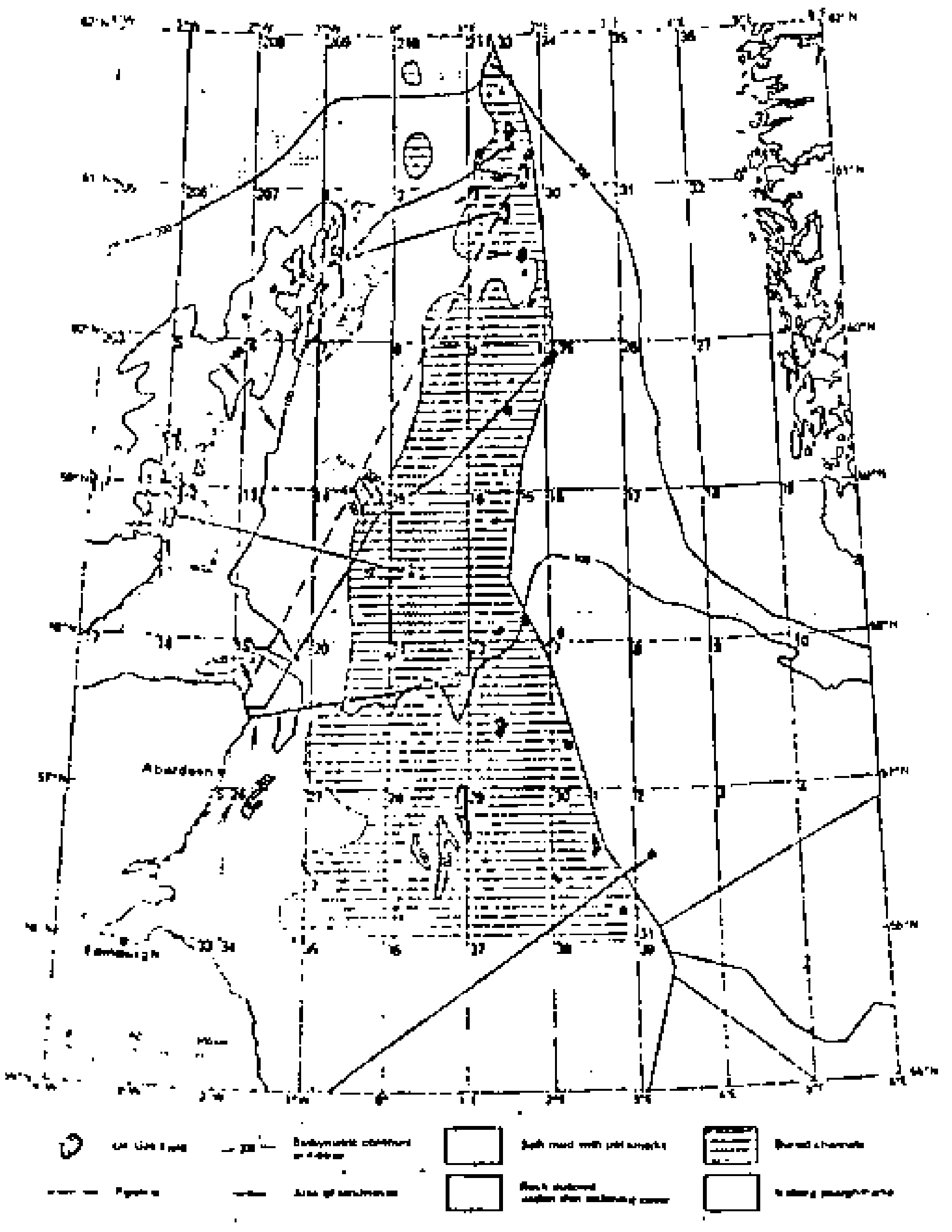

Figure 9: Distribution of potential geological hazards in the northern North Sea

resumed. These may include:

1. Sorting, with the winnowing away of fines to leave a coarse lag deposit.
2. Temporary loading by ice or desiccation producing over-consolidation or both small or large scale fissure systems.
3. Partial diagenesis producing layers with abnormally high shear strength.
4. Local leaching of marine sediments by fresh water causing a reduction in remoulded strength and an increase in sensitivity.
5. Cryoturbation by permafrost producing local fabric rearrangements.

Glacial terrains also contain a variety of positive relief features which may be preserved on the seabed or remain as buried remnants of earlier glaciations. Features such as moraines, drumlins or esker trains commonly contain concentrations of sand and gravel and usually have a distinct morphology with predictable trends and distributions. Uncertainties over the direction of ice movement and marine erosion makes the recognition of these features difficult but moraine material has been identified in some areas.[8,9]

In contrast sediments deposited under marine conditions exhibit more uniform properties because they have been deposited under a standing body of water. Sedimentation usually continued in a regular pattern without significant breaks, although if arctic conditions prevailed gravel grade material may have been deposited due to melt out or overturn from icebergs.[14] These mechanisms can produce either scattered matrix-supported pebbles and boulders or lenses of gravel. Normally consolidated marine sediments may also contain zones of over-consolidated soils where large icebergs or ice shelves have temporarily grounded. This would have occurred during glacial periods when sea level may have been 120m or more below present OD with the northern North Sea reduced to a shallow embayment.

The late glacial and Holocene clays are apparently uniform over large areas but where they represent the last phase of channel filling they may exhibit considerable evidence of post-depositional disturbance, perhaps as a result of de-watering (figure 10).

It is therefore apparent that extreme care must be taken when attempting to correlate laterally and vertically the extremely complex sediments of the northern North Sea. Some of the features observed in modern high resolution seismic records are improperly understood and until these can be adequately explained in terms of the physical properties of the soil it is imperative that closely spaced samples are taken to calibrate the seismic records.

2. Open and buried channels

Over a large part of the northern North Sea open and buried channels form a complex anastomosing network with a local relief which may exceed 200m.[3,4,7,16] Some channels have been cut during periods of lowered sea level and exhibit typical fluvial channel patterns. Most however appear to have been cut sub-glacially and show a quite different morphology having been cut and filled under a different hydrodynamic regime.[8,11,17,18]

Several widespread phases of channelling have occurred during the Pleistocene and some of these may be correlated regionally.[7] Both open and filled channels are present on the seabed between 56°N and 58°N and these were probably cut during the maximum of the last glaciation. Earlier periods of widespread channel erosion have also been mapped and probably mark earlier glacial advances (figure 7). Local channel erosion is however common and, where energy levels were high below the ice sheet, isolated deep channels may be cut.

Sub-glacially eroded channels form closed systems with no strong regional orientation and with considerable relief. They are filled with a variety of sediment types but commonly may contain sand or gravel lenses. These may act as aquifers providing drainage ways for adjacent clays, permit migration of fluids on a large scale, or act as reservoirs for gas zones. The channels are filled by intermittent pulses of sediment, usually with locally high sedimentation rates. Sediments deposited in this way often exhibit loose grain packing arrangements and may therefore be subjected to subsequent settling with a re-ordering of grain structure or liquefaction.

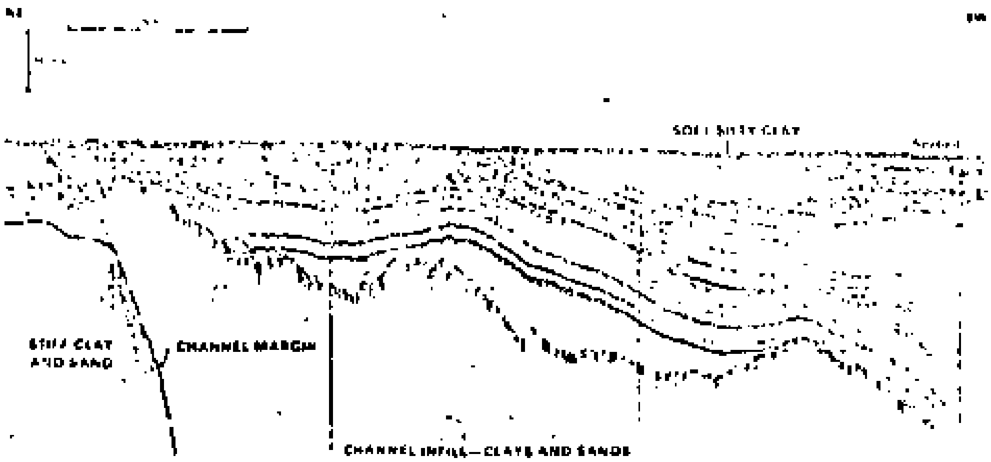

Figure 10: Edge of a filled channel close to seabed. Note evidence of at least two stages of filling, variation in seismic texture and post depositional disturbance of layering. Position 57°35'N 1°35'E.

Channels may also contain a clay grade sediment deposited by settling during quiescent periods, by mass flow of unconsolidated material from the sides of the channel or in the form of partly consolidated clasts and blocks of clay excavated from the channel walls. Channel sediments may therefore vary across the channel reflecting changes in the hydrological regime or down the channel as deposits become finer distally.

Channel systems therefore require careful mapping not just to define their distribution but also with instruments of sufficiently high resolution to identify subtle changes in the fill material and supported by sufficient sampling to calibrate the seismic records.

3. Rock platform areas and mobile sediment

Rock platforms, although not representing hazards in themselves, may be the site of a number of potential problems (figure 9). By definition the rock platform is an area largely devoid of sediment cover. This often means that high wave energy conditions prevail and that a patchy cover of recent sediments is shifting over the seabed. It is also important to know the distribution of rock types and main fault lines so that the rock properties may be taken into account when considering sites or routes in areas of rock exposure.

Seabed topography, on both the large and small scales, reflects the geology. Well indurated sedimentary rocks typically form smooth rock platforms (figure 11) while less well-cemented sandstones and shales can form an irregular topography with a relief of a few metres. Inter-bedded or intruded volcanics in a soft sedimentary sequence may also produce a similar effect.

Hard crystalline rocks may form extensive ridges, with considerable overall relief and significant micro-relief (figure 12). Some crystalline rocks may also show considerable relief because of deep but irregular weathering profiles. This is particularly the case in some granites where on land *in situ* weathering to a depth of 20m is common. The result may therefore be to produce zones of unconsolidated weathered granite with pinnacles of hard fresh rock.

The sediments found on rock platforms are usually of sand or gravel grade although Pleistocene tills and Holocene clays may fill hollows. Areas of gravel usually represent a fairly stable seabed with movement only during major storms. Sandy material, however, may move rapidly in the form of sand waves, sand ribbons or linear sand banks. Sand waves vary from less than a metre in height and a few metres wave length to large sand waves or sand banks up to 60m high with smaller sand waves moving on both the lee and stoss faces

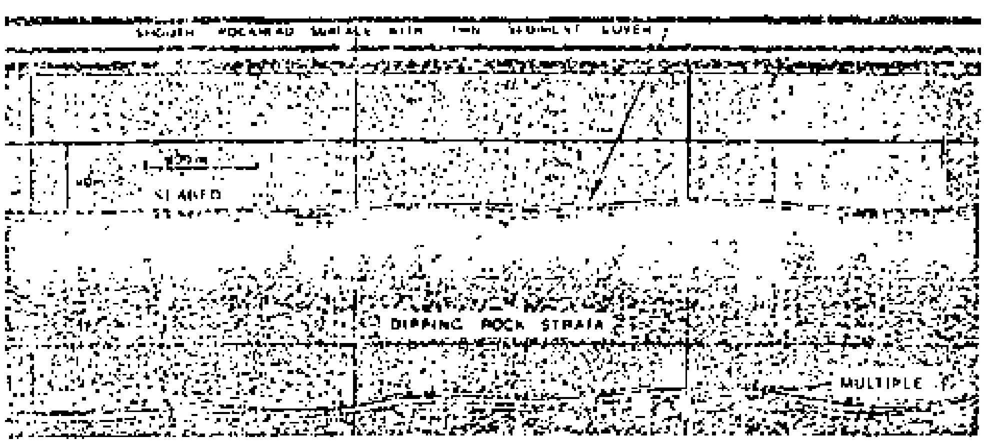

Figure 11: Smooth rock platform with a thin veneer of sediment.
The ice planed surface is composed of gently dipping Mesozoic siltstones.

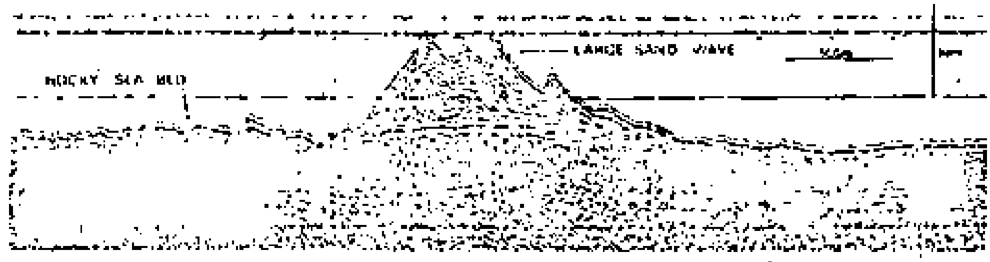

Figure 12: Irregular rocky seabed with a micro relief for several metres downstream from a large sand bank with mobile sand waves on the stoss side.

(figure 12). Important sand wave fields may also be developed near significant headlands where there is a net sediment movement in one direction (figure 9). Such fields are seen south of Rattray Head near Aberdeen and south of Duncansby Head in Caithness.

Little is known about the rates of sand movement or the relative levels of sorting during and between major storms. Transport rates are important however, since the emplacement of an obstruction may be sufficient to induce scour around or beneath structures, or sediment movement may re-expose previously buried pipelines. In high energy zones typical of rock platforms the sediment distribution on the seabed reflects the intensity of the last major storm thus, as with the water mass, it is important when planning installations to consider the sediments in terms of 'the 100 year seabed'. The conditions of transport during major storms are the critical factors rather than the present sediment distribution.

4. Pockmarks

Pockmarks are shallow depressions found extensively in the northern North Sea (figure 9), including the Norwegian Trench, in areas of late glacial and Holocene soft silty clays (figure 13).[7,8,19] They were first recognised by King and MacLean[18] off the east coast of Canada and have subsequently been reported from the Gulf of Mexico and the Mississippi Delta, the Barents Sea, around Malaysia, Thailand and Borneo, and off New Zealand and they are probably present in the Black Sea, the Persian Gulf, the Mediterranean and off the Brazilian coast. They are thus a worldwide phenomenon, and have attracted particular attention in the North Sea because, if they are still being formed, particularly if they form catastrophically, then they may pose a threat to seabed installations. The danger they represent may be due to either or both their morphology or mode of formation.

Pockmarks can vary considerably in size and shape but in the northern North Sea, in the IGS study area,[21] they are typically oval in form with a mean long axis of 57m oriented at N21°E,[22] parallel to the principal direction of bottom current flow, and with a length/breadth ratio of 1.4:1 (figure 14). The features have a mean depth of 2m, though they may be up to 8m deep and are commonly asymetrical in profile with evidence of slumping of material into the depression. The distribution density on the sea floor varies greatly but where they are most common a mean density of 35 per square kilometre has been mapped. The distribution in some areas appears to be random while in others distinct lineations along a mean axis of N10°E have been observed.[22] High resolution shallow seismic records also show that buried pockmarks are present in the North Sea within the late glacial and Holocene clays, but neither surface nor buried pockmarks have yet been observed in the clay areas off the west coast of Scotland or in the extensive Holocene clays in the Irish Sea.

It is now generally accepted that pockmarks are an escape phenomena and model experiments have shown that gas bubbled through soft clays will produce a shallow depression. The mechanism of formation is that fine sediments are entrained in the escaping medium and carried into suspension.[23] Bottom currents then carry the fines away and material within the seabed collapses inward either by fluid flow or by mass movement due to rotational shear and results in a shallow depression. The medium which entrains the sediment could be young biogenic gas of shallow origin or old petrogenic gas from depth, or water. In the North Sea gas seems the most likely medium, though water derived from buried ice or permafrost in the late Pleistocene clays and escaping through a cover of younger clays cannot be excluded as a possible mechanism. It must also be remembered in the case of the North Sea that the deep source

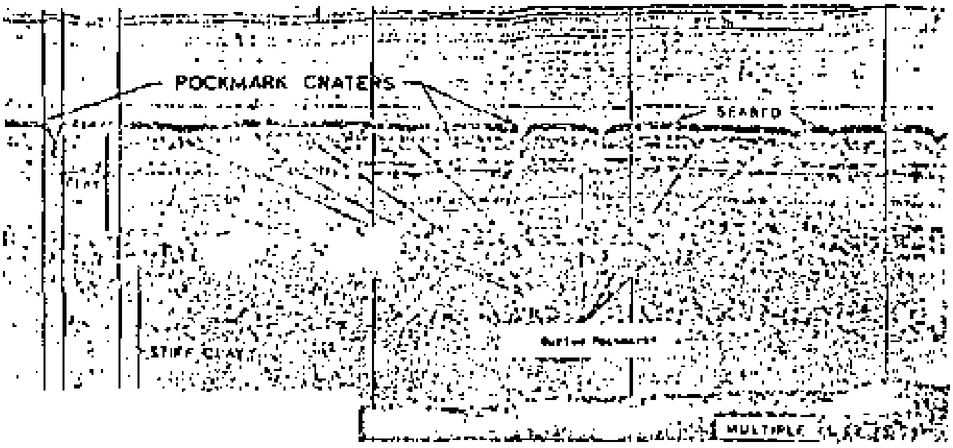

Figure 13: Deep tow sparker record from IGS pockmark study area showing acoustically multilayered soft clays overlying stiff clays and sands. Pockmarks are common both on the seabed and buried at several horizons below the seabed.

petrogenic gas must migrate through at least 600 m of stiff Quaternary clay to reach the surface. It seems most likely therefore that Quaternary and Holocene biogenic gases form the most important of the entrainment media in the North Sea.

In considering the trigger mechanisms which might induce the escape of gas several points must be considered. Firstly, since there has been little net sedimentation in the Holocene, pockmarks could be relict features formed in much shallower water during periods of lowered sea level. Secondly, the silty clays in which pockmarks occur have low undrained shear strengths (less than 20kPa) and therefore could not allow the development of a gas reservoir at any significant pressure. Thirdly, high resolution seismic data show little evidence of disruption of the sediments beneath pockmarks which would indicate feeder pathways from depth. Fourthly, no statistical relationship has been demonstrated between areas of acoustic blanking (figure 15) (interpreted as gassified sediment) and the presence or absence of pockmarks. Lastly, the North Sea clays are fairly typical of open shelf sediments and do not have a high organic content.

Taking these points into consideration it seems unlikely that large quantities of gas at significant pressure could accumulate before venting. This implies that pockmarks could not be formed by major catastrophic events, but rather by intermittent low pressure emissions over a period of time. It is also possible that gas could vent below normal escape pressures if the sediment were disturbed by some other mechanism. Neither excessive sediment loading nor wave induced drag are likely causes in the North Sea because of the water depth and low rate of deposition of sediment. However earthquake shocks are of sufficient magnitude and frequency to disrupt the sediment and these could provide the trigger to initiate the process of gas release.

The mode of formation of a pockmark therefore depends largely on the physical properties of the sediments in which it is formed. The volume of gas, its method of venting (through fissures or loosely packed grains), the gas pressure which is allowed to accumulate, and the cohesion of the sediment all affect the way in which the pockmark is formed and the morphology of the pockmark which results. Pockmarks produced by frequent emission of low volumes of gas and fluid flow of sediments laterally to fill the void created by vented material are less likely to cause problems than pockmarks produced by single or few events with translation of sediment by rotational shear. Studies of pockmarks have so far failed to prove even the basic questions of whether pockmarks are still being formed or whether gas is in fact the principal entrainment medium. The question of their hazard potential therefore remains unanswered.

5. Gassified sediments

Gas bubbles, filling voids in the sediment, are common in many parts of the North Sea. In seismic records the presence of such gas is recognised as a cloudiness, often called acoustic blanking, (figure 15) or as 'bright spots' in processed records.[24] It is less frequently observed in actual core samples (figure 6) but several shallow gas 'blow outs' have occurred during North Sea drilling.

In most cases the gas is biogenic methane produced within the Quaternary succession, perhaps

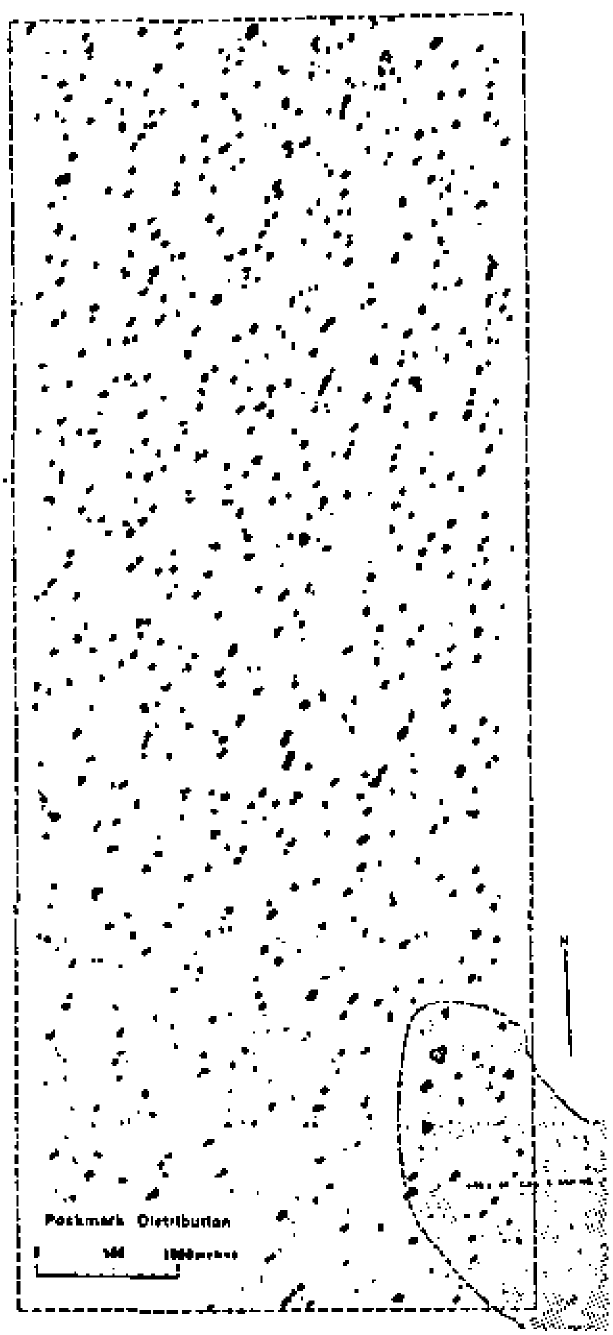

Figure 14: IGS Pockmarks study area showing shape and distribution of pockmarks.
The area is centred at 58°5'N 0°35'E.

derived from inter-glacial peats. It is possible however that some gas zones, particularly in the lower part of the Pleistocene succession, may be derived from petrogenic gases seeping from depth.

The affect of acoustic blanking is produced by the absorbtion of acoustic energy within gassified sediments and gas contents of as little as 1 per cent are sufficient to cause significant cloudiness on the seismic records. Where sand lenses are present, however, the normal laws of reservoir entrapment apply and relatively high pressure concentrations of gas can be developed. The North Sea Quaternary succession lends itself to this situation with a dominantly clay succession forming the source and 'caprock' and relatively rare sand lenses and sand filled channels, both of which allow easy migration and concentration, forming the reservoirs. Divertor systems are not installed until the conductor has been set at about 200m. It is therefore essential that gas pockets are identified before drilling commences because of the danger of ignition on blow out or, in extreme cases, loss of buoyancy for floating vessels where large quantities of gas escape into the water body. Where gas reservoirs are present at higher levels these may effect settlement in gravity structures or the bearing capacity of piles.

6. Seismicity

Britain and Scandinavia have traditionally been regarded as stable areas but recent geological evidence has shown that during the Pleistocene significant subsidence has occurred in the Central Trough of the North Sea and faulting has been observed in the Pleistocene of the Rhine Graben, an extension of the Central Graben of the North Sea.

Significant earthquake activity has been recognised in the area of commercial interest in the North Sea despite observational difficulties.[25] This activity may be related to the rapid subsidence indicated by the geological record. Instrumental monitoring problems arise because of the inadequate density and geometry of existing land based seismography networks. There is a tendency for national networks to look inwards to the land areas where local earthquakes can be located more accurately. Thus in the northern North Sea earthquake epicentres can only be plotted with an accuracy of ±40km while little is known of the depths of the events. The historical data are incomplete and less accurate, depending on subjective records with the larger earthquakes reported on both sides of the North Sea.

A northern North Sea seismic monitoring net is currently being constructed, but in the mean time some general statements can be made from existing modern network data and historical records. Between 10 and 20 offshore events are being detected each year in the magnitude range three to five. The largest well-authenticated British event occurred on the Dogger Bank in 1931[26] and the average reported magnitude (MS) was 5.5 with a maximum of 5.9. Damage was caused along the east coast of England up to 100km from the epicentre of the earthquake. A study of historical records from the southwestern Norwegian coast and adjacent offshore areas shows that twelve earthquakes greater than magnitude 5 have occurred in the past 500 years.[27] In Britain, the IGS file of British earthquakes contains details of five such events in the North Sea since 1866.

Earthquake seismicity has clearly been underrated in the North Sea in the past and careful study is required to allow realistic estimates of activity to be made.

7. Slope stability

Considerations of slope stability in the North Sea have received little attention because the slopes encountered have rarely exceeded more than one or two degrees. Recent work however on continental shelves and slopes in various parts of the world have shown that normally consolidated sediments can fail on slopes of less than 1°.[16-21]

Stiff over-consolidated clays are present close to the surface in parts of the North Sea but many of the normally consolidated late-glacial and Holocene clays which do occur on the shelf and particularly

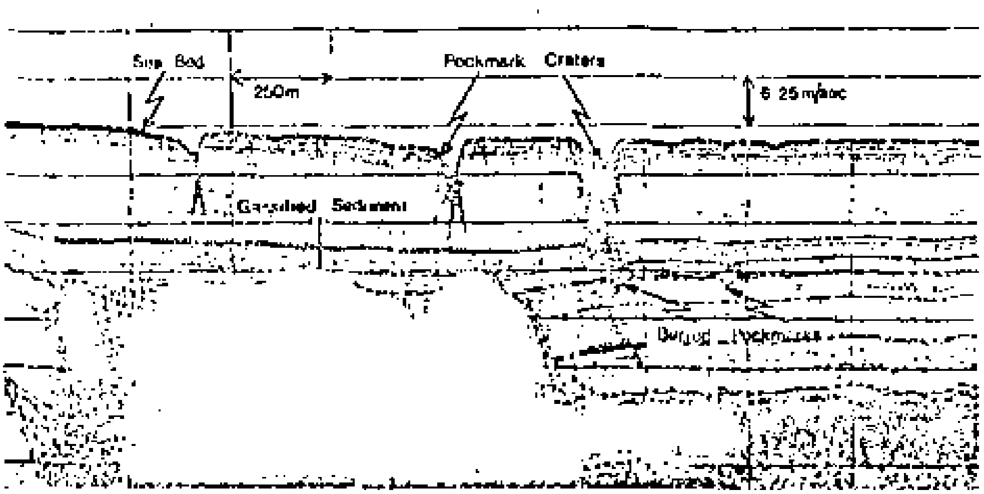

Figure 15: Acoustic blanking in a deep tow boomer record. The cloudiness in the record is attributed to energy absorption by sediments with gas bubbles in the pore spaces.

on the upper part of the continental slope are susceptible to failure where even slight slopes are present.

Little data are available on slope conditions in the North Sea but the study of bathymetric maps from the western side of the Norwegian Trench shows evidence in some profiles of significant breaks in slope suggestive of slope failure with rotational shear escarpments (figure 16). More significantly, at the top of the continental slope off southwestern Norway, a massive zone of slumping has been mapped where an estimated $800 km^3$ of sediment have been removed.[24] Insufficient information is available to identify slope failures on the edge of the British continental shelf but there is no evidence to suggest that such failures would not occur in this area.

The crude bathymetric maps available for the continental shelf and upper slope can only be used to identify large scale slope failures. Regional slopes on the plateau of the northern North Sea shelf are usually less than 0.5° and may be less than 0.1°. Local slopes may exceed this however and in some cases slopes of several degrees have been recorded. Normally consolidated sediments in these zones are therefore liable to failure. On the western slope of the Norwegian Trench elongate scars, observed on side scan sonar records, sub-parallel to the slope and with a downslope displacement of a few metres have been mapped on regional slopes of about 1°. This evidence suggests that the slope failure at all scales may be present on the continental shelf.

In shallow water, slope failure may be caused by excessive sediment loading or wave induced drag. Neither of these mechanisms are currently operative in the North Sea though both conditions could have applied in the northern North Sea at the end of the

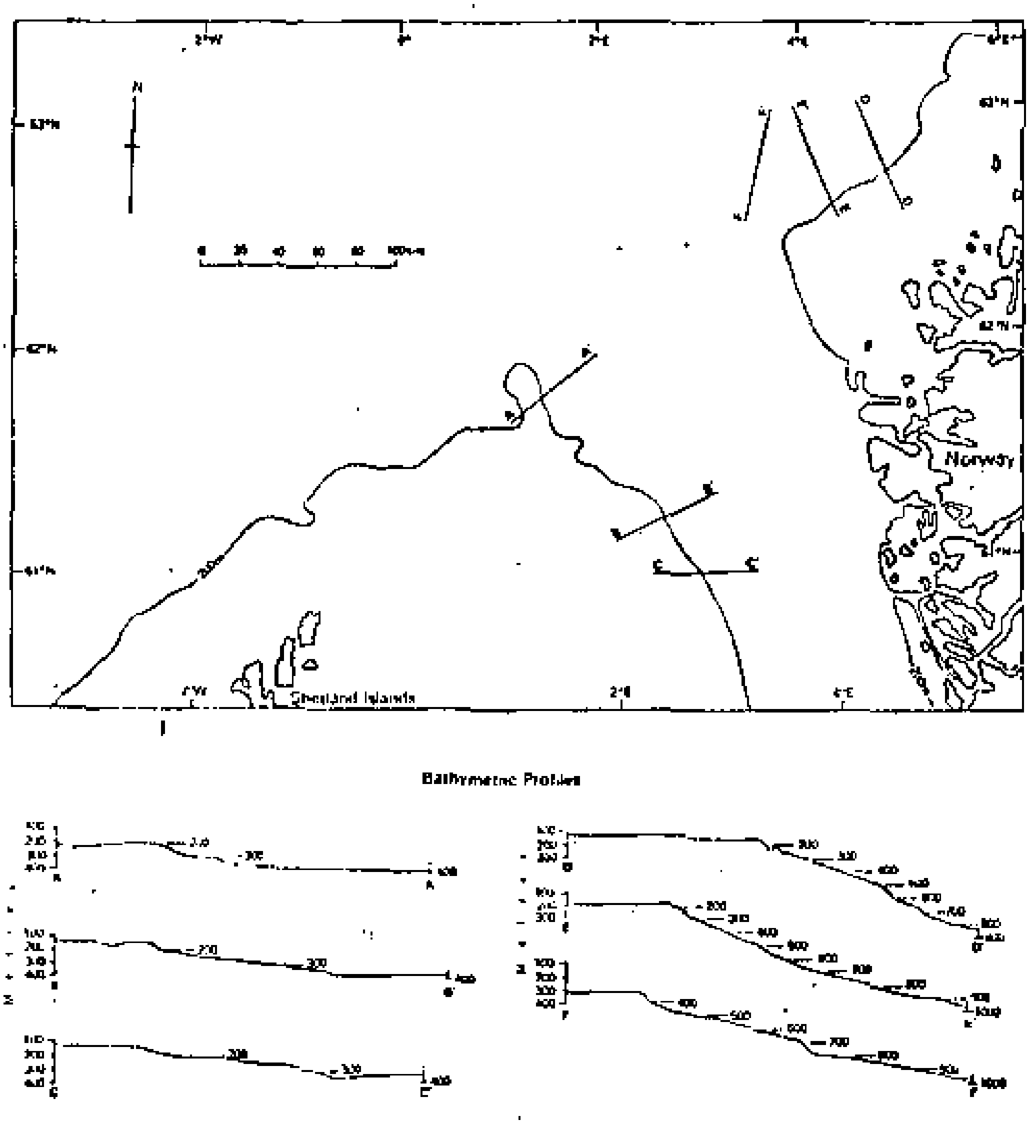

Figure 16: Seabed slope profiles from the northern North Sea.

last glaciation. Under present conditions however earthquakes can provide the trigger necessary to cause slumping[33] and it seems likely that catastrophic slope failures can and will occur on the shelf and at the top of the continental slope.

* * *

The ultimate objective of the site engineer is to install the necessary equipment as economically as possible having made adequate provision for the safety and maintenance of the site. In the offshore industry he is constrained by a number of factors, principally the choice of the site, the time scale of construction, and basic information on which to define his design criteria. There is little he can do about the first two constraints but it is clear that, during the development of North Sea hydrocarbons, the site investigation industry has been allowed to lag unacceptably far behind other aspects of offshore construction, with consequent unnecessary over-design and more seriously, under-design, usually requiring remedial action during or immediately following construction.

If offshore site investigation is to improve more and better data must be provided to the design engineer. The geological evidence discussed above has shown that the geological environment is extremely complicated and that at the present simplistic level of site investigation some at least of the potential hazards would not be recognised. It is imperative therefore that both the quality and quantity of sampling is improved.

In practice this means that more *in situ* measurements are required either using seabed or down-hole probes. This should include direct measurements made using piezometers, accelerometers and a vastly increased use of geophysical techniques (natural gamma, gamma-gamma, neutron, resistivity and direct velocity measurements). It is important also that measurements of ground conditions should not be confined to a one-off measurement in optimum environmental conditions. Recording instruments should be used to provide data over at least one full winter season and of course every major structure should include a full suite of monitoring devices both on the structure and in the surrounding foundation. It is interesting for example to note that in the case of large structures on land, such as major power stations where the site is immediately observable, that between three and five times as many boreholes are used to explore the site as are drilled during the site investigation for offshore structures.

Offshore considerable emphasis is placed on shallow geophysics. In the past standards of geophysical work have varied greatly and it is only in the last few years that instruments have become available which can produce data of the quality required by the site engineer. The most commonly used instruments in the past were the pinger and sparker. The pinger gives a penetration from a few metres to perhaps 20 metres with a resolution of one to two metres. The sparker on the other hand could give penetrations of over 100 metres but with resolution of at best several metres and an initial pulse width commonly between seven and ten metres. The modern deep tow sparkers and boomers given penetration of up to 50 metres and resolution of about 0.5 metre, a performance which approaches the engineer's requirements.

The geological evidence has shown however that further calibration is required before the geophysical records can be fully interpreted. It is thus absolutely essential that a complete record of the stratigraphic succession is obtained. Current offshore sampling practice falls far short of this where samples representing about 21.5 per cent of the borehole are obtained totalling a 47 per cent recovery rate of the cored portion of the hole (table 1).

These data are supplemented by some *in situ* measurement, but it is equally important that samples are obtained so that the depositional environment of the sediments is identified. This will also permit better prediction of soil behaviour and reveal features such as fissuring, gassification or sediments liable to liquifaction.

The quantity of data available to the design engineer can therefore be increased. The interpretation is however, more difficult because it is the product of a complex interaction of a number of processes which is being measured. Some of these processes are geological events occurring on a 'geological' time scale and there is simply not enough information to make a statistical estimate of the frequency with which events may occur. Thus until a reasonable data base is established of for example the frequency of macro seismic events, or the rates of sediment transport during severe storms it is impossible to quantify the real potential of many of the hazards identified above.

It is clear therefore that offshore site investigation practice can be improved and recent advances have

Table 1: Typical core recovery during site investigation drilling

Sampling intervals	*% of hole cored*	*Actual % of sample recovered*
0–10m	70.4	38.0
10–20m	56.4	19.8
20–30m	36.0	17.5
30m+	19.0	10.8
Average for hole	45.8	21.5

(32)

shown that some progress is being made. At present however the pace of development is motivated almost entirely from within the industry and it is pertinent to question whether it is reasonable to expect industry to carry this self-regulatory responsibility. It could for example be suggested that UK governmental practice should follow that of the USA where detailed standards and requirements have been established and where there can be intervention in the selection of areas on site foundation criteria prior to their release for licensing.

The opportunity now exists for governmental and regulatory authorities to cooperate with industry in defining reasonable codes of practice. Recently environmental factors have received considerable attention. However, care is required to ensure that the real dangers are fully appreciated. Transitory phenomena such as minor pollution attract great publicity but other aspects concerning the ultimate stability of a platform or the fact that attempts to trench a pipeline in boulder clay may put the pipe at greater risk than leaving it on the seabed are more important.

The current dialogue between the scientific and technical disciplines and industry and government can only help to improve current standards and practices. In this respect the fullest understanding of the geological environment in which the industry operates is considered essential for efficient and economic development in the future.

ACKNOWLEDGEMENTS

Much of the work discussed in this paper represents the results of studies by a number of IGS geologists. The assistance of these colleagues, too numerous to mention by name, is gratefully acknowledged, as is their advice and constructive criticism during the preparation of the manuscript. The views expressed herein are however those of the author. Permission to publish the paper is given by the Director of IGS.

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4 Northwestern Gulf of Mexico — engineering implications of regional geology

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McClelland Engineers Inc

As a result of accelerated petroleum exploration and development activities during the past several years, many geotechnical site investigations have been required and conducted on the continental shelf and upper continental slope in much of the northwestern Gulf of Mexico. Although the influence of geology on offshore foundation design has been recognised for many years,[1] recent investigations have shown that an understanding of the regional geology can be a dominant factor in planning an offshore site investigation. In the shallow shelf areas with relatively few potentially hazardous geologic conditions, and in areas where it is unlikely that features occurring at relatively short distances from the proposed construction site will affect the site, only a detailed study of the immediate vicinity is necessary. However, for upper slope sites and sites near depocenters such as the Mississippi River Delta, large areal surveys are required to understand geologic processes in the area and how these processes might affect a structure. Surveys for the siting of a single structure in such areas may need to cover 100 to 200 square miles.

The purpose of this paper is to briefly describe the geologic setting and the late Cenozoic geologic history of the northwestern Gulf of Mexico as they influence the engineering geology, and to illustrate the effect of regional geologic conditions on the design of site surveys. No attempt is made to thoroughly describe geologic features or to assess their potential engineering significance.

GEOLOGIC SETTING

Physiography

Structures are being sited in two major physiographic provinces of the northwestern Gulf of Mexico: the continental shelf and the upper continental slope. The continental shelf is the offshore extension of the Gulf Coastal Plain[2] and has been referred to as the Texas–Louisiana Shelf.[3] The shelf is primarily a constructional feature and extends as much as 60 to 140 miles offshore. Based on physiographic considerations of regional seafloor slope, the seaward edge of the shelf approximately coincides with the 600-foot depth contour (figure 1). In several areas, however, the shelf break is in shallower or deeper water. The shelf generally slopes gently to the south or southeast at less than 0.2 per cent. The seafloor is typically smooth to broadly undulatory, but with distinct, local irregularities, such as fault scarps, mud mounds, and exposures of resistant cap rock, salt or coral. The upper part of the continental slope is several tens of miles wide and is commonly characterised by much more irregular topography than the shelf to the north and northwest. Much of this irregular topography is believed to be the result of extrusion of salt from beneath the thick section of Cenozoic sediments to the north.[4] The seafloor slope is typically 4 to 8 per cent, but locally is much steeper.

Stratigraphy

The Cenozoic sediments in the northwest Gulf of Mexico comprise the western part of a large clastic wedge. These sediments were deposited over several thousand feet of evaporites, chiefly salt. The clastic wedge is basically comprised of a progradational sequence of sediments deposited in depocenters that gradually migrated seaward. The salt is probably correlative with the Louann salt found beneath the onshore portion of the Gulf Coastal Plain.[4] The salt is typically one to several thousand feet thick. Overall thickness increases basinward, probably as a result of southward flowage due to loading by the prograding wedge of sediment.[4]

Most of the sediments overlying the salt were brought into the Gulf of Mexico by the present and ancestral river systems draining the south-central parts of the North American continent. These chiefly terrigenous sediments are more than

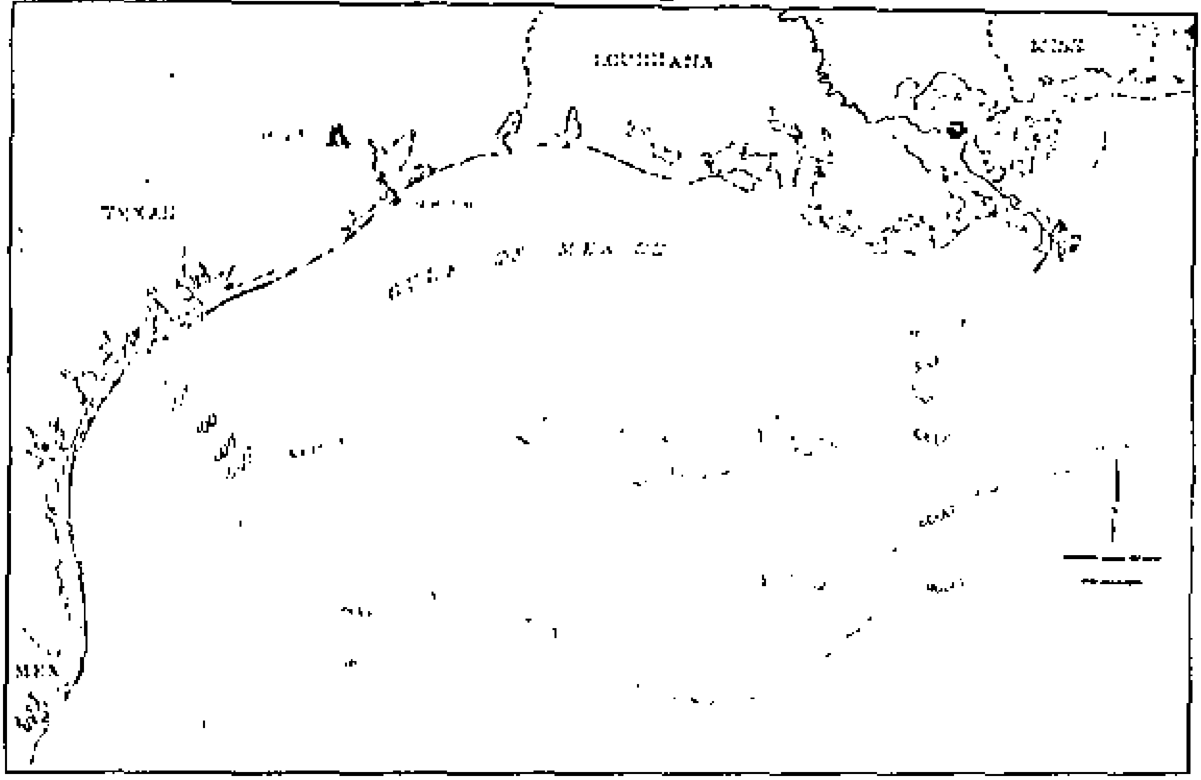

Figure 1: Generalised bathymetric map, northwestern Gulf of Mexico. Depths are in feet

fifty thousand feet thick in some areas.[5] They consist chiefly of interbedded sandstones and shale with the shale being predominant and increasing down-dip.[4]

The Quaternary section ranges from about 2000 feet thick nearshore Texas to more than 12,000 feet thick near the shelf edge offshore Louisiana.[7] Quaternary sediments consist of units of unlithified to partly lithified sand, silt and clay. There are Quaternary submarine landslide deposits several hundred feet thick in some parts of the outer shelf and upper slope. Evidence for these deposits is high-resolution seismic data and anomalous sequences of neritic and bathyal microfossils.

Holocene sediments are probably less than 20 feet thick over much of the shelf, but are several hundred feet thick in the vicinity of the Mississippi Delta.[8] The most extensive Holocene sediments on the middle and outer shelf and upper slope are silt and clay.[8]

Structure

Salt and shale diapirs and down-to-the-basin faults are the two most significant structural features in the northwest Gulf of Mexico. Together they account for nearly all of the known local structural deformations in the Cenozoic section and are probably intimately related to one another.[4] The nature and distribution of diapiric structures is shown in figure 2. On a larger regional scale, strong subsidence of sediments within the Gulf Coast Geosyncline was continuous throughout most of Cenozoic time and is probably continuing through the present period. Except where locally disturbed by diapirs or faults, the upper Cenozoic beds generally dip basinward.

Regional, down-to-the-basin growth faults, roughly parallel to the shelf edge, are common in the area. These faults are believed to result from lateral, basinward flowage of salt at depth.[4] By definition, they formed contemporaneously with sedimentation and significant expansion of the section on the downthrown side is characteristically associated with the faults. Recent active growth faults have seafloor expression and are mostly confined to the outer shelf and upper continental slope. Seafloor scarps and steep slopes associated with these growth faults are known to be as much as 100 feet high. Older buried growth faults are common beneath most parts of the shelf. Smaller, antithetic faults related to the major faults and other local faults associated with diapirs also occur.

The large growth faults are several tens of miles long and generally extend to depths of several thousand feet. Bruce interprets deep seismic data to indicate that the fault plane of at least some growth faults flatten with depth.[9] Garrison and Martin state that the fault planes of growth faults commonly dip 35° to 70°; they are generally steepest near the seafloor and flatten with depth.[10]

LATE CENOZOIC GEOLOGIC HISTORY

The late Cenozoic history of the area is characterised by progradation of the coastline and shelf break. As depocenters migrated basinward, progressively younger growth faults and diapirs formed in association with them. Although overall progradation and basin subsidence has probably continued through the Quaternary, several major

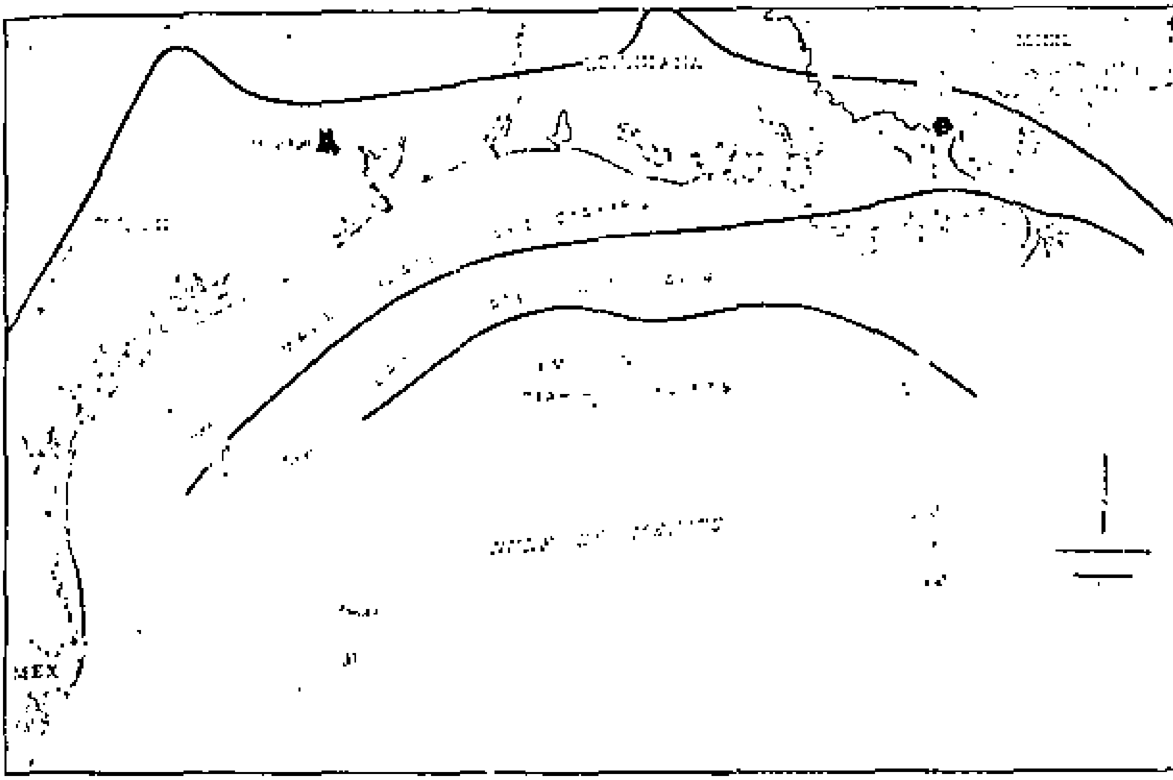

Figure 2: Areal distribution of major groups of salt diapirs on the continental shelf off Louisiana and Texas (after Woodbury et al[4])

oscillations of sea level during the Pleistocene significantly modified depositional patterns in the northwestern Gulf basin. Maximum eustatic lowering during the late Pleistocene (Wisconsin) may have been as much as 525 feet and occurred about 15,000 years ago.[11]

As a result of this most recent lowstand, rivers migrated across the shelf and formed deltas at, or near, the present shelf edge. Some of these deltaic deposits define parts of the shelf edge. Many of the deltas probably formed relatively rapidly, resulting in large masses of poorly consolidated sediments. In several areas, large volumes of these deltaic deposits were transported several miles downslope by massive sliding.[4] One possible triggering mechanism was storm-wave loading. A wave-loading mechanism would help to explain why few, if any, massive slides have occurred during the Holocene on the upper slope in the northwest Gulf of Mexico (that is, as sea level rose, average bottom loads imposed by storm waves would have decreased). Another possible triggering mechanism was seismic shock. Although earthquakes in the northern Gulf region are rare, they do occur and it is likely that some during the Pleistocene triggered massive slides.

Throughout the Quaternary the Mississippi River, the largest river system in North America, has shifted its active delta lobe and area of maximum sedimentation many times. Within the past 500 years, the modern or Balize delta lobe has built far seaward, almost to the edge of the continental shelf. As a result, sediments from the river are now being deposited on the outer continental shelf and upper continental slope. The rapid sedimentation associated with the delta has resulted in instability in the sediments. This instability has induced movement and produced massive mudflow deposits surrounding the delta. These features have been discussed by several scientists.[12,13,14,15,16,17]

ENGINEERING IMPLICATIONS OF REGIONAL GEOLOGY

Geologic characteristics of engineering significance to offshore sites include the nature and distribution of the shallow earth materials, including gassy zones, the potential for seafloor instability that would affect a site, and seafloor topography. The nature and extent of these features or conditions can not be reliably identified from soil boring information alone. Soil boring data must be combined with high-resolution geophysical data to define and assess the potential significance of engineering geologic characteristics of offshore sites. Although sites are routinely investigated, the arbitrary limits often put on survey areas in the past do not always allow all conditions that may affect the site to be adequately investigated.[18] The area required to be covered by geophysical surveys to acquire design information is a function of the regional geologic characteristics of the area.

Numerous recent studies indicate that the northwest Gulf of Mexico can be divided into three major subareas on the basis of engineering geologic characteristics. The most extensively studied subarea is the continental shelf. The upper continental slope is another subarea. Although more and more work is being done on upper slope sites, the engineering geology of this important subarea is still poorly known. The third subarea includes portions of the shelf and upper slope near the Mississippi Delta. Sedimentation rates near the Delta have been high through the Holocene. These rapidly deposited sediments commonly contain geologic features of particular engineering significance.

Continental shelf

Sites on the continental shelf typically pose no unusual engineering geologic problems. Most of the continental shelf in the northwest Gulf of Mexico is characterized by undeformed, flat-lying or gently dipping strata (figure 3). The seafloor slopes are typically less than one per cent and present sedimentation rates are low, except in the vicinity of deltas. Shallow sediments typically consist of very soft to stiff clays and loose to dense sands and silts which exhibit lateral continuity over wide areas. The seafloor sediments over much of the Louisiana shelf consist of several feet to several tens of feet of very soft clay. However, extensive areas of seafloor instability are not known in areas underlain by these materials except in the vicinity of the Mississippi Delta. Locally, abrupt lateral and vertical variations in shallow materials often occur near reefs, salt domes, buried channels and faults (figure 3). Relatively steep slopes, rugged topography, and scarps are also common in some areas of reefs, salt domes, faults and mud mounds. Where soft sediment is associated with these features, local instability may result. All of these features can be mapped and adequately assessed for siting typical platforms by surveying areas no more than a few squares miles in extent (figure 4).

Upper Continental Slope

All features found on the continental shelf occur on the upper slope, except buried channels. The most significant difference is the increased seafloor slope. On a regional scale, upper slope topography is quite irregular, partly as a result of numerous, broad salt ridges (figures 1 and 2) just below the seafloor.[4] Active faults are also generally more common than on the shelf (see figure 5).

Several sites on the upper slope offshore Texas and Louisiana are underlain by slide deposits. The most recent sliding apparently occurred during the late Pleistocene when sea level was lower and sedimentation rates were relatively high on the upper slope. Individual areas affected by this sliding cover as much as several tens of square miles (figure 5). Failure of prodelta sediments, possibly triggered by wave-loading of thick sections of underconsolidated sediments with high pore pressures, was apparently common. Although no major Holocene slides have been documented, the potential for sliding should always be determined for upper slope sites. This requires that a relatively large area be studied (figure 4). At least several high-resolution geophysical survey lines should extend upslope and downslope from the detail survey grid surrounding the site in question. These lines should extend several miles, and in the upslope direction, preferably to the shelf-slope break, to allow determination of slope steepness, thickness of inferred soft sediment, and mapping and identification of any features that may indicate incipient failure. Care must be exercised in not assuming that nearly flat, local areas on the slope

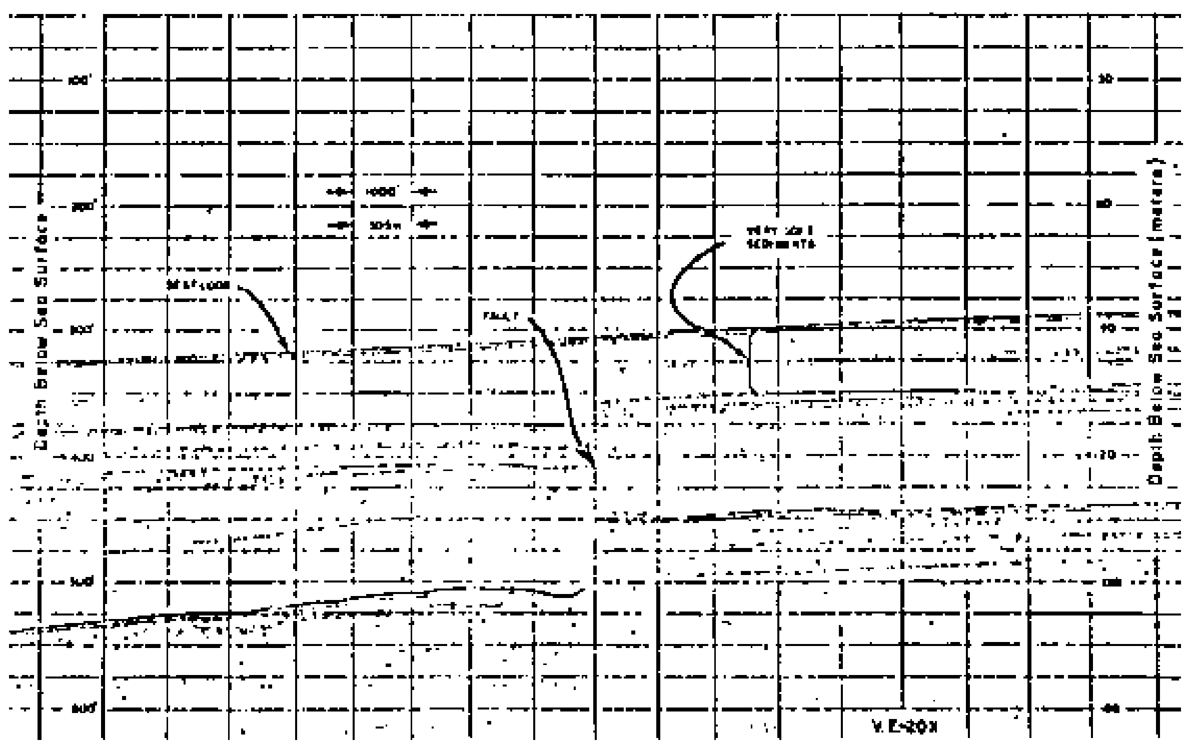

Figure 3: Boomer record of portion of continental shelf offshore western Louisiana

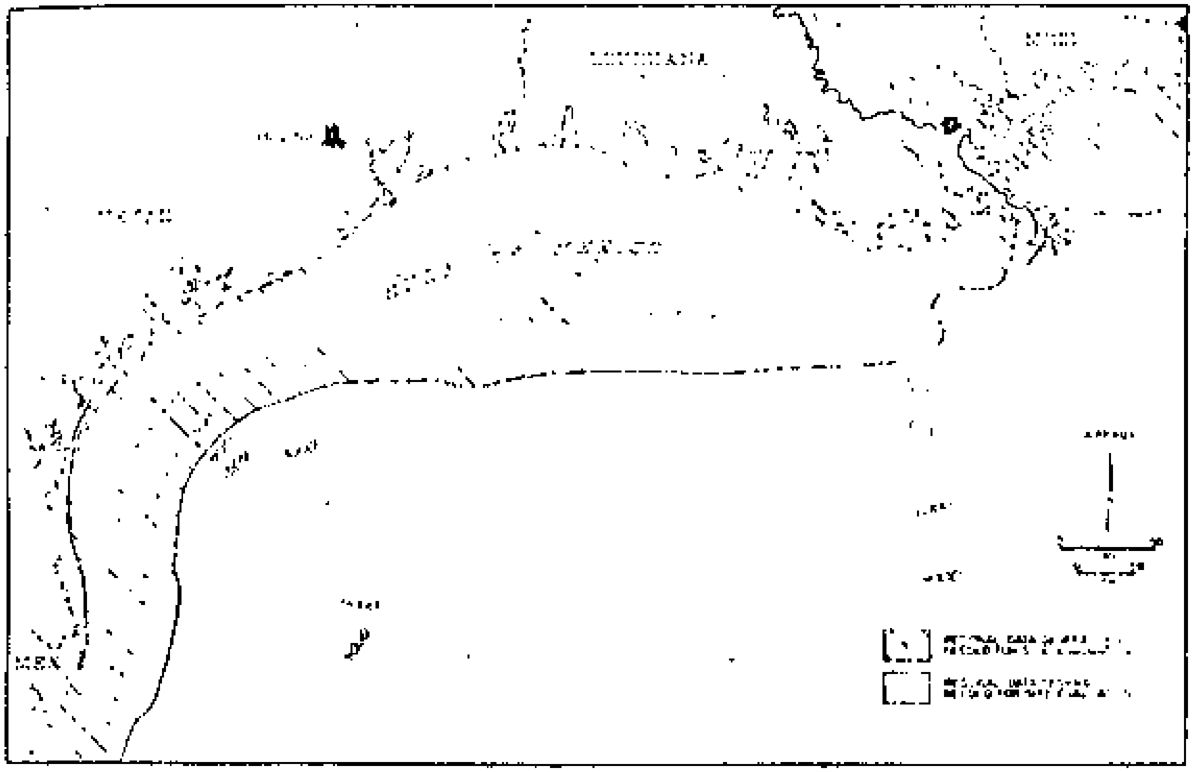

Figure 4: Generalised map showing areas where regional geologic and geotechnical data are generally needed and not needed for offshore site evaluation

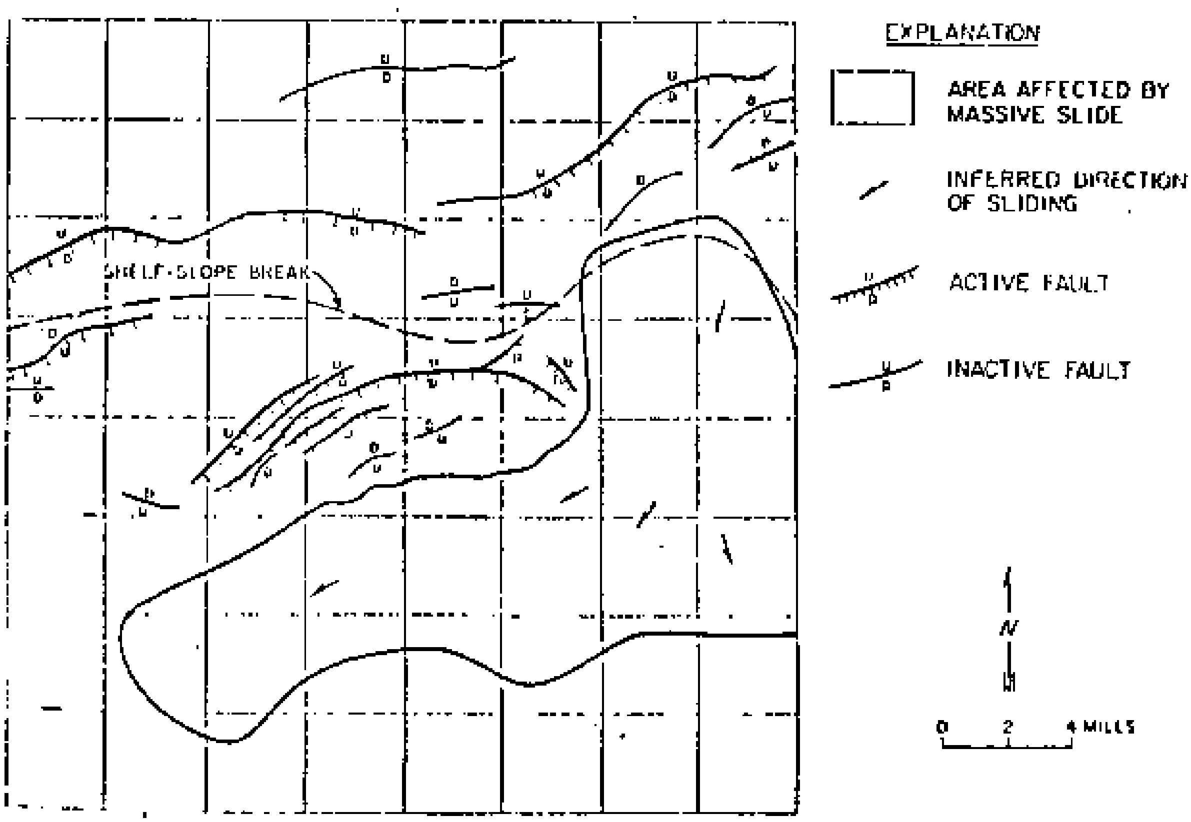

Figure 5: Generalised map of area south of Galveston, Texas, showing deep-seated growth faults and area affected by large, late Pleistocene slide. Group of curvilinear faults between shelf break and northern boundary of slide area is related to growth of a large salt ridge at depth. Each grid square covers an area equivalent to a USA standard lease block, about 9 square miles.

will not be affected by slope failure; slide material can move across such areas when failure occurs further upslope.

Offshore Mississippi River Delta

Most of the geologic features and conditions of engineering significance found elsewhere on the continental shelf and upper slope are present in the vicinity of the Mississippi River Delta. Rapid sedimentation in this area, however, has produced a variety of other features that are also of engineering significance. The most important of these are mass movement phenomena. A few hundred square miles surrounding the Delta are covered by mudflow deposits (figures 6, 7 and 8). In addition the seafloor sediment is creeping and locally slumping. The overall nature of the mudflow deposits is not well understood, however Coleman, Prior, and Garrison have presented an excellent description.[11] Cause of the mudflows apparently is a combination of rapid sedimentation near the river mouth and gas in the sediments. Sediment loading creates instability in the soft, underconsolidated clayey sediments. In addition, the sediments contain organic debris which decays rapidly producing biogenic gas. This gas raises pore pressures and further decreases their stability.

Several platforms are sited on these mudflow deposits, and additional structures are planned. When siting a structure on these deposits, not only must the stability of the sediments at the site be considered but so must the stability of deposits upslope which have the potential of flowing and inundating the platforms' support members. Thus, large areal studies are required (figure 4).

The need for large areal studies in the delta area is also exemplified by the site shown on figure 6. A limited-area site survey covering one or two square miles would reveal no apparent conditions of concern. A regional survey, however, would reveal considerable faulting in the area as well as the mudflows immediately upslope. Because it is not known how frequently new mudflows may be occurring or whether new mudflows are continuing to spread into deeper water, regional studies are needed to evaluate the mudflow and the potential for mudflow at the site. Such studies should include high-resolution acoustic studies repeated on a regular basis to document seafloor movement.

CONCLUSIONS

Safe and environmentally acceptable offshore structures can be designed to perform adequately in adverse environments provided the nature of the environment is understood. Multidisciplinary site investigations are necessary to understand the nature of the environment so that these factors can be considered in design. With regard to the geologic environment, an understanding of the regional geology is necessary. The lessons learned in the northwestern Gulf of Mexico can be applied to other areas of the world. Clearly, the submerged portion of major river deltas and the upper continental slope may commonly warrant large areal studies for structural siting. Shelf areas where sedimentation rates are low can often be adequately evaluated with only a limited-area site investigation.

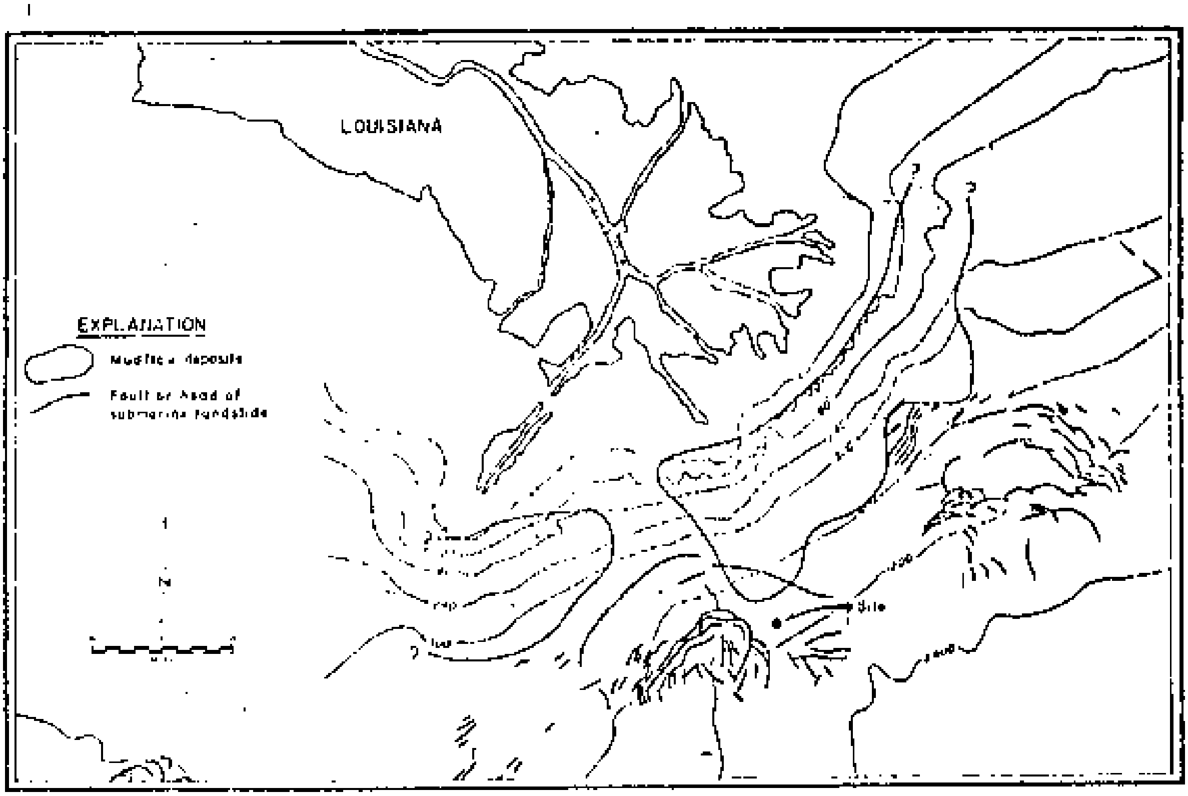

Figure 6: Deformational features offshore Mississippi Delta. All locations are approximate. (Deformational features based on references 12, 13 and 14)

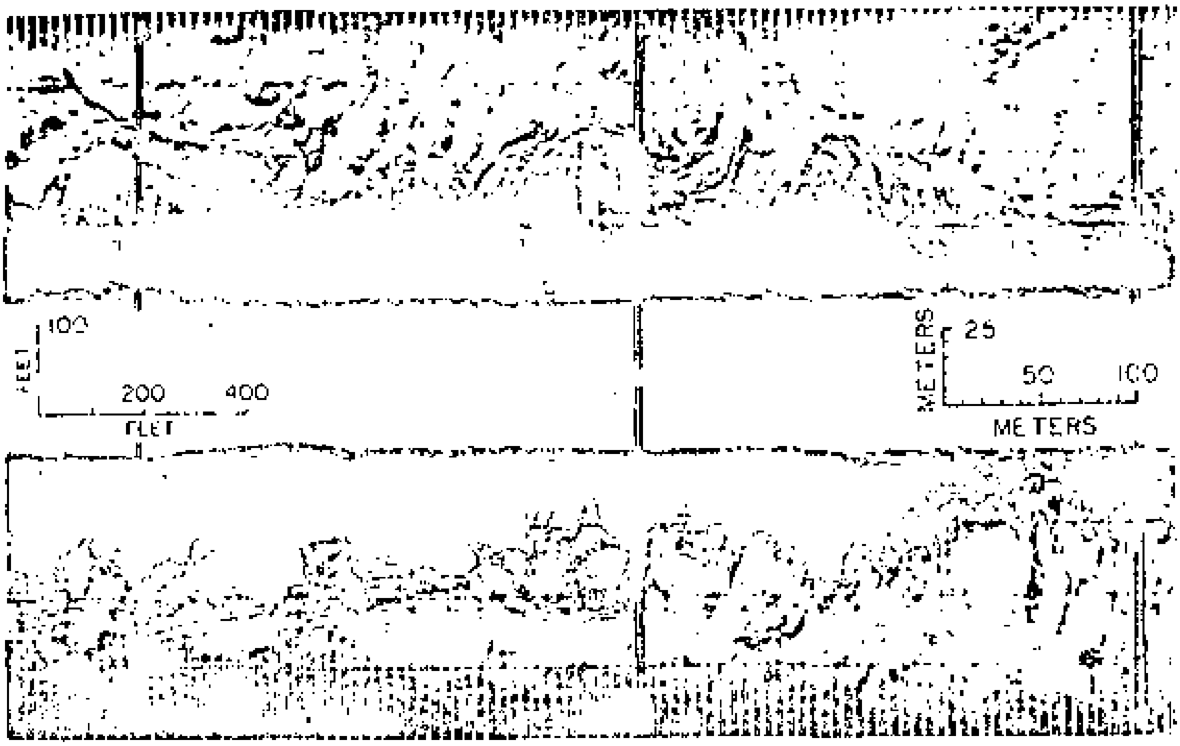

Figure 7: Side-scan sonar record showing surface of mudflow deposits offshore Mississippi Delta

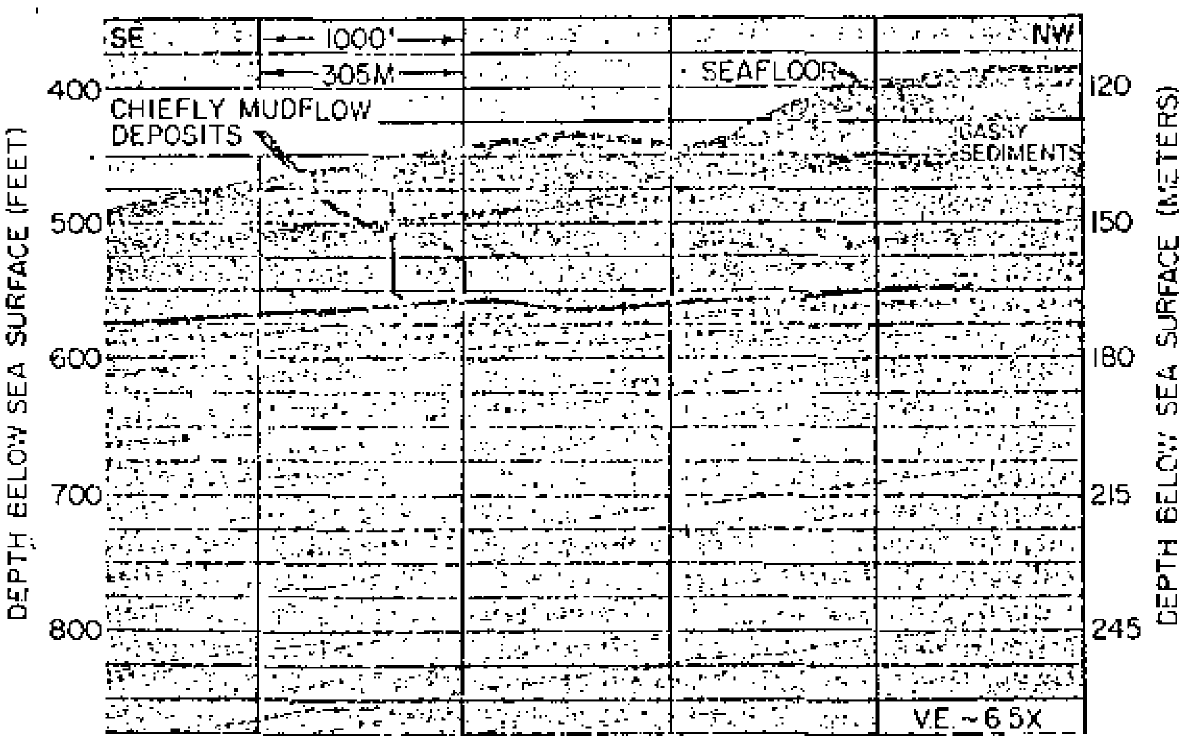

Figure 8: Boomer record showing mudflow deposits offshore Mississippi Delta. Note truncation of normal marine deposits at base of mudflow deposits

(40)

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PRINCIPAL POINTS FROM THE PRESENTATION

This paper was presented by Dr James Vernon, McClelland Engineers Inc., on behalf of Drs Ploessel and Campbell

Let us first consider the dimensions of the problems in the northern Gulf of Mexico. Production platforms are being designed for and installed on the upper slope in water depths of more than 1,000 ft. One of the most significant advances recently has been the installation of Shell's Cognac A platform, in about 1,030 ft of water, offshore the Mississippi Delta. The size of deep-water structures such as Cognac is enormous. Cognac's overall height above the sea floor is 1265 ft. This is more than twice as tall as the Washington Monument and slightly taller than the main body of the Empire State Building. The investment required for the design, construction and installation of such large structures can rise as high as several hundred million dollars. Such structures are major engineering works, and no one would today argue the need for a thorough geotechnical site assessment prior to foundation design. This is in sharp contrast to the practice in the late 1940s when offshore foundations were designed without the benefit of site investigation, whereby piles were merely driven to practical refusal with the biggest possible hammer. The need for site investigations soon became obvious.

Over the years, working in increasingly deeper water, the development of new structural design concepts and the growing awareness of the potential engineering significance of shallow geologic conditions have combined to increase the scope and complexity of foundation design and site considerations. It is now clear that comprehensive and multidisciplinary site assessments are required for the design of safe and economical offshore foundations. The increasing importance of adequate site assessment is underlined when the large capital investment required for the development of deep water sites is considered.

Three basic types of production structures are being installed, or considered for installation, in deep water sites in the northern Gulf of Mexico. Existing production platforms in the Gulf of Mexico are pile supported. Large piles driven into the sea floor provide the vertical support for the structure. Pile diameters for upper slope structures will range between 48 and 111 inches, depending on anticipated loading. The Cognac structure is an example of a pile-supported structure. It is supported by 24 piles of 84 inches diameter driven to about 450 ft below the sea floor.

Another structural design for deep water sites, the Exxon guyed tower, consists of a relatively slender steel structure of uniform cross section. The structure rests on a spike which can be expected to penetrate several tens of feet into the sea floor and is held in position by a set of symmetrically placed cables. The cables connect to either anchor piles or conventional drag type anchors through clump weights. The clump weights are designed to lift off the bottom only during the passage of very large waves, thus reducing the requirements for restraining force. The only existing guyed tower structure is a prototype installed by Exxon in about 300 ft of water in the Gulf of Mexico.

A third type of structure that is being considered for deep water sites is the tensioned leg platform. This is a floating structure that is held in place by cables attached to anchored piles or deadweight anchors that must sustain large tensile loads to minimise lateral movement due to wind, and wave lashing. The tension in the cables can be adjusted and maintained by winches. The similarity of this design to the semi-submersible type of exploration rig is obvious.

On what basis would one of these three types of structure be selected for installation on a given site. The answer is, on the basis of expected cost. The guyed tower and tensioned leg platform designs tend to become more economical than pile supported structures as water depth increases. This is because there is a possibility of recovery and use at additional sites, especially in the case of the tensioned leg platform, and because the amount of steel required is generally less than that required for pile supported structures. However, because pile supported structures obtain most of their stability from deep penetration piles, they may prove to be the most reliable structures in areas where the upper faulting tends to 100 ft or so of sediment which may experience downslope movement due to creep or mudflow activity. Thus, mitigating geologic factors could result in selection of an initially more costly design, which over the long run would be more reliable, and thus ultimately less costly, than another design.

How do a structure's foundations perform during its service life and what must be considered to predict this?

Foundation design considerations include basic properties of sediments, such as strength, unit weight, plasticity and gradation, anticipated environmental loads such as those due to winds, waves and currents, and seismic shaking, and, perhaps considered as a special group of environmental loads, the geologic factors such as active faults, mudflows, massive slides and so forth. Of course the relative importance of each factor will vary somewhat, depending on the design of the structure involved, but it is commonly the geologic factors that are least amenable to engineering analysis, and hence are of particular concern to the geotechnical engineer.

What are some of the geologic features on the outer shelf and upper slope of the northern Gulf of Mexico that are of potential engineering significance? Let us look at several examples. Comment will be restricted to active faults, slides and mud-flows. As one might expect, most of the 'action' seems to be in the vicinity of the Mississippi Delta and is related to the vast quantity of fine grained sediment found in this area. However, massive slide deposits and active faults are found further to the west, offshore Texas as well.

First, active faults. For our purposes, faults of two basic types are to be found in the northern Gulf of Mexico, deep seated faults which typically extend to depths of several thousand feet, and shallow faults which affect the upper few hundred feet. An example of a deep-seated fault is from the area near the shelf slope break south of Galveston, Texas. Such growth faults are common on the outer shelf and upper slope, and, as suggested by Lehner, probably result from slow basinward migration of deeply-buried salt. Seafloor relief in the vicinity of active faults [illegible] be as much as several tens of feet [illegible] average rate of movement over the last 80,000 years [illegible] one large fault has been 2-3 ft per thousand years.

Shallow reverse faults in the Mississippi Trough area exhibit very small throw but seafloor expression can be seen. Such faults have apparently been formed in response to slow downslope movement of the upper 200 ft or so of sediment.

Shallow slump faults commonly are arcuate in plan and typically flatten with depth. The exact origin of these faults is unknown, but they do indicate present or very recent downslope movement. The two types of shallow fault described may be restricted to slope areas underlain by thick, rapidly deposited fine-grained sediments. Mudflow deposits are seen in the vicinity of the Mississippi Delta on slopes as low as 1 percent.

The composite thickness of coalescent mud flows can be more than 100 ft, and, as one might expect, the surface of the mudflow deposits commonly appears ropey. Although little is known about however often movement occurs, rate of movement, or the details of triggering mechanisms, Coleman has recently presented evidence indicating that the movement of several thousand feet occurred sometime between two observations made about a year apart. It is suspected that individual flows probably move relatively rapidly. The loss of Shell's Platform B in in 1969 during the Hurricane Camille is believed to have been caused by a mudflow.

Massive slide deposits are also found on the upper slope in the northern Gulf of Mexico. One example is approximately 21 miles long and the slide deposits reach a maximum thickness of 500 ft. Slope failure occurred as a catastrophic event about 15,000 years ago following the deposition of deltaic sediments at the shelf edge during the late Pleistocene. Younger, late-Pleistocene, deltaic sediments have buried the slide deposit and the slide scar. Evidence of similar failures have been observed in the vicinity of the Mississippi Delta: one 200 ft-high buried scarp at the head of a large, probably late-Pleistocene slide at the shelf edge clearly demonstrates a catastrophic origin.

Why are relatively old slides, for example, of concern to engineers? The question invariably arises, 'Will it happen again?' 'Is the slide completely stabilised?' Without detailed and regional studies, we cannot begin to answer such questions with confidence. At this point relatively little is known about offshore geologic processes such as faulting, massive sliding, and mudflow activity. However, it is likely that rates, magnitudes and frequencies will be so low in many cases as to be insignificant in an engineering context, since offshore structures typically have a design life of only a few tens of years. On the other hand, it does not take much imagination to envision the potential effects of faulting, massive sliding, or mudflow activity on structures not designed or sited with them in mind. For example, consider a 60-ft thick mudflow imparting an abnormal lateral load to a pile-supported structure. Assume the rate of movement to be of the order of 10 to 20 ft/hr. The load due to the mudflow alone would be about twice the anticipated maximum due to winds, waves and currents combined. Although from this example it is clear that geologic processes could induce significant abnormal stresses on production structures, the presence of abnormal stress in itself does not necessarily mean that safe structures cannot economically be constructed. For example, the bending stresses imparted to a pile by moving sediment can often be compensated for by using thicker walled, and thus stiffer, piles. Of course, such a solution will result in greater cost to design, build and install a platform, but it is essential that those in industry who are acting in an advisory capacity do not lose their perspective on the ability of engineers to build safe and economical structures for many sites affected by adverse geologic conditions.

I shall now say a few words on recent developments in geotechnical site assessments.

Additional methods of site investigation, that is soil sampling and engineering analyses, based on the results of sample testing, have never enabled the identification and assessment of geologic features such as active faults, mudflows, and slides. Thus, perhaps the most significant single development is the now routine procedure of using soil-boring data in conjunction with high resolution geophysical data and effectively integrating this information through the use of multi-disciplinary teams of geologists, geophysicists and engineers. Admittedly, even with this approach fully reliable assessments of offshore geologic processes for engineering purposes cannot always be made, but this situation is rapidly changing as these people work more closely with one another, and as knowledge of these processes increases.

Another recent development is the realisation that standard block surveys are not always adequate to assess the megafeatures found on the upper slope. For example, a recent survey covering twelve leased blocks — more than 100 square miles, with a line spacing of 100 ft by 3,000 ft — was done solely to assess overall stability of a single proposed platform site off the Mississippi Delta. I am certainly not suggesting that all upper slope sites need such extensive survey coverage for adequate assessment, but it is clear that the requirements for a site investigation should be determined by geologic conditions rather than by arbitrary location of block boundaries. It is likely that most site studies for upper slope areas will require that some regional survey lines be extended outside the site proper to adequately assess the significance of nearby features or conditions, especially those up slope and down slope from the site.

A third development is the use of repeat, or time-lapse high resolution surveys whereby specific areas are resurveyed a number of times over a period of several years. At least one such programme is under way in the Mississippi Delta area and will help to determine mudflow activity and the relationship of deformational processes to large storms.

How does all this relate to the known geologic conditions? Let us look at a specific site which has required investigation. The site is in deep water, in the depocentre of off the mouth of the Mississippi River. A limited area site survey covering one or two square miles would reveal no apparently hazardous geologic conditions. However, upslope is an area of mud flow deposits. To the west, upslope and downslope, there are faults, slump faults, and varied slide scars. It is obvious that these regional features must be evaluated with respect to the site. The approach used here is to employ high resolution acoustic techniques to attempt to determine the nature, areal extent, history, and present activity if any of these features. Some are still being monitored by repeated surveys to determine whether movement is taking place at present.

In the Gulf of Mexico most sites above the break in slope can be adequately investigated by surveys that cover only a few square miles. The area requiring regional surveys extends from about the shelf break into water deeper than 6,000 ft, covers hundreds of square miles and extends up to the shelf break and many miles downslope.

In summary, development of upper slope sites will be a challenge. Very large and costly production structures are now being installed, or proposed for installation, on upper slope sites in the northern Gulf of Mexico. Active faults, mudflows and massive slide deposits are known to occur in many upper slope areas in the northern Gulf of Mexico, especially offshore the Mississippi Delta. It is essential that multi-disciplinary regional site investigation be carried out prior to design and construction of production platforms to assess the potential engineering significance of these and other geologic factors. However, the presence of adverse geologic conditions does not necessarily preclude the construction of safe production structures.

DISCUSSION

E Oele (Rijks Geologisch Dienst): Dr Vernon has pointed out that there is quite an extensive area where no regional investigations are required. Is this because there are so many data already available, or is the situation so homogeneous that there is no necessity to do any investigation? There must have been quite some change during the Holocene sea-level rise in the area. In earlier periods the area was land, and there would have been quite some differences in the various subareas.

The history from late Pleistocene and early Holocene may mean that the area is less homogeneous than it is perhaps thought to be at the moment.

Chairman: This question refers to the comment that there is a requirement for a regional survey to be carried out on the slope, but not for the inshore area. Is this because sufficient data already exists on that inshore area?

Dr Vernon: That is part of it, but the main problem with the break in slope is that it was an area of rapid deposition during the previous low stands of sea level, and sediments in those areas are less stable than on the shelf.

Dr Oele: During the late Pleistocene the shelf was above sea level, and then it underwent transgression to the Holocene sea level. This process so much reworked the shelf area that regional information must also be available, additional to site investigations.

Chairman: Some of the earlier papers have shown the variability one can get on the shelf area. But perhaps the problem that has been identified in the last paper is the very gross difficulties that arise with slides on the slope, and which are obviously quite an enormous hazard. It is partly the degree of emphasis, rather than a situation of no requirement for survey on the shelf itself.

A R Biddle (Shell International Petroleum): Is Dr Vernon aware of any work being done to study the mechanism of deformation at growth faults, particularly where such faults are shown from seismic records to be very near to outcropping on the seabed surface?

He mentioned a movement of between 2 and 3 ft per 1,000 years. Is that based on a throw of the fault of 160 to 240 ft divided by 80,000 years, or is it known to be a progressive movement at the surface?

Dr Vernon: I do not know on what the estimate is based. I do not know how they arrived at those numbers.

B McClelland (McClelland Engineers Inc): There are current measurements being made of active faults, not offshore but onshore, in the Gulf Coastal Plain, faults which have the same geologic character as those referred to in the paper. These are of concern to the siting of various structures that might, by virtue of their placement directly in the position of the fault scarp, be damaged by current movement.

Specific measurements of additional movement have taken place over the last two or three decades and are a matter of record.

As to the specific information on rate of growth given in the paper, both Dr Vernon and I would have to refer to the authors for their sources on that. I recall direct observations of pavement breaks, for example, on the Gulf Coast Plain, where vertical displacements of the order of 12 or 18 inches within the last decade or two are visible to the eye eye.

N G T Fannin (Institute of Geological Sciences): It is interesting to draw a parallel between conditions on the Mississippi Delta and conditions in the North Sea and on the edge of the British shelf. They are quite different environments. The Mississippi Delta area has not been glaciated, but a lot of the collapse failures on the edge of the shelf there are induced by the very high rate of sedimentation coming out from the Mississippi, and creating loading at the top of the slope.

It is interesting to compare this with the British continental shelf where there is a glaciated terrain.

The significant point is that during the Pleistocene, particularly during the late Pleistocene, we believe that very large masses of material were dumped over the edge of the continental shelf by glacial processes. The British Isles have largely been stripped of their Tertiary cover in the northern part. That sediment has been swept out into the North Sea and over the western shelf of Britain and dumped over the edge of the slope. Thus in a glaciated terrain much the same sort of conditions have been produced as in a delta situation where large masses of material have been dumped very rapidly, and often in an unstable condition.

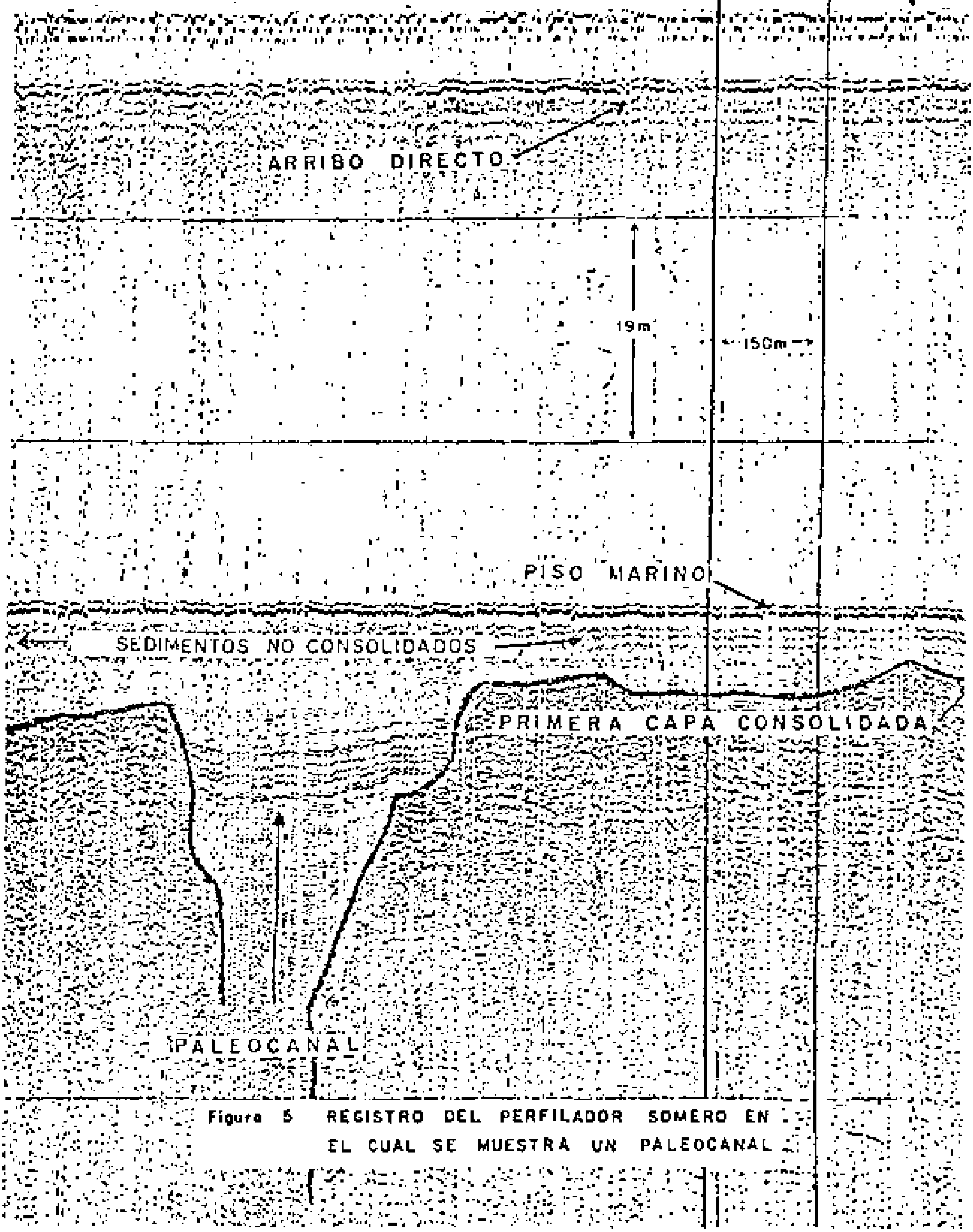
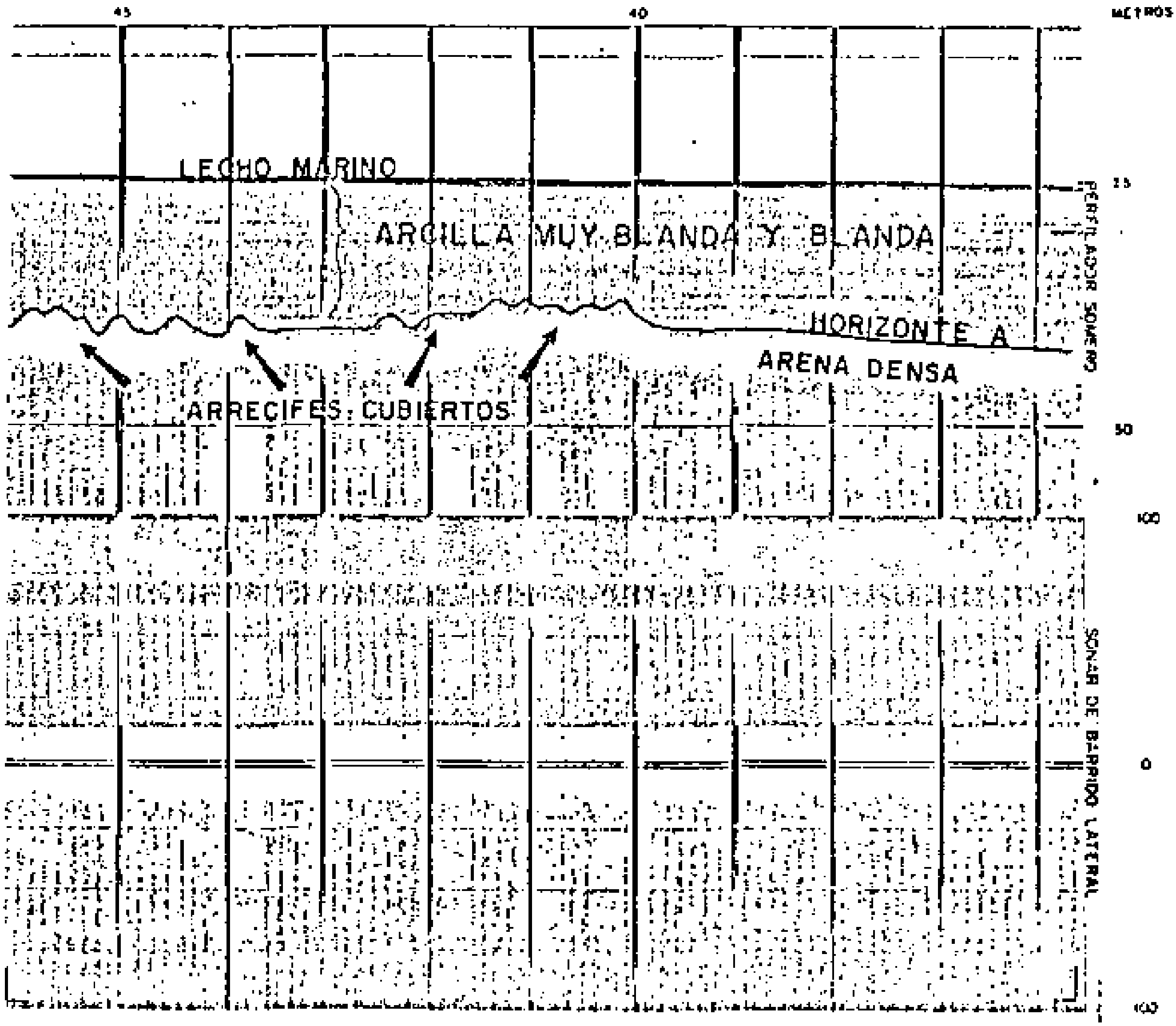


Figura 5 REGISTRO DEL PERFILADOR SOMERO EN EL CUAL SE MUESTRA UN PALEOCANAL

419



LINEA CENTRAL DEL PERFILADOR SOMERO DE ABKATUN 1A A ABKATUN 4

FIGURA 12

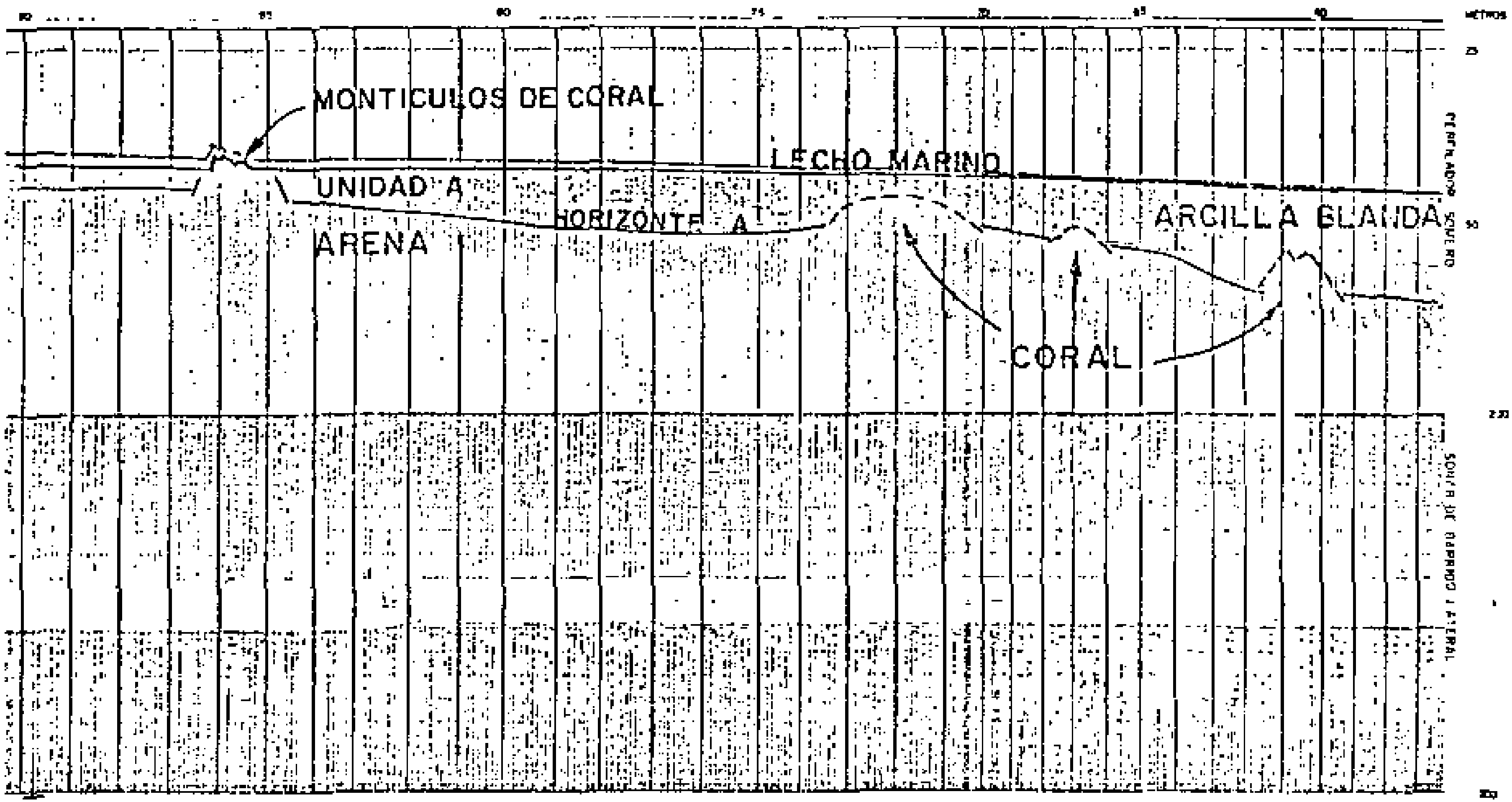
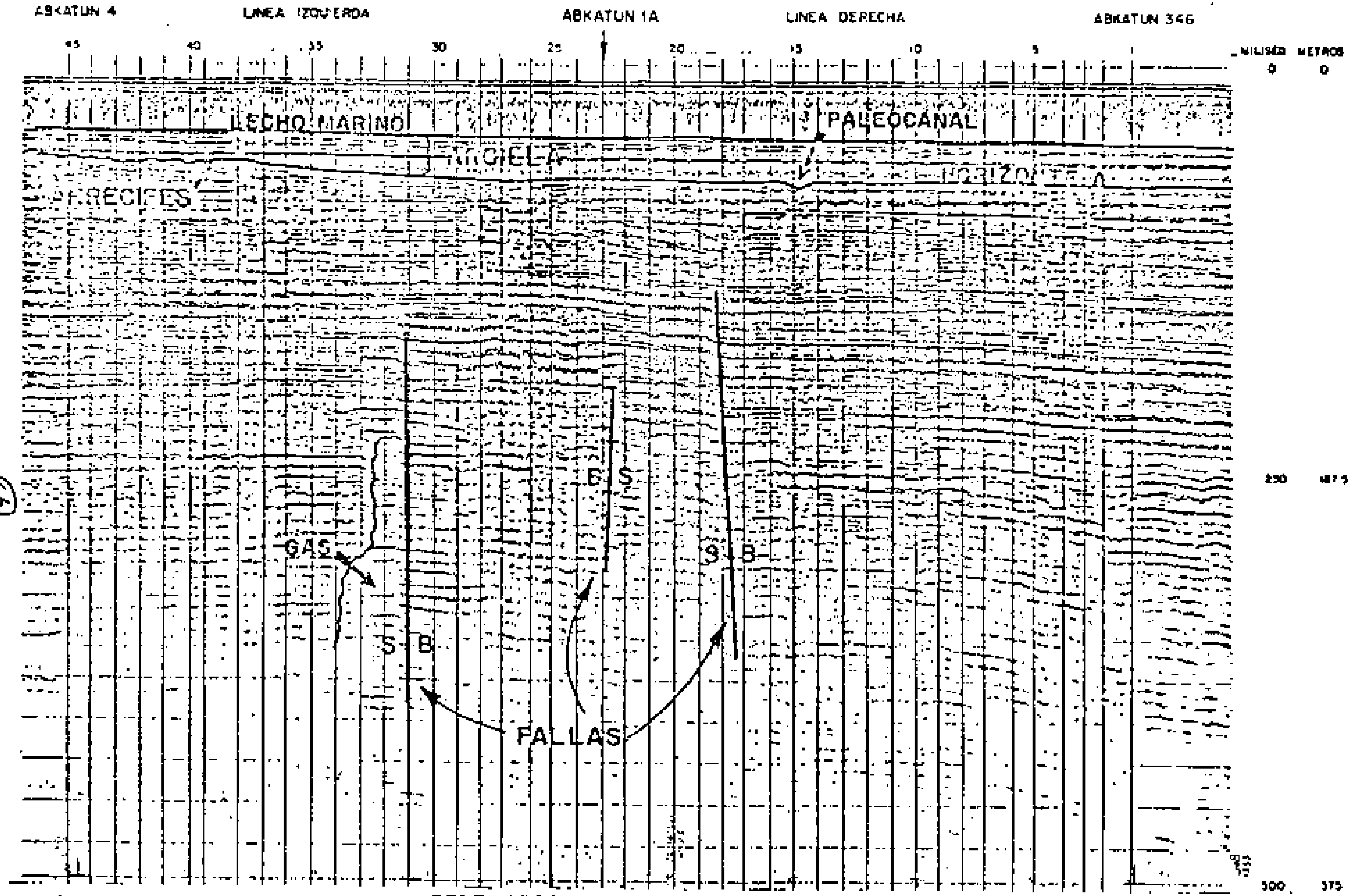


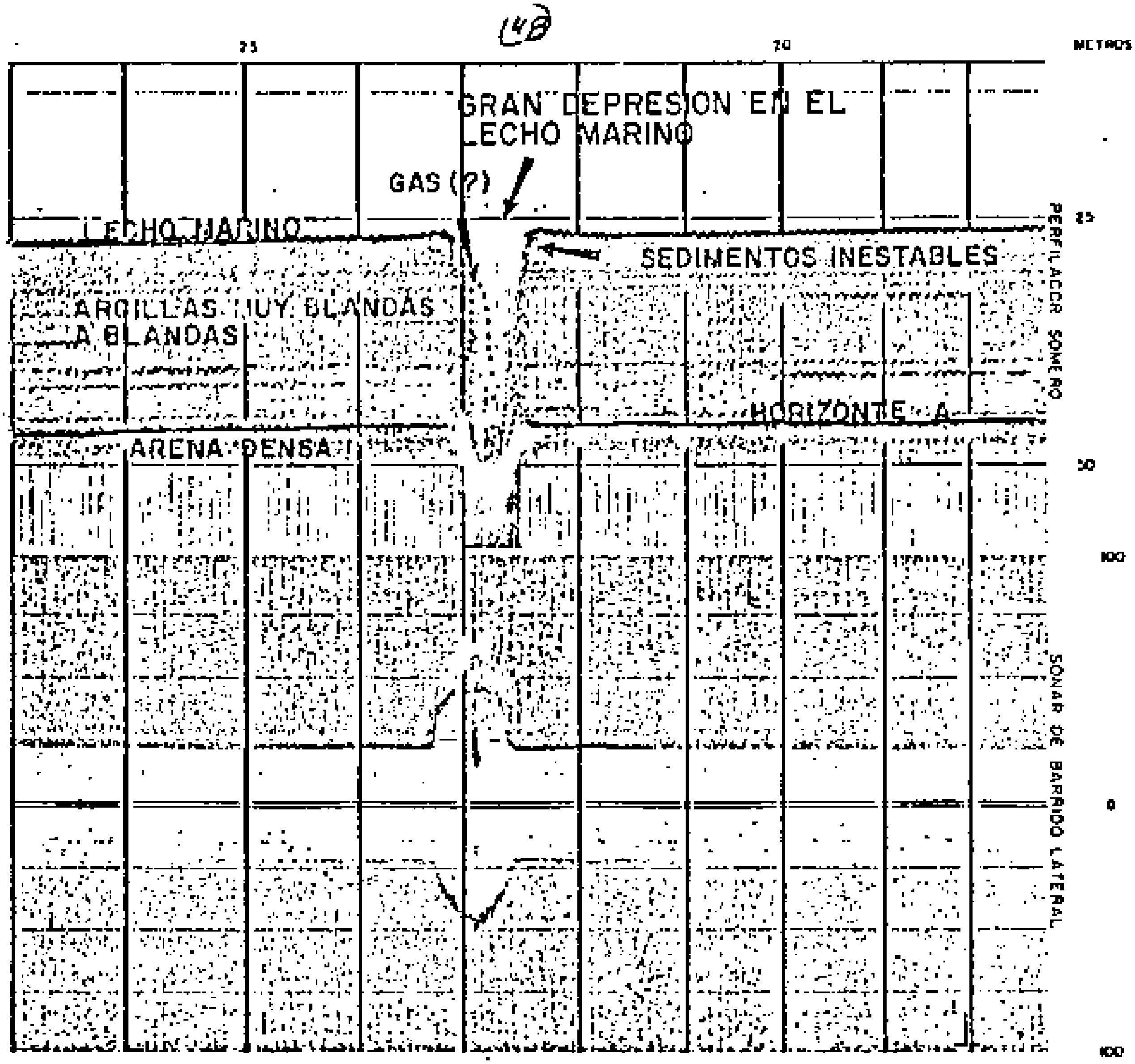
FIG. 14 LINEA KA - CENTRAL DEL PERFILADOR SOMERO



PERFILADOR PROFUNDO DE ALTA FRECUENCIA

LINEA DERECHA DE ABKATUN 1A A ABKATUN 346 Y LINEA IZQUIERDA DE ABKATUN 1A A ABKATUN 4

FIGURA 13



LINEA DERECHA DE ABKATUN 1A A ABKATUN 5 DEL PERFILADOR SOMERO

FIGURA 14

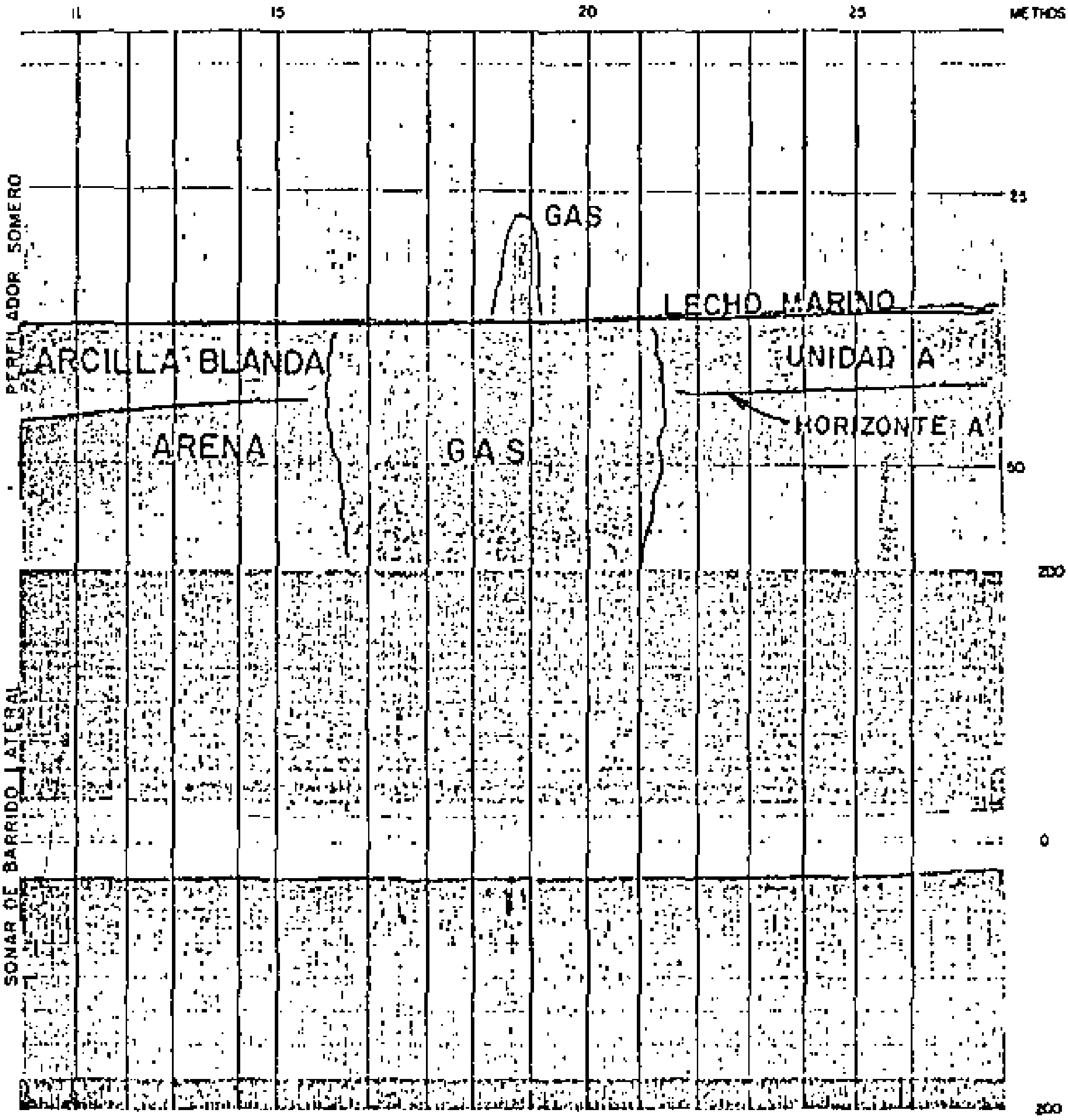


FIG. 15 LINEA ACN - CENTRAL DEL PERFILADOR SOMERO

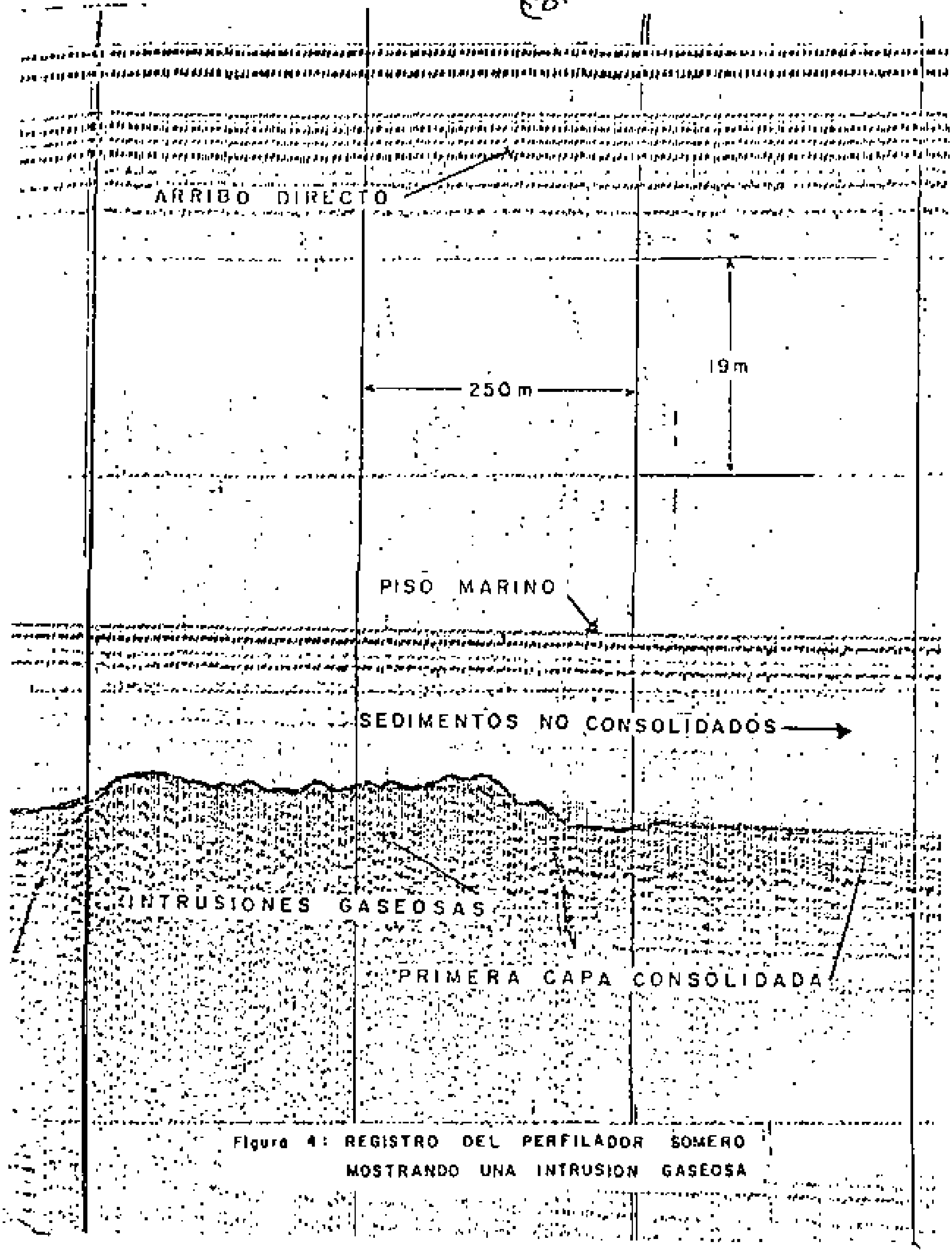
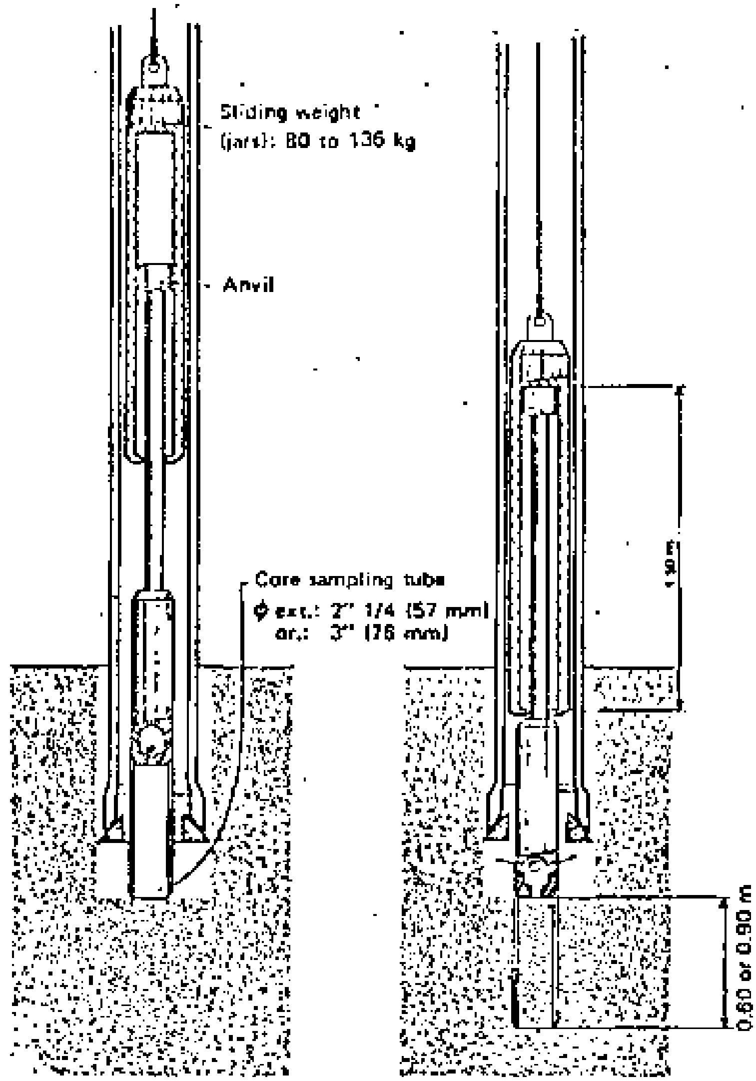


Figura 4: REGISTRO DEL PERFILADOR SOMERO MOSTRANDO UNA INTRUSION GASEOSA

(57)

- Inestabilidad del Fondo Marino
  - Deslizamiento de Taludes
    - Arcillas blandas pendientes de 1°
    - Arenas y gravas pendientes de 25°
  - Erosión ó Transporte de los Suelos
    - Corrientes Marinas
    - Tormentas
    - Variación de la Marea


Sliding weight
(jars): 80 to 136 kg
Anvil
Core sampling tube
ϕ ext.: 2" 1/4 (57 mm)
or.: 3" (76 mm)
1.50 m
0.60 or 0.90 m

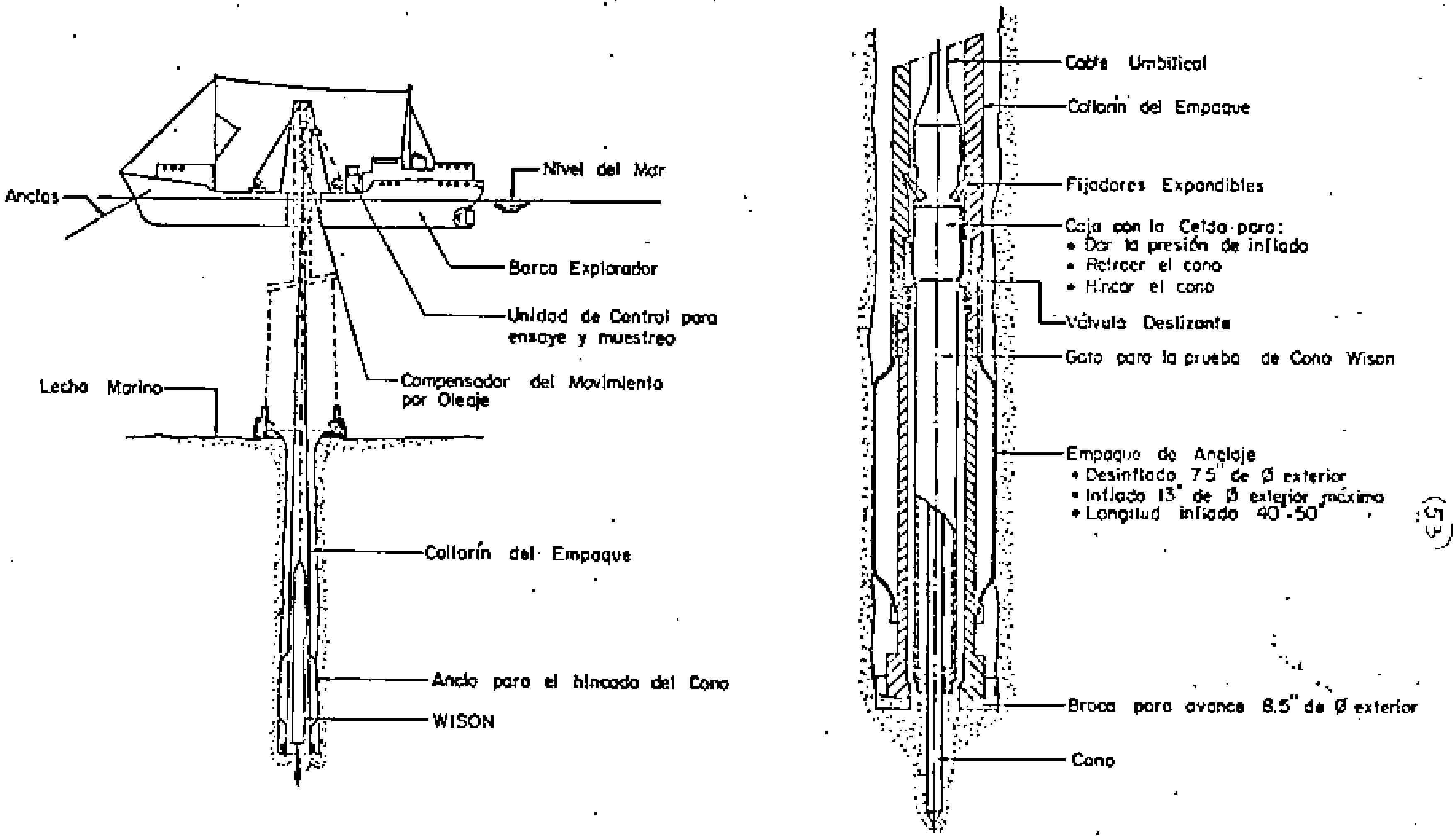


Fig. 3 SISTEMA WISON PARA PRUEBAS DE RESISTENCIA A LA PENETRACION CON CONO HOLANDES

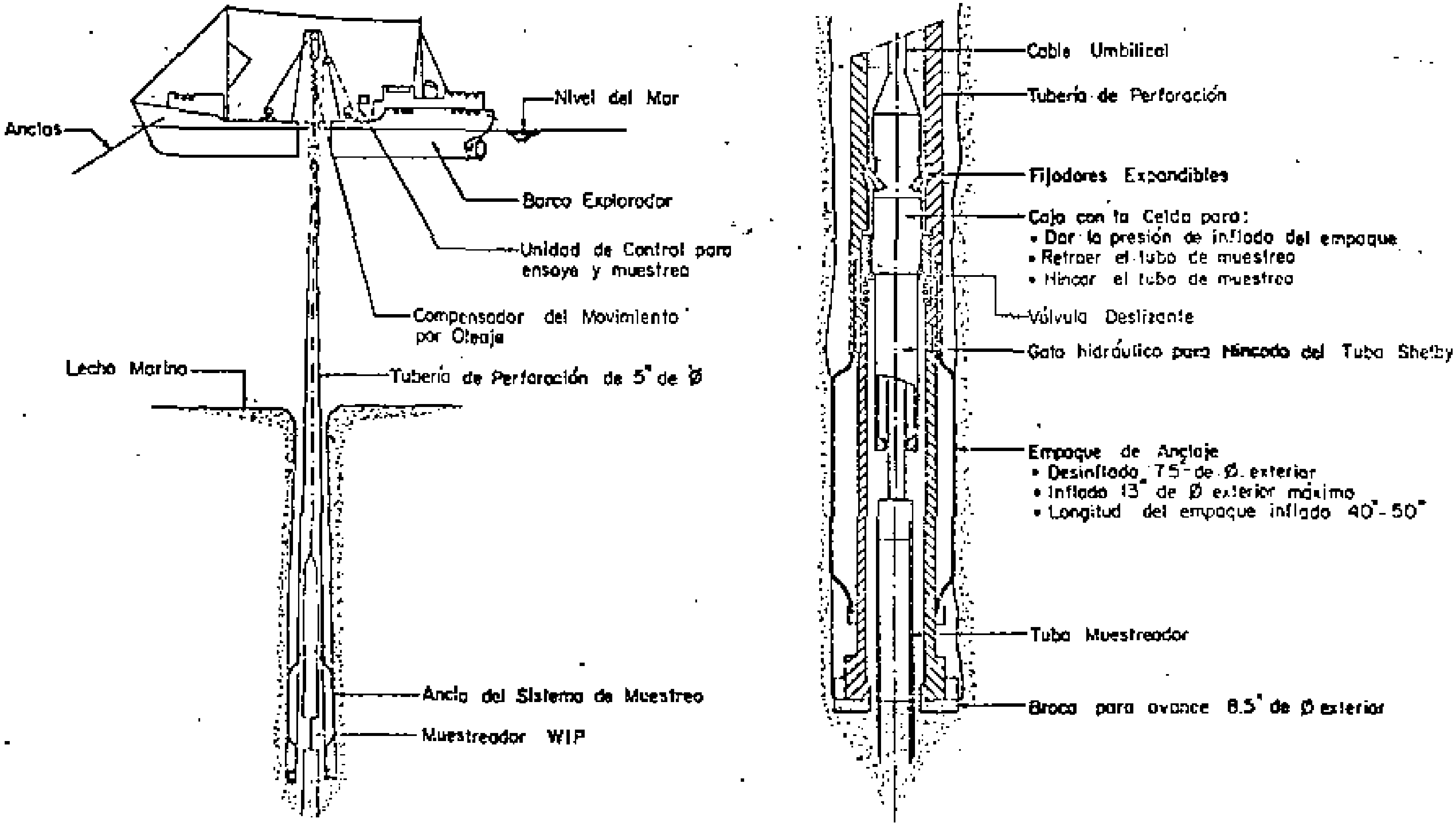
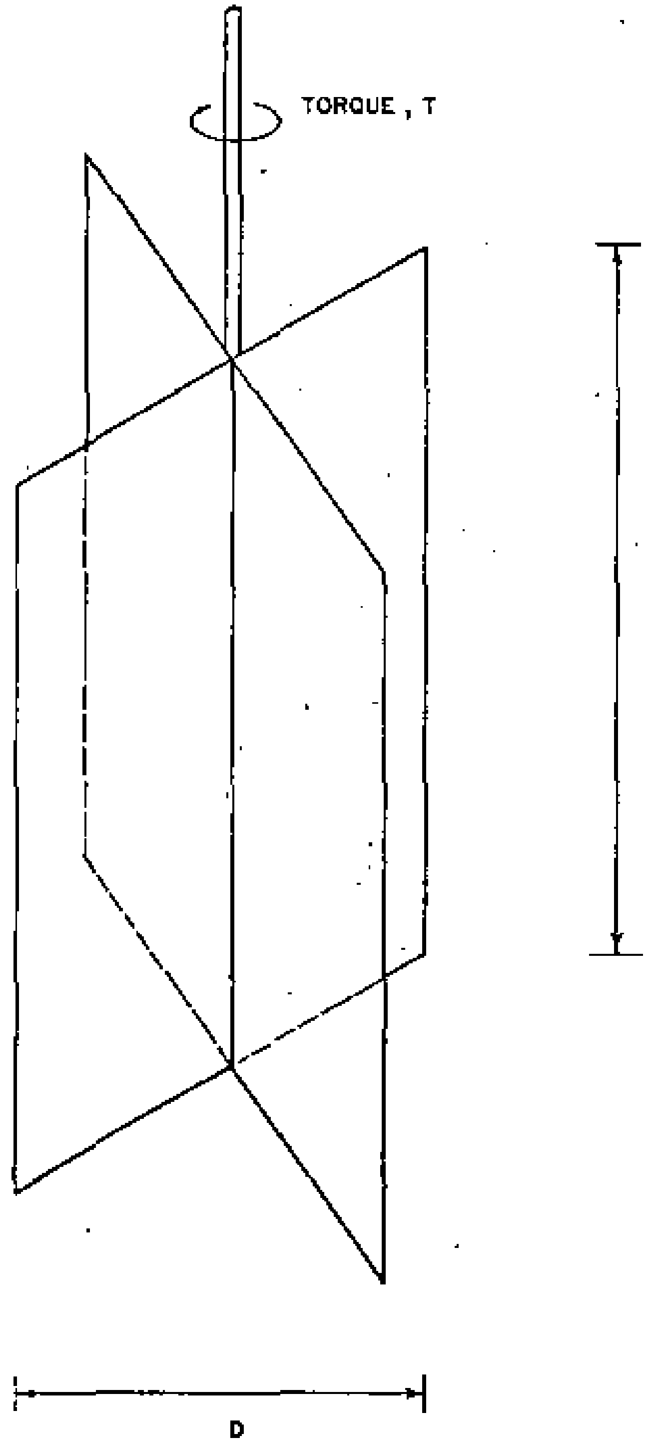


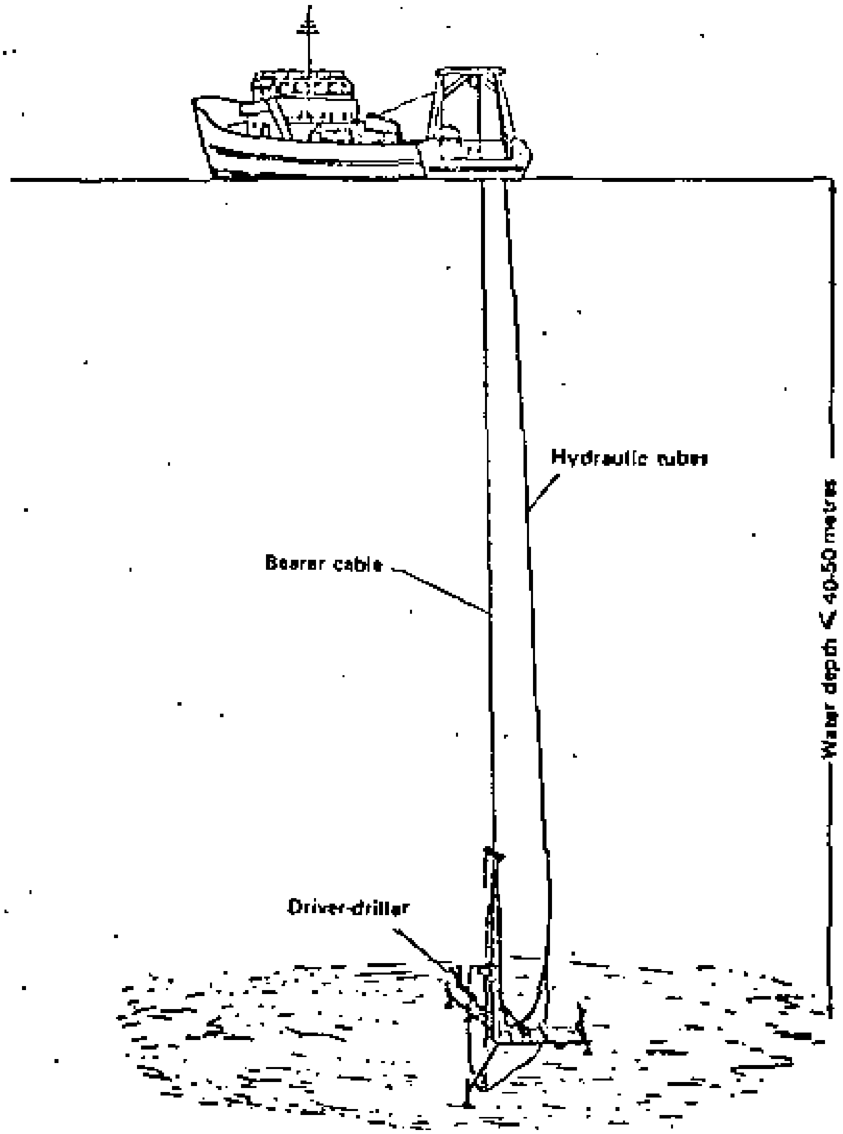
Fig. 2 SISTEMA DE MUESTREO INALTERADO WIP

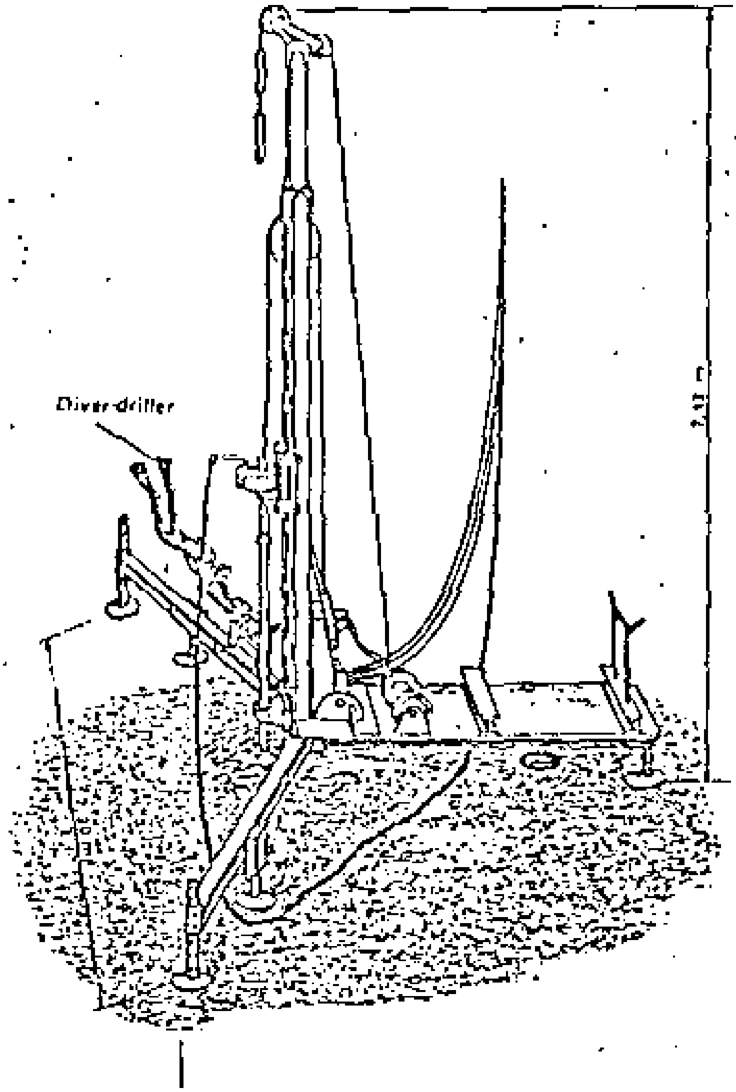
(55)

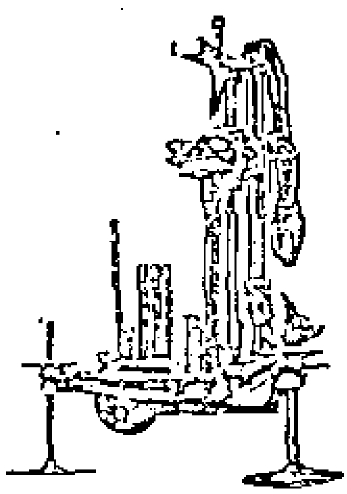
Veleta

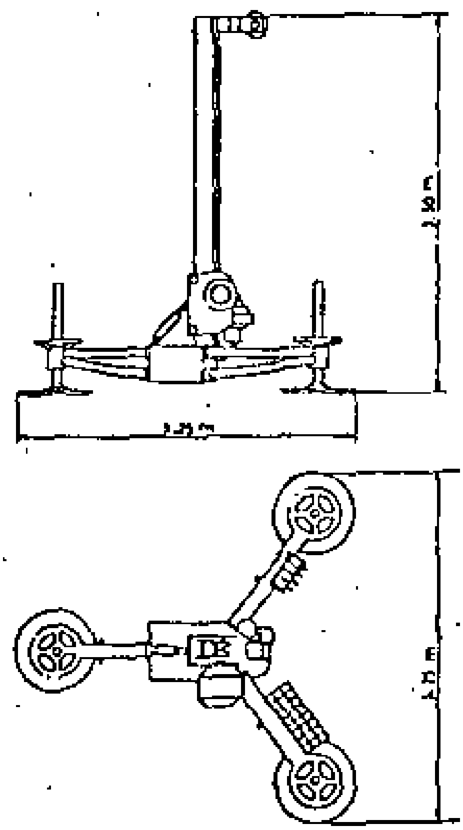


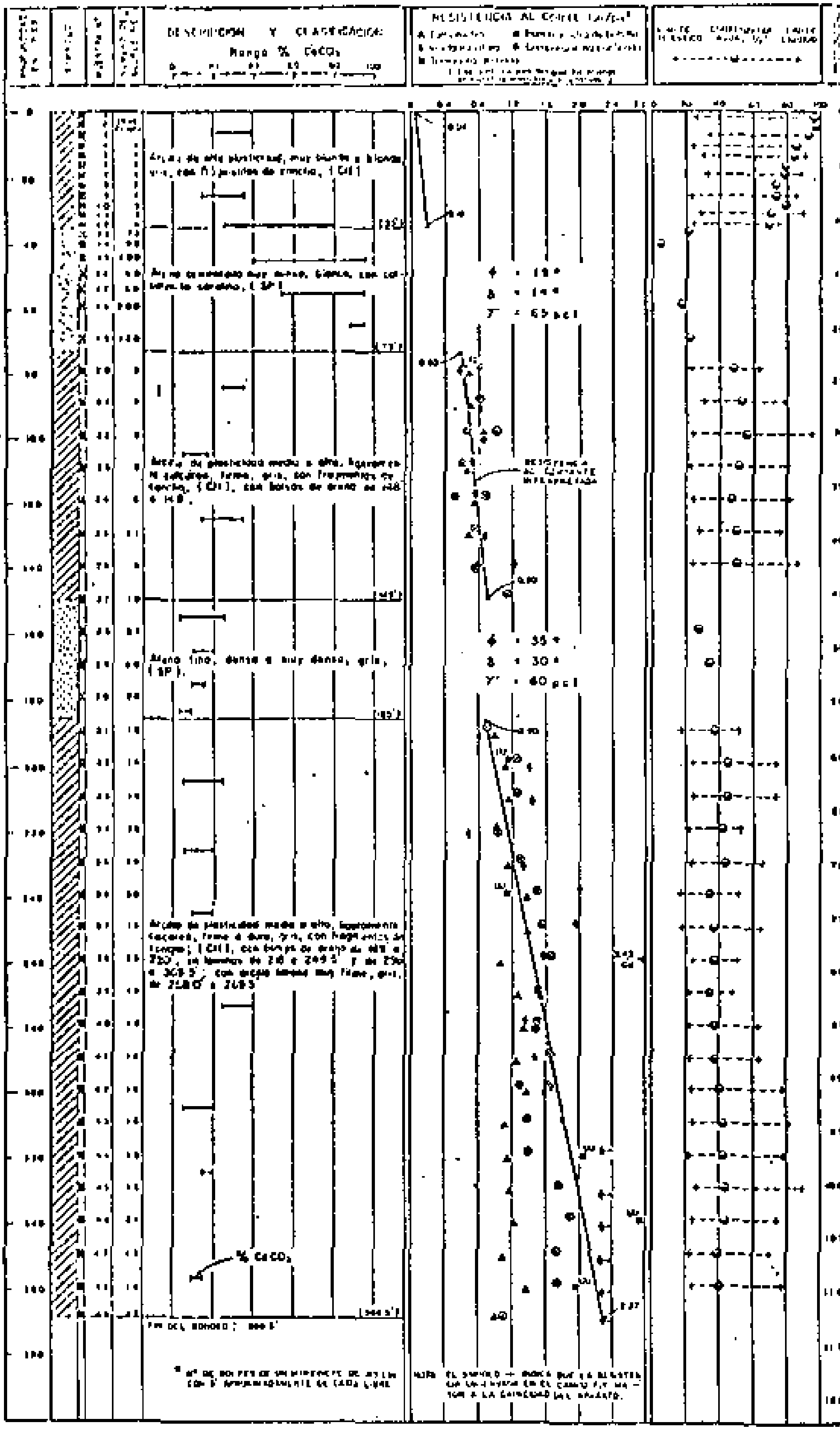
SEMINARIO INTERNACIONAL
DE
INGENIERIA OCEANICA


Hydraulic tubes
Bearer cable
Driver-driller
Water depth ≤ 40-50 metres


Diver-driller







REGISTRO DE SONDEO Y RESULTADOS DE PRUEBAS

KU - 27 ; BAHIA DE CAMPECHE

GOLFO DE MEXICO

17 The application of laboratory and in-situ data to the design of deep foundations

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This paper describes the application of in-situ and laboratory test results to the design of deep foundations. Whilst the methods described in the paper are applicable to many types of foundation, most emphasis has been placed on open-ended steel pipe piles installed by driving. This type of foundation is presently the most commonly used by the offshore industry and its design relies on a wide range of soil parameters.

The driven pile foundations of an offshore structure must be designed to satisfy the following criteria:

a) The piles and pile groups must have sufficient capacity to resist maximum anticipated loads with adequate factors of safety.
b) The piles and pile groups must have acceptable load deflection characteristics in all modes for the anticipated loading conditions.
c) Pile steel and grout stresses must remain within allowable limits under extreme loading conditions.
d) The available pile driving plant must be able to install the piles to their required penetration without overstressing.

The foundation design is based upon soil parameters derived from in-situ tests and offshore and onshore laboratory tests. Each aspect of design, should be analysed using at least two and preferably more methods, each based on different soil parameters. This procedure allows the relevance of any design method to the prevailing soil conditions to be assessed and increases the reliability of the final design.

It should be noted that soil testing and foundation engineering are relatively inexpensive when compared to the total cost of an offshore site investigation. When viewed in terms of the total cost of a platform their expense becomes even less significant. Generally time and money spent on a rigorous engineering analysis is more than justified by the increase in foundation reliability and/or the consequent reduction in foundation construction and installation costs.

IN-SITU DATA

In-situ testing methods available for use offshore include the pressuremeter, the down-the-hole vane and the electric cone penetrometer. The most widely used test is the cone penetration test (CPT) and only the results of this in-situ test applied to pile design are considered here.

There are a number of versions of two basic types of CPT apparatus, seabed penetrometers and down-the-hole penetrometers. With seabed penetrometers the cone and its rods are jacked into the seabed without the need for making a borehole. To use a down-the-hole penetrometer a borehole must first be made to the required depth, the penetrometer is lowered to the bottom of the drillpipe and then forced into the soil at the base of the borehole. Detailed information concerning these penetrometer types may be found in the technical literature.[1,2]

The two types of CPT apparatus described above provide the same soils data. They each measure the resistance on the tip and sleeve of the cone as it is forced into the soil. These are termed the cone resistance (q_c) and sleeve friction (f) respectively. These data may be used directly for many aspects of design.

OFFSHORE LABORATORY TESTING

On most offshore investigations a certain amount of laboratory testing is performed on location, see table 1. The reasons for performing this work are:

a) to check on the quality of samples recovered
b) to obtain soil parameters for a preliminary engineering evaluation of the site. The purpose of the preliminary engineering analysis is to ensure that a sufficient number of boreholes to an adequate depth have been performed before the drilling vessel leaves location. This is a very necessary precaution in view of the high cost of mobilising an off-

shore drilling vessel and the relatively short weather window in some parts of the world.

APPLICATION OF OFFSHORE DATA TO PRELIMINARY DESIGN

From the results of the in-situ and laboratory tests performed offshore, preliminary assessments of the following design aspects can be made:

a) axial pile capacity — generally preliminary axial capacity calculations are based on the API and Lambda methods in clay and the API and CPT method in sand;[3,4,5]

b) lateral pile capacity — it is possible to obtain some indication of the lateral pile capacity and load deflection characteristics, providing a previous parameter study has been undertaken. In most cases this is not necessary, since normally the lateral loads on the piles are not finalised until a fairly late stage in the structural design of the platform;

c) pile drivability — soil resistance at time of driving may be determined from methods related to the cone resistance, calculated static skin friction and undrained shear strength.

Also at this stage of the investigation it is possible to determine the relationship between the cone resistance and undrained shear strength of a cohesive soil. This relationship is given by the formula:

$$\tau_u = \frac{q_c}{N_k} \qquad (1)$$

where N_k is an empirical cone factor determined by a correlation between cone resistance and shear strength measured by other means in corresponding strata. The value of N_k usually lies between 15 and 20 for North Sea clays. It should be noted that for typical North Sea clays it is not the practice of the authors' company to subtract the overburden pressure from the cone resistance in equation (1).

ONSHORE LABORATORY TESTING

As soon as possible after the return of the samples, the onshore laboratory testing is commenced. The majority of soil parameters used in design are obtained from the results of these laboratory tests. Table 1 shows the soil parameters most commonly used in design and the test methods from which these results are obtained.

SOIL PROFILE FOR BASIC DESIGN

The first stage of the geotechnical design work for any structure is the preparation of the design shear strength profile. Great care must be exercised in the preparation of the design shear strength profile, since the validity of all design calculations which follow depend on this initial assessment. In many instances the computations for pile capacity and drivability are made independently for each borehole at a location or alternatively for average and upper and lower bound soil profiles. Also different profiles may be used for different aspects of design.

The basic design soil profile comprises soil parameters corresponding to undrained failure conditions in cohesive soil and to drained failure conditions in cohesionless soil. However, for each strata whether clay, sand or intermediate material,

Table 1 : Soil Parameters derived from Laboratory Testing

SOIL PARAMETER	SYMBOL	TEST
—	—	visual description*
Bulk density Submerged density Moisture content	γ γ' M_c	density test* moisture content test*
Undrained Shear Strength	τ_u	unconsolidated undrained triaxial test*; pocket penetrometer*; torvane*; fall cone*; consolidated undrained triaxial test (if in-situ stresses can be estimated)
Remoulded Shear Strength	C_r	plasticity index; moisture content; remoulded triaxial test
Horizontal and Vertical Elastic Soil Moduli	E_h E_v	unconsolidated undrained triaxial test (cohesive soil)* consolidated drained triaxial test (cohesionless soil)
Strain required to mobilise 50% of maximum soil shear strength (cohesive soils only)	ϵ_{50}	unconsolidated undrained triaxial test* consolidated undrained triaxial test
Overconsolidation Radio Coefficient of Earth Pressure at Rest	OCR k_o	oedometer test; unconsolidated undrained triaxial test* consolidated undrained triaxial test; plasticity index
Effective Shear Strength parameters	c' ϕ'	consolidated undrained triaxial test; consolidated drained triaxial test; plasticity index; particle size distribution

*denotes tests performed both onshore and offshore

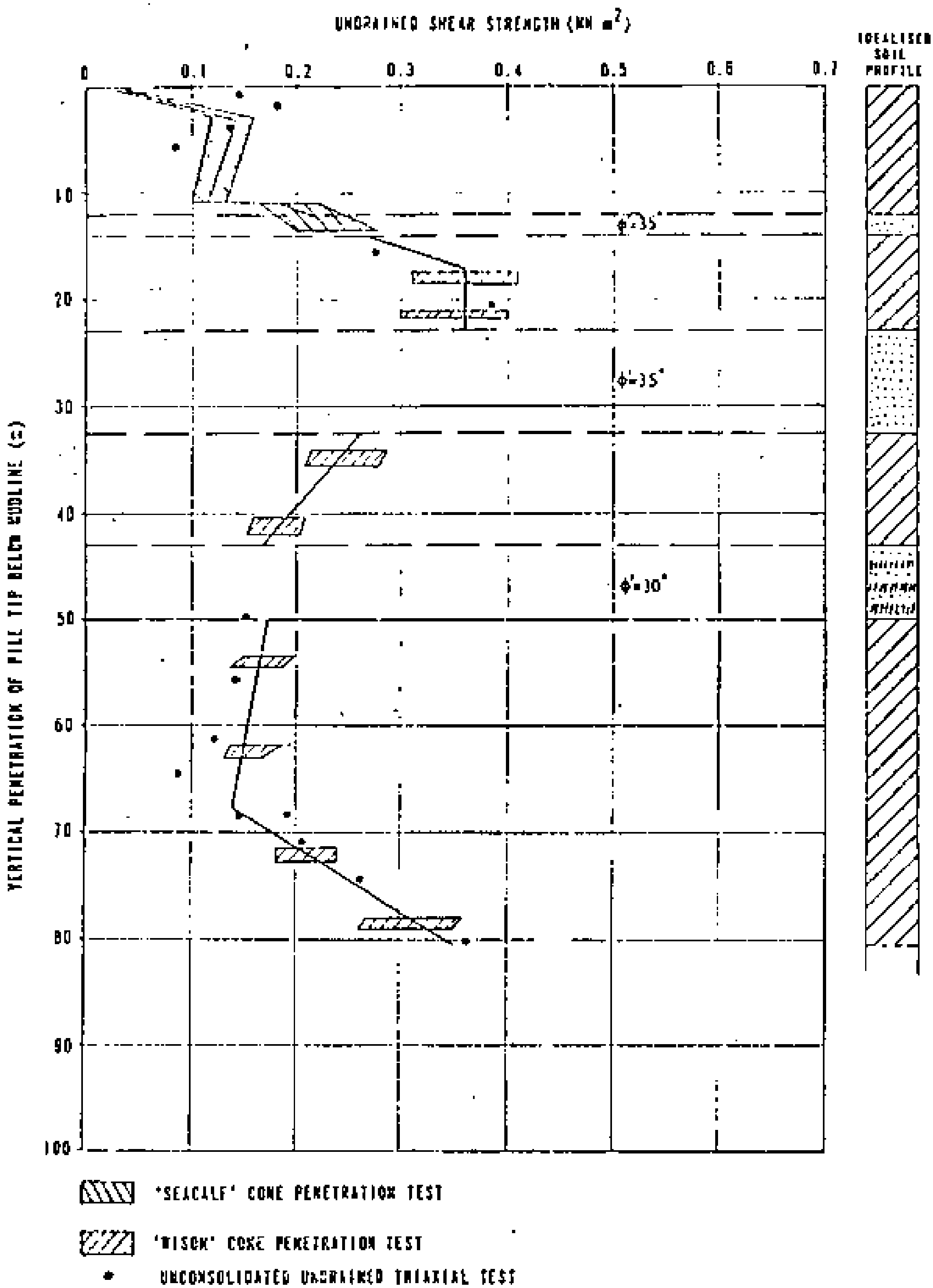


Figure 1 : Typical soil profile for basic design

the authors' company always prepares secondary soil profiles based on effective stress parameters and CPT results.

Frequently, a more complete picture of the soil conditions at a site can be obtained by combining the profiles described above. For example, the effective stress parameters obtained from consolidated drained or undrained triaxial tests may be used in an undrained shear strength profile, provided the in-situ stress conditions can be reasonably estimated. Also extensive use is made of the application of CPT data to undrained shear strength in a cohesive soil.

For the determination of design profiles in all types of soil, all available data, both from in-situ and laboratory testing should be considered. Ideally at least three boreholes should be drilled at the site of an important structure or complex of structures. This not only enables an assessment to be made of the lateral variation of soil conditions but also allows cross correlation between boreholes to determine if anomalous data in a particular borehole are valid or not.

A typical North Sea design soil profile which has been prepared from in-situ and laboratory test results is shown in figure 1. This is a good example of the way in which data from the methods and apparatus described earlier are combined to form the final profile.

Cohesive soil

The basic design profile for a cohesive soil will include the variation with depth of:

a) the undrained shear strength
b) the unit weight or density
c) the moisture content

The majority of the initial engineering analyses are prepared using the data from this profile. At a later stage more sophisticated analyses will be performed using soil parameters obtained from a supplementary soil profile including the results of tests for plasticity index, effective stress parameters and consolidation data. The application of these soil parameters to geotechnical design are discussed later in this paper.

Cohesionless soil

The basic design profile for a cohesionless soil will include the variation with depth of:

a) the effective angle of internal friction, ϕ'
b) the unit weight or density
c) the moisture content

Many methods exist for the determination of ϕ' in a sand. Within the authors' company most weight is given to the determination of this parameter from the results of consolidated drained triaxial tests and CPTs.

AXIAL PILE CAPACITY

Referring to figure 2, it can be seen that the ultimate static capacity of an open-ended pipe pile is given by:

$$Q = \Sigma f_s A_s + \Sigma f_i A_i + q_p A_w \quad (2)$$

where

Q = ultimate static capacity
f_s = unit shaft friction (outside pile)
A_s = shaft area of pile (outside)
$\Sigma f_s A_s$ = accumulated outside skin friction
f_i = unit shaft friction (inside pile)
$\Sigma f_i A_i$ = accumulated friction between the inside surface of the pile shaft and the column of soil inside the pile
q_p = unit end bearing capacity
A_w = cross-sectional area of steel wall at toe of pile

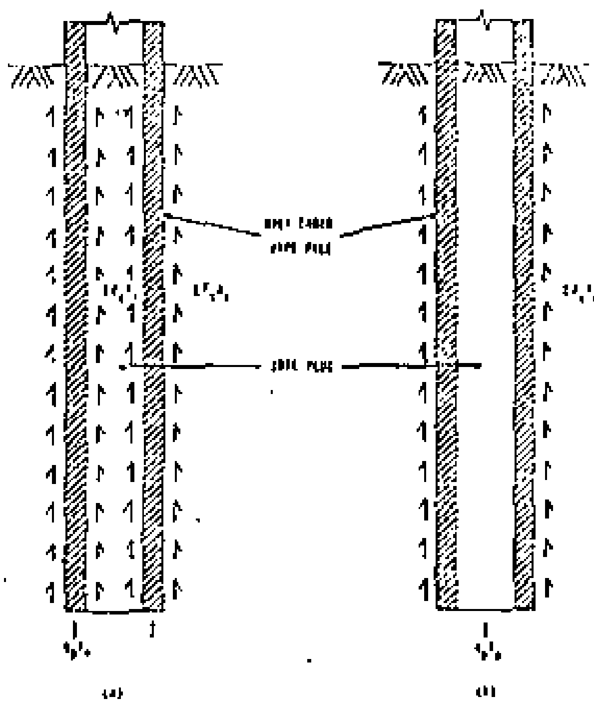


Figure 2 : Capacity of open-ended pipe piles
(a) unplugged (b) plugged

Under certain conditions the accumulated inside skin friction may exceed the ultimate static capacity of the soil below the toe of the pile. The pile then behaves as though it is closed ended or plugged. Its static capacity is given by:

$$Q = \Sigma f_s A_s + q_p A_p \quad (3)$$

where A_p = gross end bearing area

Many static methods for the determination of pile capacity exist, all of which provide different means of calculating the values of q_p and f_s, the unit end bearing and skin friction respectively. For a given pile and site each design method will predict different unit values. It is not possible to state with complete confidence that one method is correct and all others are incorrect. However, some pile design methods are more applicable to certain soil conditions than others. Rather than placing complete reliance on one design method it is preferable to analyse piles using a variety of methods in an attempt to bracket the correct capacity. For any particular problem, methods should be chosen for their varied theoretical background and dependence upon different soil properties. The design methods in most common use for offshore driven piles, together with the soil parameters required, are shown on tables 2 and 3. These tables cover cohesive and cohesionless soils respectively

Table 2 : Pile Design Methods in Cohesive Soils

Method	Soil Parameters Used		Applicable Soil Types
	End Bearing	Skin Friction	
API No. 1[3]	τ_u	τ_u or τ_{nc} * whichever is less	under, normally and lightly overconsolidated clays
API No. 2[3]	τ_u	τ_u	heavily overconsolidated clay
Lambda[4]	τ_u	γ', τ_u	under, normally and lightly overconsolidated clays
Beta[7]	τ_u	γ', OCR, K_o, ϕ'	all clays
Parry & Swain[8,12]	–	γ', c', ϕ', OCR K_o, PI**	all clays
Janbu[13]	γ', c', ϕ'	γ', c', ϕ', OCR K_o, PI**	all clays
Vijayvergiya[14]	τ_u	γ', c', ϕ', OCR K_o, PI**	all clays
Critical State[10,11]	τ_u	γ', ϕ, PI, LL K_o, OCR, C_c, C_s†	all clays

* τ_{nc} is shear strength clay would have if it were normally consolidated

** When OCR and K_o values have not been measured directly, they may be estimated indirectly from the plasticity index

† C_c and C_s are compressibility and swelling indices respectively

Table 3 : Pile Design Methods in Cohesionless Soils

Method	Soil Parameters Used		Applicable Soil Types
	End Bearing	Skin Friction	
API[3]	γ', grading	γ', grading	all cohesionless soil
CPT[3]	q_c	q_c	all cohesionless soil
ϕ'[17,18]	γ', ϕ'	γ', ϕ'	all cohesionless soil

and indicate the soil conditions to which they are most applicable.

There are many occasions when the use of an inappropriate design method would lead to misleading axial capacity predictions. The applicability of the various methods is discussed in more detail below.

Cohesive soils

API Method 1. Two methods for determining the skin friction of driven piles in clay are described in API RP2A.[3] It is stated in this document that Method 1 is suitable for 'highly plastic clays such as found in the Gulf of Mexico'. The authors do not limit the use of this method to highly plastic clays and apply it to all clays except those which are heavily overconsolidated. For the purposes of this paper a heavily overconsolidated clay is one which has an overconsolidation ratio (OCR) greater than two.

API Method 2. In API RP2A it is suggested that Method 2 should be used for all clays which are not highly plastic. The authors use this method for heavily overconsolidated clays irrespective of plasticity. The authors do not use it for normally consolidated or lightly overconsolidated clays.

The reasons why the authors do not follow the advice of API RP2A with regard to the plasticity of the clay are based on traditional UK practice[6] and recent published works on calculating pile capacity.[7,8]

Lambda Method. The Lambda Method has been developed mainly from tests on piles of varied length in normally consolidated and lightly overconsolidated clays and on short piles in heavily overconsolidated clays. For short piles in normally consolidated clays the modified curve described by Fox[9] should be used. The authors use the Lambda Method for the same soil conditions for which they

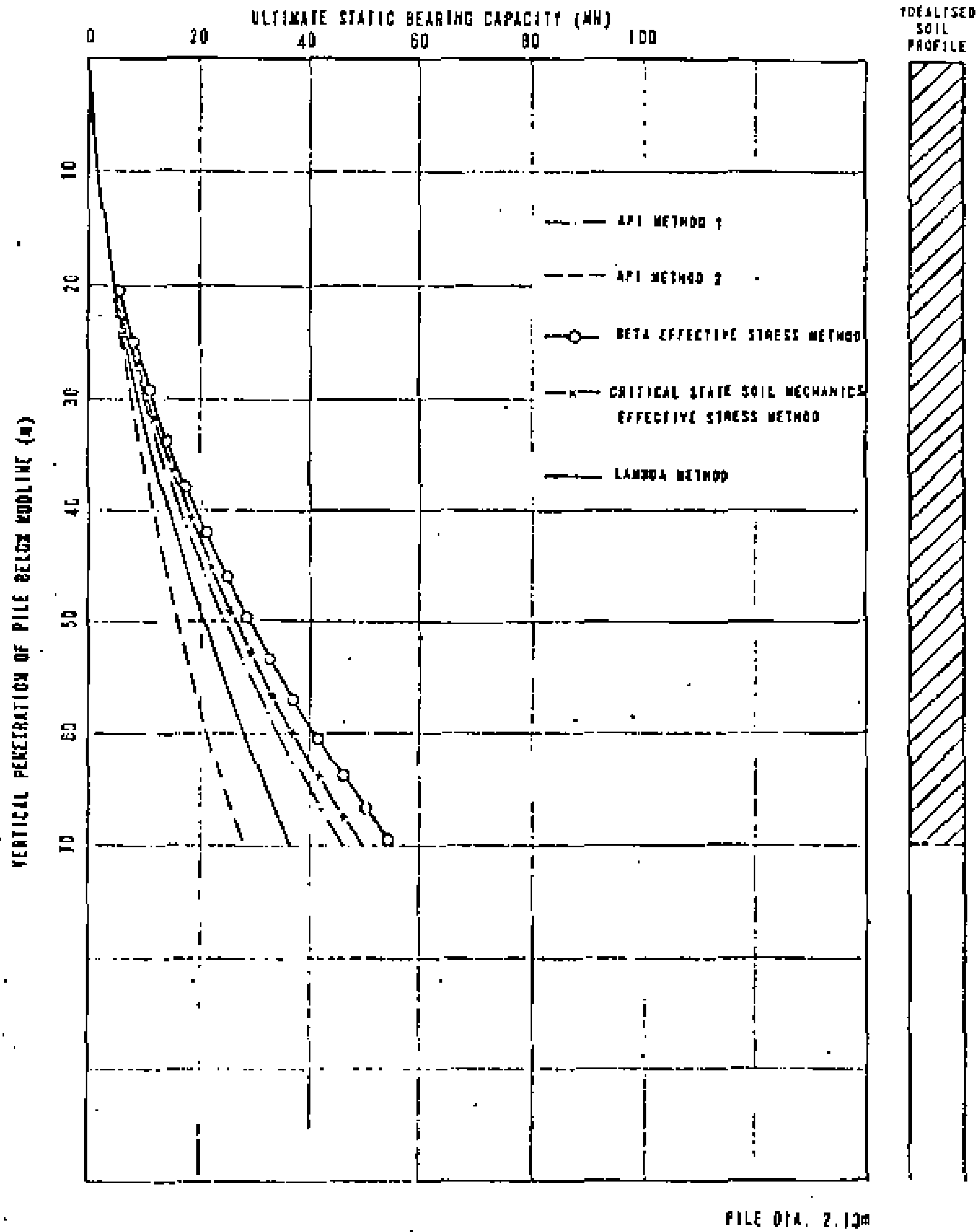


Figure 3: Comparison of axial capacity methods for a normally consolidated clay site (c/p = 0.25, PI = 18).

employ API Method 1. Caution must be exercised when applying the Lambda Method to soil profiles which show sudden and significant increases in shear strength at depth. This is because the method contains a depth factor which significantly reduces the skin friction contribution of a deep hard clay stratum.

Effective stress methods. A number of effective stress methods[10,11,12,13,14] have recently been developed and are already being applied to calculate the capacity of offshore piles. All these methods involve the use of the effective stress parameters, c' and Φ', of the clay and require the determination of the long term stress levels in the soil around the pile. The methods are easy to apply in normally and lightly overconsolidated clays and correlate well with load tests in such soils. In heavily overconsolidated soils difficulties can be encountered in estimating the value of the earth pressure coefficient. These can be overcome by ensuring that the in-situ and laboratory testing programmes allow the use of several methods to determine the value of this parameter. If all methods yield similar results, then values can be adopted with confidence.

CPT methods. A considerable amount of research has been performed, and is continuing, concerning the direct application of CPT results to the design of piles in clay.[15,16] To date, the results have been encouraging but are not directly applicable to the large pipe piles used offshore. At the present time the authors do not use CPT results directly for the design in piles of clay. However, they can be used to determine an undrained shear strength profile which can be used in conjunction with the appropriate API method.

Comparison of methods. The extent of variation in axial capacity predicted by the different design methods is illustrated in figures 3 and 4. Figure 3 shows the axial capacity given by API Methods 1 and 2, the Lambda method, and effective stress methods for a site with normally consolidated soil. Figure 4 presents the same information for a heavily overconsolidated site with a non-linear soil profile. In each case the end bearing has been computed on the basis of $9 c_u$. These figures show why API methods should be used with care.

Cohesionless soils

There are three types of design method for piles in cohesionless soils, those based on:

1 grading, e.g. API[3]
2 measured Φ', e.g. Meyerhof[17] and DnV[18]
3 in-situ tests, e.g. CPT[5]

All contain large degrees of empiricism and those based on gradings and in-situ tests suggest upper limits on the values of unit skin friction and end bearing. Those which involve measured Φ' values do not always mention limiting values and assume the user will apply his own. For long piles, such as those used offshore, the calculated capacity may be governed more by the selected limits than by the design method itself. It is the authors' practice to use limiting values of 15MN/m² for unit end bearing and 0.12MN/m² for unit skin friction for all methods in all cohesionless soils provided that such limits can be justified on the basis of measured CPT values. This is covered under the CPT design method which is described later. For strata in which CPT tests have not been made the authors use published limiting values which depend on grading.[18]

Precompression of a granular strata appears to have a considerable effect on the magnitudes of skin friction and end bearing which can be developed in that soil. Pile design methods which are based on grading or measured Φ' values contain the implicit assumption that the soil has not been precompressed. Part of its effect can be incorporated into skin friction calculations by using earth pressure coefficients (K) which are at the upper end of the suggested range. For most methods K values can be selected within the range 0.5 to 1.0. Methods based on CPT results automatically take account of preloading, since preloading also affects the cone resistance.

Comprehensive descriptions of the pile design methods based on grading and measured Φ' are available in published literature.[3,17,18] However, there are very few references in English on the use of the CPT method for offshore piles. For this reason the way in which the authors use CPT results to design piles is described below.

CPT Method. In this method values of unit end bearing and unit skin friction are derived directly from CPT results.

Unit end bearing. The basic computational technique by which the ultimate unit end bearing is obtained from a CPT profile is shown in figure 5. The calculated value is subjected to correction factors based on grading and degree of precompression as shown in figure 6. The unit value to be used in the computations is limited to 15MN/m².

Unit skin friction. The unit skin friction is set to zero at mudline and increased linearly to the computed value at the depth at which lateral deflections due to shear loading are small. Below this level computed values are used. This procedure is used to account for any long term changes in lateral soil stresses which may occur near seabed due to cyclic shear loading of the pile. The computed value of unit skin friction is obtained by dividing the measured cone resistance by a factor of 300 for piles loaded in compression and 400 for piles subjected to uplift. A limit of 0.12MN/m² is applied to the computed value.

Comparison of Methods. In figure 7 pile capacities computed by the DnV method and CPT method in sand at a typical North Sea location are compared.

Dynamic Methods

In addition to the static design methods for cohesive and cohesionless soils described above, dynamic techniques are available for predicting pile capacity. These are based on an analysis of measurements made on piles during redrive tests. As such they are outside the scope of this paper. However, it should be mentioned that the correlation between predicted capacities and measured static capacities is good and their use in conjunction with static methods is invaluable for pile acceptance.

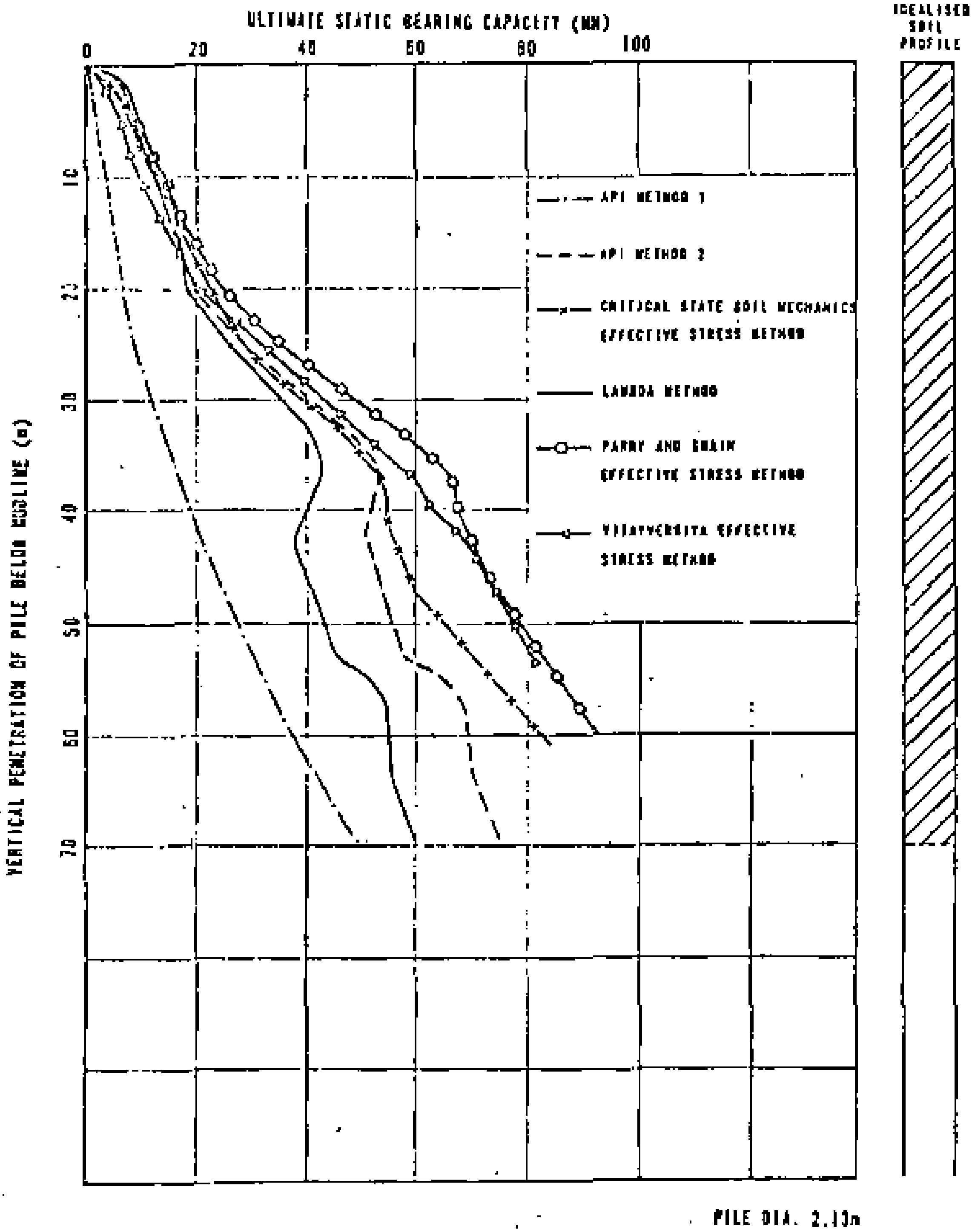
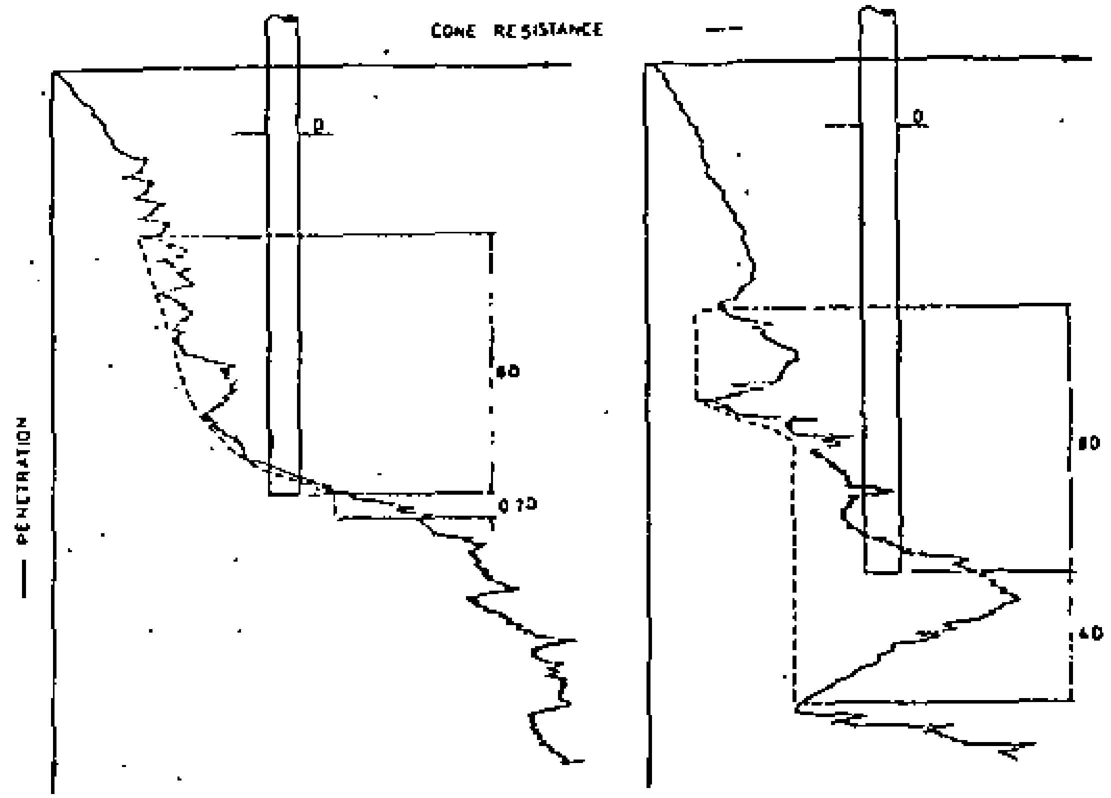


Figure 4 : Comparison of axial capacity methods for a heavily overconsolidated clay site



The ultimate unit end bearing "q_u" is calculated from:

$$q_u = \frac{\frac{I + II}{2} + III}{2}$$

in which:

- I = average cone resistance below the foundation over a depth which may vary between 0.7D and 4D, selected in such a manner that the most unfavourable condition is obtained with respect to the computed bearing stress
- II = minimum cone resistance recorded below the foundation over the same depth of 0.7D or 4D
- III = average of the envelope of minimum cone resistance recorded above the foundation level over a height of 8D. In determining this average, values above the minimum selected under II are to be disregarded.
- D = Outside diameter of pile.

NOTE: For an open ended pile, the ultimate unit end bearing of the annulus may be assessed directly from the cone resistance (qc). It must not exceed the limit value.

Figure 5 : Computation of the ultimate unit end bearing of a pile from a cone penetration test

LATERAL CAPACITY

There are two approaches for calculating the lateral capacity of a long pile. The first involves the assumption that, at failure, a plastic hinge forms in the pile shaft below mudline. The lateral capacity is then the shear force at mudline which can form the hinge and overcome the soil resistance above it. The soil resistance is a function of the passive pressure.[19] The soil parameters required are density (γ), the undrained shear strength of clay (τ_u) and the angle of shearing resistance of sand ϕ'. In the second approach the soil resistance is described by non-linear ground springs known as P—Y curves. The pile is treated as an elastic "beam-column" against which the P—Y curves act. A computer program is used to produce a curve showing the relationship between shear load and pile deflection. The ultimate lateral capacity is the shear force above which pile head deflections become indeterminate. The soil parameters required for developing P—Y curves are discussed later in this paper.

It should be noted that for the majority of offshore piles the maximum allowable shear load is not directly related to the lateral capacity calculated by either approach. It is determined on the basis of acceptable deflections and the stresses in the pile wall due to combined axial and lateral loading. However, under certain circumstances the factor of safety against shear failure may be governing, and it should always be checked.

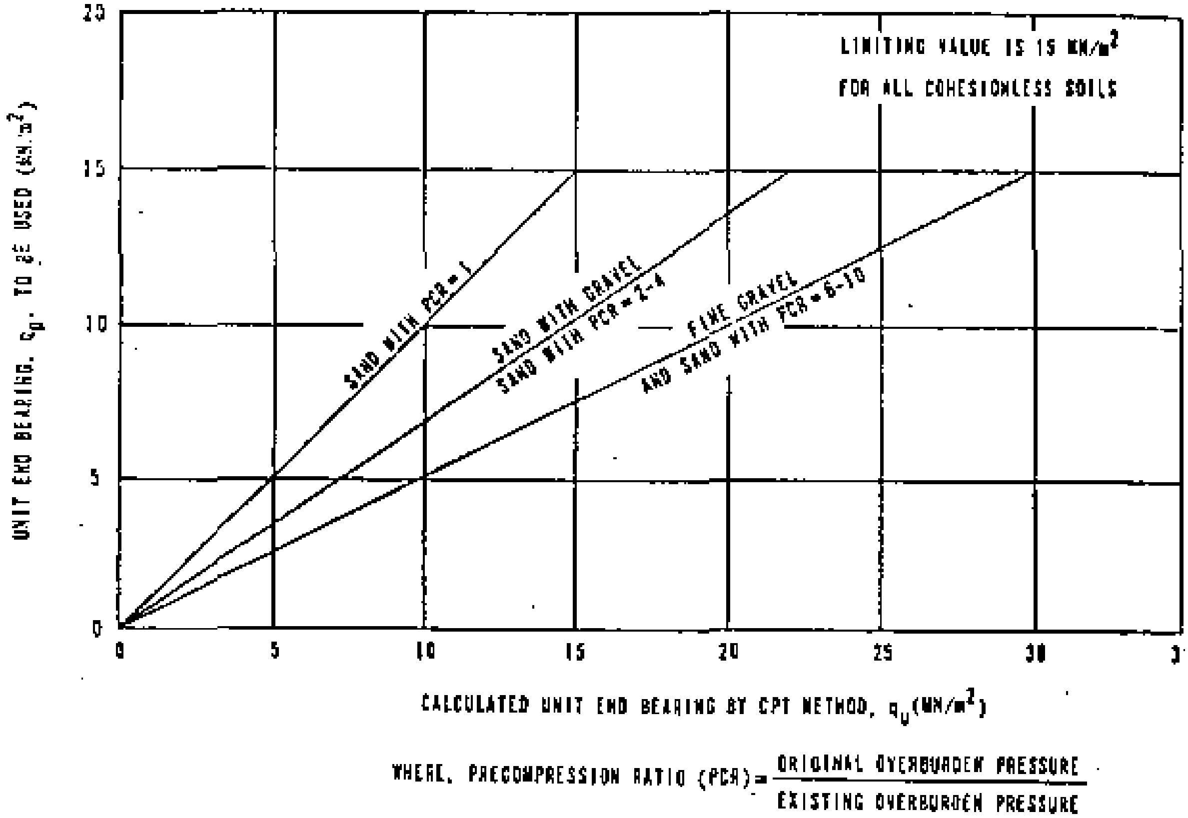


Figure 6 : Correction factors for the point capacity of driven piles. (after te Kamp)

LOAD-DEFLECTION BEHAVIOUR

To estimate the load-deflection behaviour of piles and pile groups the authors use discrete element techniques and elastic methods. For the discrete element approach the soil is modelled as a series of non-linear ground springs. These are known as P—Y curves for horizontal deflections and T—Z curves for axial deflections. The pile is divided into elements and the relevant P—Y curve and/or T—Z curve acts on each element. A beam-column computer program is used to determine the load-deflection curve.[20] Generalised solutions are available for the load-deflection behaviour of a pile in a homogeneous elastic continuum.[21] To use these the only soil parameters required are the overall soil moduli for horizontal and axial loading.

P—Y Curves

It is important to realise that P—Y curves are analytical tools and not fundamental soil properties. Numerous forms of P—Y curves have been derived by backfiguring ground springs from lateral pile test results through 'beam-column' programmes. For the soil conditions in the North Sea and cyclic shear loading it is the authors' practice to use the P—Y curves listed in table 4. The soil parameters required to develop these P—Y curves are also presented in table 4.

Table 4 : Methods for Determining P—Y Curves

P—Y Curve Derivation Method	Soil Parameters Used	Applicable Soil Types
Matlock[22]	τ_u, ϵ_{50}	all clays
Reese, Cox & Koop[23]	τ_u, ϵ_{50}	Overconsolidated clays
Reese, Cox & Koop[24]	ϕ_d, γ, Relative Density	sand

The P—Y curve computation methods should not be used blindly. Those listed above have been derived for relatively homogeneous soil conditions. They require careful adaptation when used in stratified soil conditions. For example the 'stiff clay P—Y curves' can give a much softer response than the 'soft clay P—Y curves' when applied to the same soil parameters for conditions of large strains see figure 8. Before finalising the P—Y curves, the geotechnical consultant should examine them in the light of the deflected shape of the pile obtained from the beam column analysis.

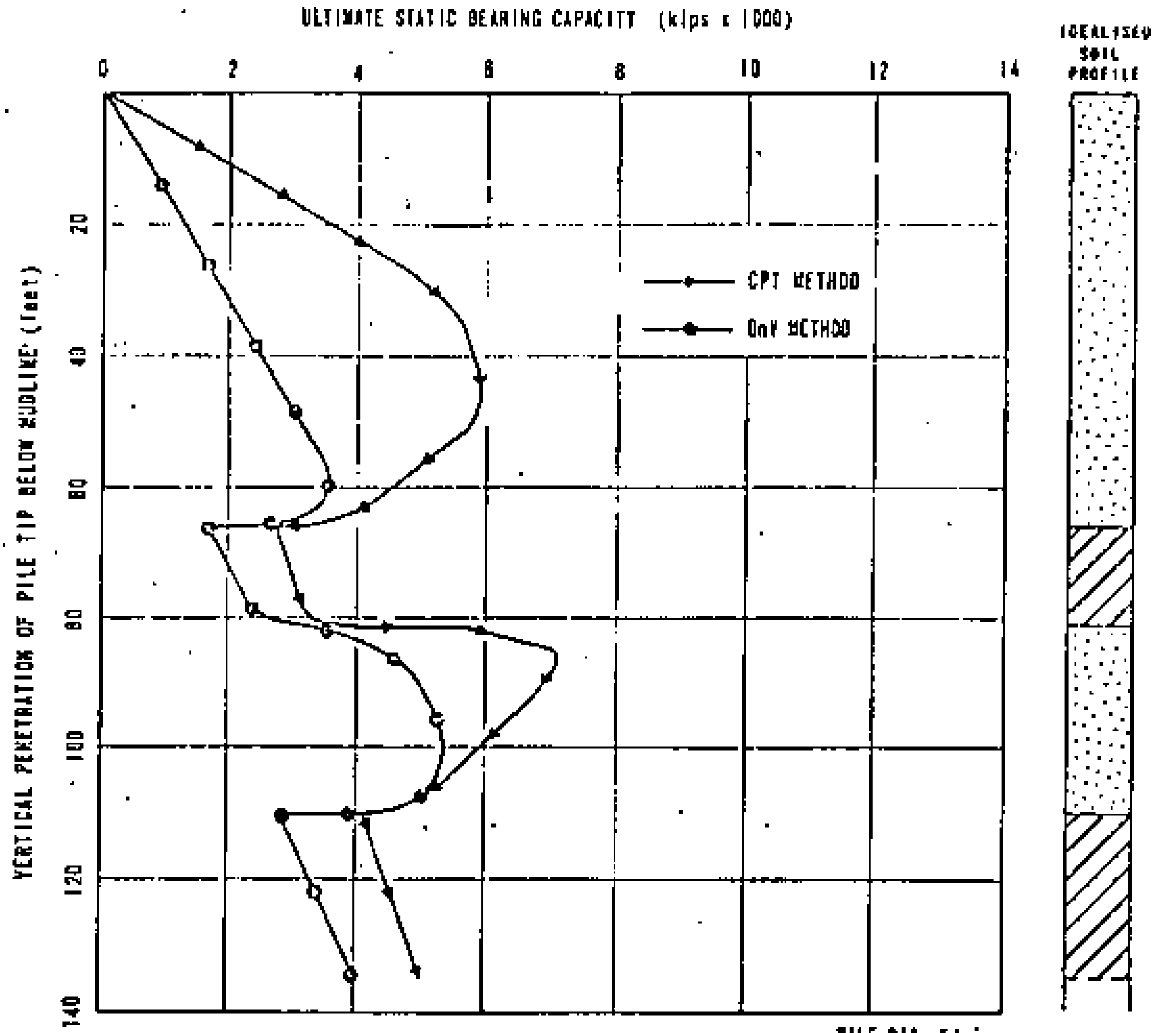


Figure 7 : Comparison of axial capacity methods for a sand site

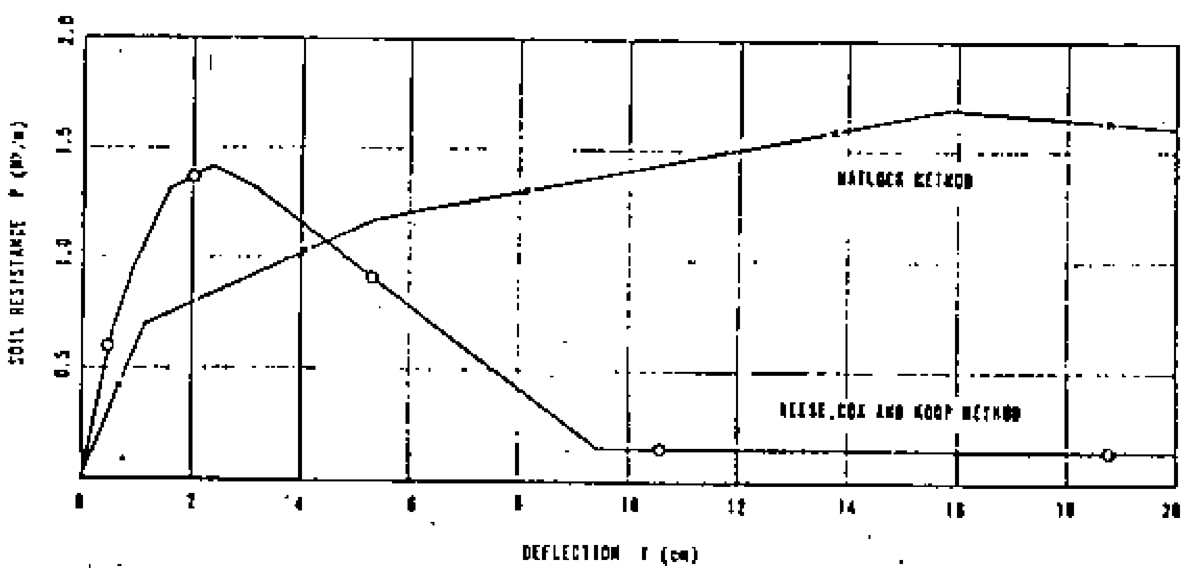
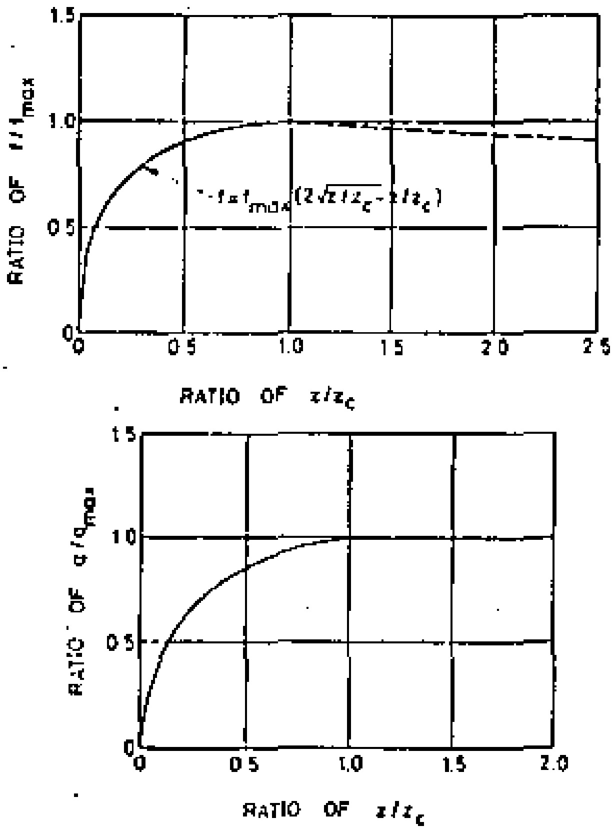


Figure 8 : Comparison of P–Y curve computation methods, for clay

T—Z curves
A variety of forms of T—Z curves are described in the literature.[27,28] The shape of a T—Z curve is generally independent of any soil parameters other than the peak soil resistance as shown in figure 9. This is always determined by multiplying the unit skin friction by the shaft area of an element or, for the pile tip, the unit end bearing by the base area. This ensures that the failure load indicated by the computed load-deflection curve is equal to the calculated pile capacity. Thus the only soil parameters required are automatically provided by the axial pile capacity calculations.



z_c = critical deflection
f = unit skin friction
q = unit end bearing
NOTE: The following z_c values have been used.
z_c = 7.6mm for maximum unit skin friction in sand and clay.
z_c = 0.05D for maximum tip resistance in clay.
z_c = 0.08D for maximum tip resistance in sand.
where D = pile diameter.

Figure 9 : Normalised load-deflection curves for sand and clay (after Vijayvergiya).

Soil modulus for lateral loading
To obtain the lateral load-deflection behaviour from a generalised elastic solution a suitable soil modulus, E_s must be estimated.

For laterally loaded piles in stiff clay a ratio of elastic soil modulus to undrained shear strength of 200 appears appropriate.[31] This relationship can be used to convert the shear strength profile into a profile of elastic modulus versus depth. For sites where the soil conditions are fairly uniform the determination of an equivalent elastic modulus causes little problem. However, for more typical North Sea sites where layers of normally and heavily overconsolidated clays may be interspersed with sand layers the equivalent modulus is not obvious.

From the beam-column analyses performed using P—Y curves, the distribution of lateral displacements from the pile head to the point of contraflexure (typically at 10 to 15 pile diameters beneath the pile head) will be known for working load conditions. Using these results an equivalent elastic modulus can be determined from the following expression:

$$E_s = \frac{\sum_{i=1}^{n} E_i \cdot \delta_i}{\sum_{i=1}^{n} \delta_i} \qquad (4)$$

where
n = number of elevations considered down to point of contraflexure
E_i = elastic modulus at i th elevation
δ_i = lateral displacement of i th elevation at working load.

In sand strata the authors have derived elastic moduli values from the stress-strain relationships obtained from consolidated drained triaxial tests. Analysis of several such tests at any one site enables a relationship between elastic modulus and effective overburden pressure to be obtained of the form:

$$E_s = C\sqrt{\sigma'_v} \qquad (5)$$

where
C = a constant peculiar to the sand of a particular site.

Using equation (5) an elastic modulus for sand can be obtained at any depth for substitution into equation (4).

Soil modulus for axial loading
Published data indicates that an appropriate ratio between elastic modulus and undrained shear strength for immediate settlement calculations for piles in overconsolidated clays is 400. This is for working load levels and is twice the value used for laterally loaded piles. The reason for this difference is stress level.[38] For laterally loaded piles movements are governed largely by the soil properties along the upper length of the pile shaft. Frequently the soils near seabed level are in a state of failure. In contrast, displacements of axially loaded piles are governed mainly by the soil properties at the tip and along the lower length of pile shaft. The soils in this zone are relatively lightly stressed. The work of Seed and Idriss[39] which provides a relationship between stress level and elastic moduli of soils, indicates that the factor of two mentioned above is of the correct order of magnitude.

In sand strata the soil modulus for axial loading is estimated in exactly the same way as for lateral loading, i.e. with equation (5). However, the computed value of the axial modulus is always very much higher than that used for lateral loadings. This is because only the upper sand layers are used for lateral analyses whereas the strata along the whole length of the pile shaft are included in the axial computations.

Application of in-situ tests

P—Y curves suitable for determining the lateral load-deflection behaviour of piles subjected to static loading can be obtained directly from the results of CPTs.[30] However, these are of limited application to offshore pile design since shear loads are generally cyclic in nature. Research is continuing on this subject.

As explained previously T—Z curves are developed on the basis of unit skin friction and unit end bearing both of which can be obtained directly from the results of CPTs.

For many years CPT results have been used to calculate long term settlements of shallow foundations on land. Usually a pseudo-elastic approach is used which involves the determination of a soil modulus. However, it is not considered that the correlations used to calculate this modulus can be applied to the long piles, subjected to transient loading, which are used offshore. Attempts are being made to develop suitable correlations.

New and improved in-situ tests are being developed to provide more reliable data on which to base load-deflection calculations. These include advanced pressuremeters and a seabed rig developed by the authors' parent company which can install large scale model piles and perform lateral pile load tests remotely from a soil survey vessel.

PILE DRIVABILITY

The ability of the available pile driving plant to install the piles to the required penetration must be assessed. One important step is to perform wave equation analyses using a computer. The theory and application of the wave equation to pile driving problems has been described in a number of papers.[31,32,33] The results are normally presented as curves of blowcount versus soil resistance at time of driving (SRD).

The SRD at any depth is determined from the following considerations. If the soil inside the pile (soil plug) remains stationary during driving the SRD must be made up of inside and outside friction and wall end bearing. In this situation the magnitude of the inside friction which can be mobilised may be limited by the end bearing capacity of the soil plug. Alternatively the soil plug may move down during driving in which case the inside friction must be equal to or greater than the plug end bearing. Generally the pile will behave in the manner which produces least resistance to pile penetration.

Thus at any depth the SRD will be the least of:

$$SRD = \Sigma f_s' A_s + \Sigma f'_i A_i + q_p' A_W \qquad (6)$$

or

$$SRD = \Sigma f_s' A_s + q_p' A_p \qquad (7)$$

where

f_s' = unit shaft friction during driving (outside)
f_i' = unit shaft friction during driving (inside)
q_p' = point resistance during driving

Calculation of point resistance during driving

The CPT is a model test for the penetration of a pile. The point resistance during driving (q_p') may be calculated from the cone resistance (q_c). When the pile is plugged (i.e. the inner soil plug moves down with the pile) the unit base resistance may be calculated by the method shown in figure 5. No limits are applied. Thus,

$$q_p' = q_a \qquad (8)$$

When the pile is unplugged (i.e. the inner soil plug remains stationary as the pile tip penetrates the ground) the unit resistance acting on the pile wall annulus is taken to be equal to the cone resistance at that depth,

$$q_p' = q_c \qquad (9)$$

Calculation of shaft friction during driving in cohesive soils

The contribution of skin friction to total SRD is calculated on the basis of laboratory test results. Along the length of the pile, the soil at the interface with the pile wall is strained to failure by every hammer blow. In a cohesive material the soil is compressed to accommodate the volume of the pile as it penetrates. The displacement, shearing and compression remould the soil and cause excess pore water pressures to be developed. Thus during continuous driving in a clay:

$$f'_s = f'_i = C_r \qquad (10)$$

where C_r is the remoulded shear strength.

Data on the remoulded shear strength of a clay only become available after the onshore testing programme has been completed. To produce preliminary estimates of drivability offshore, geotechnical consultants have developed empirical relationships between undrained shear strength and unit friction during driving in clay. These relationships are based on the particular offshore pile driving experience of the geotechnical consultant and are proprietary in nature.

Even in an onshore laboratory, it may prove impossible to perform sufficient meaningful remoulded tests in the time available. This is particularly the case with heavily overconsolidated clays which may have to be ground down, reconstituted and reconsolidated prior to shearing.

In order to overcome this problem, the authors place considerable reliance on empirical relationships between remoulded shear strength and other properties of a soil such as those developed by Skempton and Northey[34] and Houston and Mitchell.[35] The remoulded shear strength according to Skempton and Northey depends on a relationship between C_r and plasticity and liquidity indices derived from laboratory measurements.

Calculation of shaft friction during driving in granular soils

In a granular soil it is assumed that the unit shaft friction during driving is equal to the static unit shaft friction:

$$f_s' = f_i' = f_s = q_c/300 \qquad (11)$$

and may be calculated using the cone resistance as described previously. Limits should be applied but no allowance should be made for lateral displacement effects. Equation (11) should be applied at all levels.

CONCLUSIONS

This paper has briefly described some of the ways in which soil parameters obtained during an offshore site investigation are used for pile design. Other techniques are also employed and there are a number of aspects of the foundation design of a jacket which have not been covered; for example, group action, pile acceptance, unpiled stability and conductor design and installation. The paper is by no means comprehensive. However, if it has explained why detailed soil data from in-situ and laboratory tests must be obtained, it will have achieved its objective.

New in-situ testing techniques are under development and will eventually be introduced into offshore investigations. These should improve the reliability of the determination of the parameters K_o and E_s and also allow P−Y and T−Z curves to be developed based upon the specific properties of soils at a given site. It is expected that, in time, these additional data will lead to more economic foundations without reducing the required safety margins.

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DISCUSSION

Mr P George (Consultant): I am surprised that anyone should consider using API Method 1 for heavily over-consolidated clays since it is based on data which only relates to normally consolidated clays in the Gulf of Mexico.

Mr E Toolan: In fact it used to be used for all offshore piles.

Mr George: There is a lesson to be learned that before we start applying well publicised rules and regulations laid down by anonymous authorities we should look into the background of the data.

Mr Toolan: That is the value of meetings such as this one.

Mr R Barton (Brown and Root (UK) Limited): How do we take account of live dynamic loading in design?

Mr Toolan: Although dynamic loading may increase the carrying capacity we think this increase is built into the safety factor. We do not take dynamic loading into account explicitly.

Mr B McClelland (McClelland Engineering Inc): The API Method 1 evolved out of design practice in the Gulf of Mexico in an effort to avoid the over-conservatism resulting from the blind application of limiting adhesion and alpha factors. The Lambda method arose out of some undesirable field evidence for a very long test pile which indicated that the capacity was less than predicted by API Method 1. Thus this is an area where caution should be exercised before moving too quickly to conclusions. It should be an evolutionary process.

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19 Application of offshore site investigation data to the design and construction of submarine pipelines

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R J Brown Associates

In the context of civil engineering as a whole, a marine pipeline is a highly unusual structure. Almost all significant structures have elaborate foundations, whereas submarine pipelines are often placed on the unprepared seabed, in a previously-surveyed corridor followed only within an accuracy of a few tens of metres. One can do this with pipelines because they are robust and relatively light, because their function does not require them to be perfectly straight, and because there is usually some choice of route. The constructor of a marine pipeline does, all the same, need information about the sea and the seabed. This paper sets out to explore what information is required, why it is needed and how it is used. Our ability to obtain information naturally does not always advance in step with our needs. Sometimes the constructor would like to have information which in practice cannot be obtained economically, or perhaps cannot be obtained at all: that may initiate a demand for more advanced investigation techniques. Sometimes, on the other hand, a site investigation can provide information which the constructor suspects to be relevant, but does not yet know how to make use of.

It is unusual for the cost of acquiring environmental data to be a large fraction of the total cost of a marine pipeline, and survey is usually between two and six per cent of the total project, but the amounts involved are still quite large. The owner and the engineer will want to be certain that this money is used prudently, and that nothing is spent on gathering unuseable information, but that there are no gaps in information that have to be expensively filled at a later stage. A survey programme obviously has to be carefully planned. Less obviously, it is important to link the survey with the subsequent engineering and construction phases. The least efficient thing to do is to define a survey, and send out surveyors to follow the programme blindly, without any interaction with, or perhaps any interest in, the rest of the construction. One then risks receiving a meticulously accurate survey of a route which a few hours' study shows to be completely unsuitable for a pipeline. Except in areas which are already well known and where no surprises are to be expected, it is much better to send with the survey team an engineer who has experience of design and construction, and who can make a preliminary assessment on the spot, and use his knowledge and judgement to revise the route and perhaps alter the kind of data collected. One wants someone who can review the data, and say, for instance: 'this ridge is less steep the further east we go. If we go over another 100m, we ought to be able to keep the longest span down to 20m — which should be all right because the highest current does not seem to be more than 2 knots — and still have a straight line on shore to winch the pipe across those tidal flats. Let's take some core samples over there'.

The standpoint of the present review is that of the user of the information, and techniques and equipment are discussed relatively little. We first examine the significance of site investigation data that describe the sea itself. The depth and topography are discussed next, and this is followed by an examination of the questions involved in the description of the seabed material. Finally an example of a marine pipeline site investigation is described in more detail.

CURRENTS

Many marine pipelines are not completely buried in the seabed and are exposed to water currents. Strong currents can push the pipeline sideways, or cause free spans to oscillate because of vortex-shedding, or generate sediment movement which can bury the pipe in a sand wave or scour support points. Accurate current information is needed to assess these factors, and in the assessment of construction techniques. In laybarge construction, for example, a suspended span perhaps 500m long hangs from the stinger behind the barge, and current forces on the span can bend the pipe significantly, altering the position at which it reaches the bottom, and perhaps overloading the stinger and the mooring system.

An assessment of the stability of the pipe on the bottom requires an estimate of the extreme current close to the bottom during the operating life. The extreme current has a tidal component, a wave-induced component, and sometimes other components corresponding to wind drift, density currents (related to temperature and salinity variations) and turbidity currents induced by submarine slides. Long-term measurements of tidal currents are not usually available. The usual procedure is to set recording current meters just above the bottom, and to leave them in place for at least one month, preferably longer, to record current velocity and direction at 15 minute intervals. The record is too short for a reliable harmonic analysis, and so extreme currents are often estimated using methods derived from the statistics of extremes, like those used to estimate extreme waves and floods. It may be felt not to be logically correct to apply statistics to find extreme values belonging to a deterministic phenomenon like the tide, and such a procedure may generate design currents which are unnecessarily conservative: this subject deserves more attention. Little is to be gained from more sophisticated current recording, unless it is possible to lengthen the recording period, or of course to reduce the cost of meter setting, recovery, and data handling.

Long records of wave conditions are scarcely ever available, except where oil industry cooperative programmes have maintained ships and wave rider buoys in one place for several years, as in the northern North Sea and the Gulf of Mexico. Extreme waves can obviously not be estimated reliably from short records covering only a few months: instead it is better to estimate the extreme storms from wind records from the nearest meteorological station, and then to estimate the extreme design wave, using one of the standard forecasting procedures.[1] Density and turbidity currents are not regular enough to be easily accessible to measurement, though they may show up as anomalous data points on current meter records: they are estimated by oceanographic methods. An indirect warning of turbidity currents may be given by submarine landslide scars, seen in bathymetric records and sub-bottom profiling.

DEPTH AND TOPOGRAPHY OF THE SEABED

Knowledge of depth is needed for several reasons. It determines the external pressure, which is important in deep water, where the pipeline must be designed to be secure against buckling and collapse. It enters the calculation of wave effects on the bottom. Most importantly, it is an important factor in the assessment of the construction method: for instance, the laybarge tension required to keep the pipeline bending stress to an acceptable level is strongly dependent on depth, as are the flexibility of the mooring system and the ability to hold station by dynamic positioning. For these purposes, absolute depth below the surface is wanted. Conventional hydrographic surveying depth measurement techniques are adequate, and an accuracy of one per cent of depth is quite sufficient, provided that it is combined with an accurate and repeatable system of horizontal control. The introduction above explained the importance of on-the-spot review and evaluation of survey data, which is naturally made easier by 'real time' bathymetric systems, such as BS3 and 'Seabeam', which provide a contoured bathymetric chart on the survey vessel.[1] Systems of this kind are now becoming commercially available.

Like the surface of the land, the bottom of the sea is often rough. The configuration taken up by a pipeline on the bottom is determined by its topography, and by the relative depth at different points along the route. A uniform slope is not very significant, unless it is so steep that the bottom itself might become unstable, or that the pipeline might slip across it. Irregularity and curvature of the bottom are much more important. If the bottom is slightly uneven, the pipe will bend to conform to it, and that will induce bending stresses in the pipe wall. If the curvature is more pronounced, the pipe will rest on the high points and bridge the valleys between them, forming free spans. Often it is impossible to find a route which will not have spans. They require careful investigation, both in the initial design stage and later during construction, and tend to require frequent inspection and costly maintenance, as several notorious North Sea examples demonstrate. Bending caused by the weight of a free span may overstress the pipe, particularly during hydrostatic tests. A span is free to oscillate, and severe oscillations may be set up if vortex-shedding induces fluctuating hydrodynamic forces whose spectra have dominant frequencies close to a natural bending frequency. Oscillations of this kind may so weaken the pipe's concrete weight-coating that it falls off: this makes the pipe lighter, which worsens the situation, and leads to progressive growth of the span. They may also cause direct fatigue damage. Free spans are also subject to large hydrodynamic forces, and are particularly prone to scour and hooking by fishing gear.

It is usually possible to avoid the largest features on the bottom, such as steep submarine cliffs and ravines, but rough areas occur in most parts of the world.

Figure 1 is a map of one area of rough bottom topography, part of the seabed close to Norway. Figure 2 is a section of a sand wave, and includes a profile showing the configuration that would be taken up by a pipeline laid across the wave. These are relatively dramatic examples, but smaller features still induce significant spans. For instance, a one metre high 'hill' on an otherwise level bottom will induce two spans 50m long in a typical North Sea pipeline. Because of this, a proper assessment of bottom roughness effects on the pipe required an accurate knowledge of bottom topography, with relative heights measured to within 0.1m or better. This accuracy is much higher than that of conventional survey, and is difficult to obtain, at least in deep water. In shallow water, however, accurate profiles can be obtained: figure 3 is a profile across a submarine trench, excavated by a plough in muddy silt, and comparison between different records, and with direct measurements by

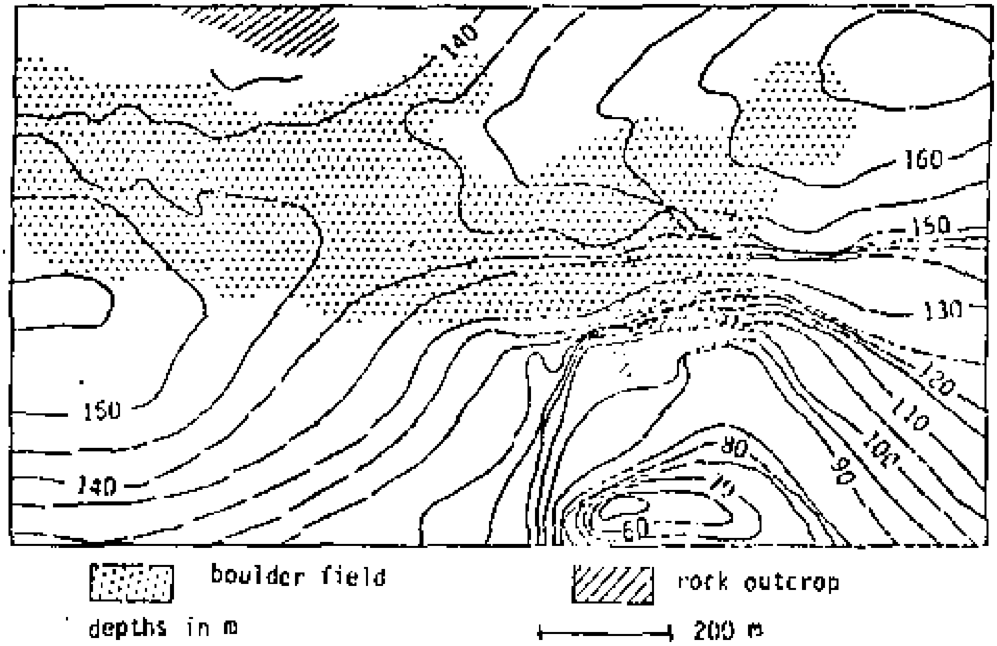

Figure 1 : Rough sea-bottom topography close to Norwegian coast

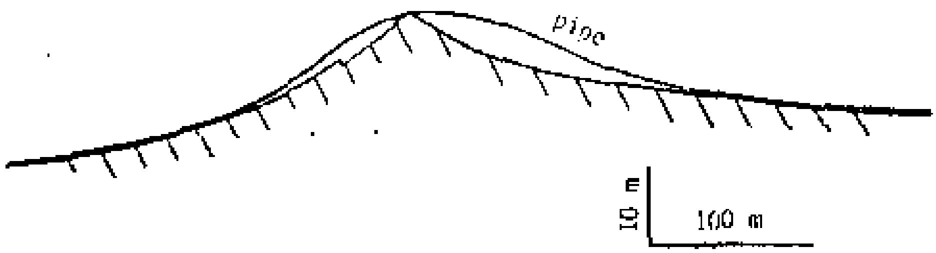

Figure 2 : Section through sand wave (vertical exaggeration 5 : 1)

a diver, indicates that accuracy of the profile is within 0.1m. This profile was obtained by a Mesotech 120kHz profiling echo-sounder.

A recent review discusses the general problem of bottom roughness measurement.[3] If a profiling fish can be towed just above the bottom, and can follow a smooth trajectory without abrupt vertical movement, a much more accurate measurement of relative depth can be obtained, because of reduction of parallax error, bottom slope effects, the influence of time measurement errors, and the effect of uncertainties in sound velocity. Absolute depth measurement can still be made with a distinct upward-looking echo sounder on the fish, or with a sensitive pressure transducer, and the advantages of better relative depth measurements remain, even if there is no increase in the accuracy of absolute depth measurement. A submersible equipped with inertial navigation and a high-frequency profiler can be used in the same way.[4]

A survey of the bottom topography of a complete pipeline route to an accuracy of the order of 0.1m, associated with good horizontal control, will obviously generate an enormous amount of data, and will be expensive. An alternative approach is to use relatively crude basic bathymetric information to identify major features like cliffs and sand waves, to study them in detail, and otherwise not to try to measure the whole route, but instead to 'sample' the bottom roughness by detailed measurements at a few places. One could then carry out a deterministic analysis of the configuration of the proposed pipeline on the sample areas, or the bottom roughness could be described statistically, by the standard deviation of depth, by the distribution of peaks and troughs, or by the surface autocorrelation and spectrum, and they could be put into the pipeline analysis. This last approach is known in mechanical engineering[5] and in aerodynamics and, of course, in the description of the

Figure 3 : Trench section obtained by high frequency profiler

upper surface of the sea. It has been applied to one special land pipeline problem, that of settlement induced by differential thawing of permafrost.[6] In general, though, systematic study of seabed roughness is only now entering the descriptive stage, and quantitative description is still in the future.

The bathymetric information provided by precise echo sounding is supplemented by the picture of the sea bottom given by a side-scan sonar, which is becoming more sophisticated. Side-scan records make it possible to identify major obstacles, such as large boulders and wreckage, and to identify unburied cables and existing pipelines. An experienced interpreter can gain a good idea of the 'texture' of the bottom.

BOTTOM SOIL: UNBURIED PIPELINES

Some pipelines are simply lowered or pulled into place on the bottom, and no attempt is made to bury them. A pipeline is relatively light: a typical large oil pipeline has a submerged weight of 4kN/m, while a small gas flowline might weigh only 100N/m. Because of this, the influence of the pipeline does not extend far into the seabed, and little settlement occurs. Figure 4 shows a pipeline on the seabed: it is a 914mm (36 inch) pipeline, weighing 460N/m, on sand in the northern North Sea.

Settlement into the seabed is influenced by the method of installation. Pulling tends to groove the bed under the pipe, and laying from a laybarge induces a concentrated reaction of the order of 100kN, close to the touchdown point at which the pipe touches the bottom, and both effects lead to more settlement than would occur if the pipe were uniformly lowered into place. Large settlements only occur in very soft sediments in deltas and lakes. The density and undrained shear strength of these materials can usually be determined by conventional soil mechanics techniques. In one instance, during a plough test in a dredged arm of a delta, investigation showed part of the bottom to be covered by an ooze which was so soft that rheological techniques had to be used to study its behaviour. They showed that the ooze behaved as a Bingham fluid with a shear strength of about $10N/m^2$.

A pipeline must be laterally stable under the action of currents. The hydrodynamic forces acting on it are calculated from the Morison equation, using empirical lift, drag and inertia coefficients. The resistance to lateral movement provided by soil under the pipe must be enough to maintain it in place against the most unfavourable combination of lift and drag.

The simplest way of assessing lateral resistance, is to treat the contact between the soil and pipe as one governed by Coulomb friction, and to assert that the pipe will be stable if the ratio of the horizontal reaction to the vertical reaction exerted by the bottom does not exceed a limiting lateral friction coefficient, usually in the range 0.5 to 1. In the case of a pipe resting on rock, this idealization is a good one. In the case of a pipe resting on soft material, it is more open to criticism, though it has proved satisfactory in practice. A more sophisticated analysis of lateral resistance is based on proper soil mechanics. If the bottom is clay, the descriptive properties are the undrained shear strength and density. The undrained strength is appropriate because wave loading is of so short a duration that significant redistribution of water cannot occur, even on a small scale. If the bottom is sand, the density, internal friction, cohesion and degree of dilatancy of the soil are relevant.

As far as lateral resistance to movement is concerned, it is the topmost layer of the bottom that is most important. For instance, sand covered by a few centimetres of pebbles will give the pipeline a resistance quite different to what it would be on plain sand. Some soil survey techniques do not reliably locate thin surface layers, and are in that sense inadequate.

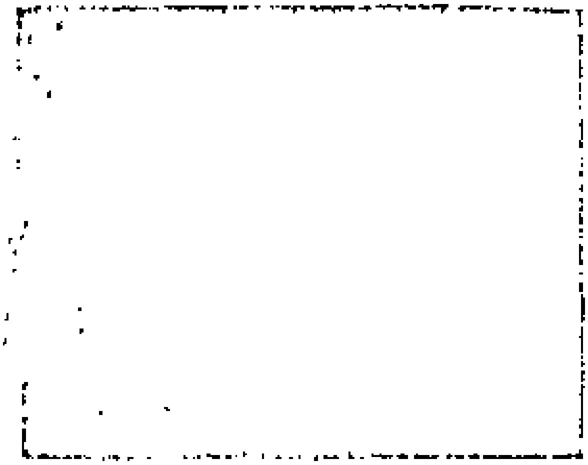

Figure 4 : Pipeline on sandy bottom

Few measurements of lateral resistance have been made, although it is a fairly simple experiment to carry out. The exceptions are *ad hoc* tests on particular sites, mostly unpublished, and the work of Lyons.[7]

A pipeline resting on sand or silt can be subject to severe settlements induced by liquefaction. Wave or earthquake loadings induce alternating stresses in the seabed, which can cause the pore pressure to increase, to a level at which the effective stress is so far reduced that resistance to shear deformation becomes very small, and the pipeline sinks into the bottom under its own weight (or if it is buried, and lighter than the liquefied soil, floats up to the soil surface). Early research on the phenomenon concentrated on the effects of earthquakes,[8] but recently there has been more interest in wave-induced liquefaction.[9,10] The time scales are quite different. Earthquake loading is relatively intense, but lasts only a few seconds, so that there are few cycles and little time for pore pressure to dissipate. On the other hand, a storm builds up over several hours, so that the cyclic shear stresses in the seabed increase as the oscillatory wave-induced loading on the bottom increases. As the cyclic shear stress intensity increases, pore pressure begins to build up, but at the same time a steady diffusion of pore pressure begins. Whether or not the material reaches a liquefaction conditions depends on the balance between the rate of increase of wave height (and therefore the rate of increase of pore pressure), and the diffusion and consequent reduction of pore pressure allowed by permeability. Analysis is obviously complicated, and has not yet got far. It is made more difficult by the fact that pore pressure generation by shear stress is an inherently nonlinear phenomenon, and cannot logically be put into linear models of diffusion and soil stress-strain relations. Wroth[11] has emphasised the need for care to avoid models that violate the effective stress principle. Ishibashi, Sherif and Tsuchiya[12] have put forward an analytical model of pore pressure generation, expressed by

$$\Delta U_N(1 - U_{N-1}) = (\tau_N/\sigma'_{N-1})^{1.4}\, f(N)$$

where U_N is a dimensionless pore pressure (normalised with respect to the initial effective confining pressure), ΔU_N is the dimensionless residual pore pressure increase in the N-TH cycle, τ_N is the cyclic shear stress applied in the N-TH cycle, σ'_N is the mean principal effective stress, and f(N) is a function expressing the fact that the effect of a given stress cycle diminishes with increasing total number of cycles. Empirically, from their experiments on Ottawa sand

$$f(N) = 6.13N/(N^{1.7} - 0.46)$$

This is a relatively complex idealisation, but is still not sophisticated enough to allow for the effects of drainage.

Liquefaction needs much more study analytically in the laboratory and in the field. From the site investigation point of view, it is still not clear which parameters are the most significant. It used to be thought that only loose deposits would liquefy, and that the critical parameter would be the voids radio or porosity.[13] Loose deposits can be identified by in-situ cone penetrometer measurements.[14] However, recent work[15] has shown that even extremely dense sand, with a relative density of 100 per cent, can still liquefy and generate increased pore pressures, and that there may also be involved a second liquefaction mechanism, in which local negative pore pressures cause cavitation. The tendency of sand to liquefy must be strongly structure-sensitive, and so in-situ measurements are more likely to be appropriate than laboratory tests on samples, which are inevitably subject to disturbance, particularly during recovery from deep water.

BOTTOM SOIL: TRENCHED PIPELINES

Many marine pipelines are lowered into the bottom, to protect them from damage from fishing trawlers and anchors and to reduce wave and current forces. There are several ways of carrying out the required excavation. A trench can be dug by conventional dredging, and the pipe then placed in the trench: this is only feasible in relatively shallow water. A second technique is to excavate a trench by jetting, after the pipe has been laid. A sled is pulled by a barge, and pumps drive water down flexible hoses and through jets on a claw carried by the sled, on either side of the pipe, so as to excavate the bottom. A third method is to excavate the trench with a plough, pulled along the route either before or after the pipe itself is placed on the bottom. Several cutter-trenching devices have been developed, and one is under field trials at the time of writing.

The effectiveness of each method naturally varies between soils. Jetting works best in medium clay. In sand it makes a wide shallow trench, with side slopes of only a few degrees, so that the amount of pipe lowering achieved in each pass can be disappointing. A wide trench does not protect the pipeline very effectively, and natural backfill may be extremely slow. Ploughs can be used in sand or clay, and cutter devices are primarily intended for stiff clay. At their present stage of development, none of the above methods is effective in rock, and it may be necessary to break the rock by conventional blasting or by shaped charges, and then to remove the broken material by dredging.

The cost and efficiency of a trenching technique are determined by the rate of progress and the depth of trench achieved in each pass. When sand is jetted, the rate of progress depends on the in-situ density and the angle of internal friction. Each contractor has knowledge and experience of his own equipment, but little systematic work has been published; an exception is the work of Reynolds.[16] Plough design has been studied quite intensively. In clay, the governing parameter is the undrained shear strength, which determines the draught needed to pull the plough forward. In sand, several theoretical questions remain open, but it appears that the most important factors are the in-situ density, internal friction, and a dilatancy parameter describing the tendency of pore pressure to drop when the soil is sheared rapidly.

The depth of trenching is rarely more than 2m except on beaches and in surf zones. It is accordingly much more useful to have many shallow samples than a few deep ones, and simple gravity corers are used. They are supplemented by sub-bottom profiling, which detects the general structure of the upper 20m of the bed, and reliably identifies rock and major changes in sediment density, so that it aids interpretation between core locations. A rapid and cheap in-situ penetration test of the first 2m of the bed would be extremely useful. A plough can be towed at several knots, and the continuous record of plough draught gives a good indication of changes in the bottom, and so it has been suggested that a small plough might be a useful survey device.

All trenching methods are seriously affected by large boulders. This is particularly true of trenching along an existing pipe, because a boulder can jam between the pipe and a moving jet-sled or trenching plough, causing severe damage to the pipe and the weight and corrosion coating. A reliable device for detecting boulders hidden in the top one or two metres of the bottom would be of immense value.

SITE INVESTIGATION FOR DRAKE F-76 FLOWLINES

The following section of the paper describes the site investigation that provided environmental data for the design and construction of a flowline bundle, from the Drake F-76 gas well, in 60m of water in Byam Martin Channel off Melville Island, in the Canadian Arctic. The investigation was carried out in the early months of 1977. The well was drilled and the flowline constructed in the following year. The complete project is described elsewhere.[17] It was carried out for Panarctic Oils Ltd. as a demonstration to provide experience for the construction of the large offshore gathering systems that will be required when the Drake and Hecla gas fields are brought into production. Although the location was somewhat unusual, the investigation exemplifies some of the points discussed earlier in this article. The survey programme and construction schedule were much complicated by the fact that the sea at the site is ice-covered almost the whole year round, and by continuous darkness and extreme cold in mid winter.

The geology of the field made it possible to choose the exact well site reasonably freely. It had to be deep enough so that the top of the wellhead would be more than 45m below the surface: this is thought to be the maximum draught of the large ice islands that occasionally drift through Byam Martin Channel. The route had, as far as possible, to meet the following conditions:

1. Minimum length of flowline,
2. A suitable onshore area for make-up of the flowline bundle, which would be completed onshore and pulled into the sea,
3. A relatively steep shore crossing, which was desirable to minimise the length that would need special protection against floating ice.
4. A flat and unobstructed area around the well site: this was needed for the final connection to the wellhead, which involved pulling the end of the flowline sideways and sweeping it across a wide area of the bottom.

At the start of the planning phase, there were available some bathymetric data obtained incidentally during earlier seismic investigations. Nothing was known about the currents. There had been some offshore drilling in the area, and that had indicated that the bottom would probably be soft clay, derived from decomposed shale on the island and carried out to sea by a seasonal river nearby, during the short summer runoff period.

A reconnaissance survey was carried out in late January 1977. Its objective was to obtain further bathymetric data to carry out a topographic survey of the shore, and to establish control points for the second phase of the survey. At the end of the reconnaissance, and after a review of several alternative wellhead sites and pipeline routes, it was agreed to concentrate attention on the route shown in figure 5.

The second phase of the survey was carried out in March and April 1977. Two sets of current meters were installed, in the locations marked in figure 5, and left in place from March 12 to April 22. Each set consisted of three Anderaa recording meters, two of them placed about 2m above the seabed and the third just below the ice. Only three of the meters operated correctly throughout the recording period: this emphasises the need for redundancy in current meter installations. The maximum current recorded was 0.2m/s. It was not possible to record current direction, because the proximity of the north magnetic pole makes it impossible to use the earth's magnetic field as a reference direction in that area, and in subsequent design work it had to be assumed that the maximum current occurred in the least favourable direction, perpendicular to the pipe.

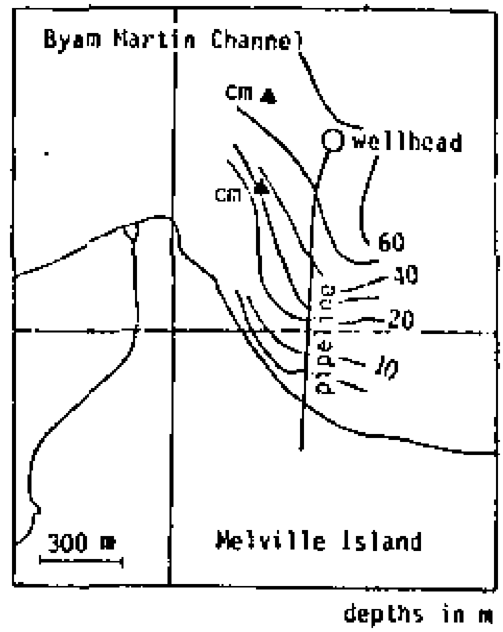

Figure 5 : Site plan : Drake F-76 flowline

Detailed bathymetry was obtained by an Edo Western echo sounder. Within a 300m wide corridor straddling the centre line, soundings were taken at 25m spacings along seven lines, 50m apart. Within a 100m corridor straddling the centre line, soundings were taken at 12.5m spacings along five lines, 25m apart. This information was used in the preparation of detailed bathymetric maps. A Raytheon RTT 1000 sub-bottom sounding unit was used to obtain soundings at 25m spacings along the corridor centre line until close to the wellhead. Good penetration was obtained, to about 20m. The records showed no evidence of rock or permafrost close to the sea bottom. The bottom was also surveyed by an EGG side-scan sonar system. This system could not be deployed in the conventional way, but was instead lowered through a hole in the ice and rotated. The records showed a moderate amount of ice scour in shallow water, and occasional scours in deep water.

A series of bottom soil samples were obtained by a Benthos gravity corer, dropped from 0.5m above the bottom: the corer penetration was usually about 1.5m. The temperature of the bottom of each core sample was measured as soon as it was recovered, and an approximate measurement of the shear strength was obtained by a hand-held Torvane instrument. The sample was then logged and shipped south for laboratory tests. A number of boreholes were also made, to about 10m below the bottom, and thermistor strings were installed to measure the temperature in the seabed, to locate the permafrost boundary. A similar investigation was carried out on shore.

The seabed soil samples turned out to be unusually uniform, and to consist of a soft highly-plastic olive-black organic silty clay, with occasional silt lenses at intervals of about 0.1m. In figure 6, liquid limit and plastic limit are plotted against the distance from the shore, and the same figure includes measured water contents at 0.5 and 1m below the bottom: there appears to be no

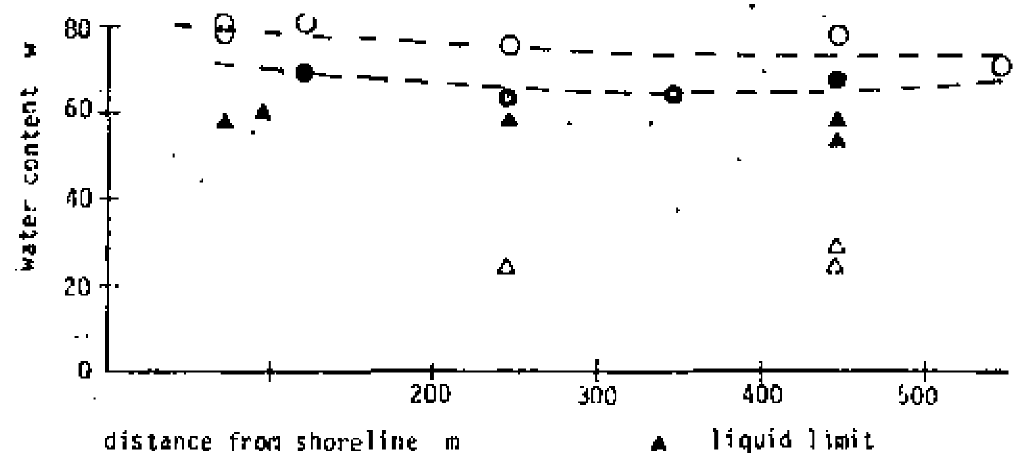

Figure 6 : Dependence of soil properties on distance from shore

systematic variation with position. In figure 7, measured undrained shear strength is plotted against depth below the bottom. There is substantial scatter, and one has to ask if this reflects a true variation in shear strength and water content, or if instead the seabed is actually homogeneous with a regular increase of strength with depth, and the scatter really reflects the influence of varying degrees of sample disturbances and errors during testing in unfavourable conditions. The second conclusion appears to be the correct one, and it was accordingly decided to base design of the shore crossing on the relationship between undrained shear strength and depth indicated by the dotted line in the figure, which is drawn through the mean value for each cluster of points.

It was clear from the results of the survey that the bottom soil was extremely weak, with a shear strength of 3kN/m². Moreover, the laboratory tests indicated some degree of sensitivity, so that the remoulded shear strength was about 0.6 of that of the samples from the corer. The flowline bundle had to be in as deep a trench as possible from the shore to 20m depth, so that it would be protected from damage by ice. A simple calculation shows that it is not possible to make a very deep trench in such a material, because it will collapse under its own weight. It was obviously important to avoid trenching methods which would remould the seabed and further reduce its strength. After examination of the alternatives, it was decided to plough a trench 1.5m deep (which approaches the limit at which the trench collapses) and to design a special plough which would disturb the surrounding soil as little as possible.[18] The trench was successfully ploughed in March 1978.

In retrospect, it would have been desirable during the survey to carry out in-situ measurements of the shear strength of the bottom soil, by a vane-shear device operated from the ice above, or perhaps by a pressurement or cone penetrometer. In-situ measurements would have increased confidence in the design shear strength for the plough and the trench. Experience with the plough in indicated that the actual strength at the bottom may in fact be higher than 3kN/m². A check was

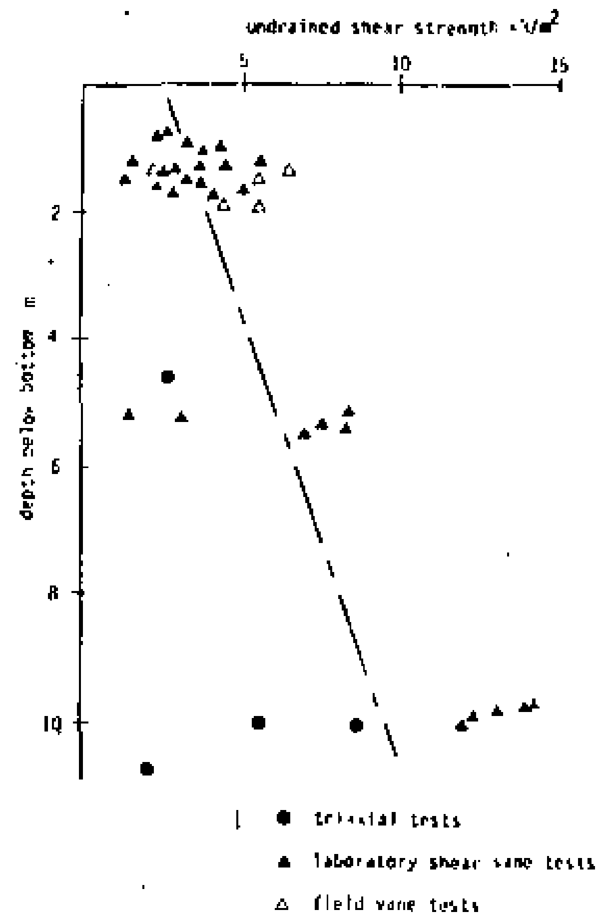

Figure 7 : Undrained shear strength as a function of depth below bottom

made on the stability of the submarine slopes in the area, and indicated that the strength of the bottom could not be much less than 3kN/m².

CONCLUSION

It may be concluded that a site investigation for offshore pipelines should never be considered in isolation from the rest of the construction programme, and that close collaboration with the ultimate users of the data is essential if resources are to be used effectively.

ACKNOWLEDGEMENT

The author thanks R.J.Brown and Associates AG for permission to publish this paper.

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DISCUSSION

Mr H McClelland (McClelland Engineering Inc): Dr Palmer cautioned against correlating cone resistance with liquefaction since even at high resistance (low porosities) there was still a possibility of liquefaction. Could he enlarge on this?

Dr A C Palmer: I have an open mind on this. The early papers on liquefaction argue that below a certain porosity there is no need to worry. Yet recent investigations show that the application of shear stresses to sands well below that value of porosity still gives rise to a major build-up of pore pressures which could lead to liquefaction. That makes me worried in saying that a very dense sand cannot liquefy.

Mr P Broughton (BNOC): I was slightly disappointed in Dr Palmer's paper in that it does not indicate the influence of the measured soil parameters on the design of the pipeline itself and on the design of the jetting or cutting equipment for doing the trenching. I would also be interested to know whether the jetting equipment used in the Gulf of Mexico is really adequate for the work currently being carried out in the North Sea.

Dr Palmer: I felt that a discussion of the application of the site data to such problems as, for example, the lateral resistance of pipelines was outside the scope of my paper. We use a combination of what is known from soil mechanics analysis and from tests that have been made.

With regard to trenching and jetting most of my experience has been with ploughs where the determining parameter is the undrained strength. Ploughs in sand are much more complicated and much more interesting since the speed affects the rate of drainage of the sand and hence its deformation and strength properties.

Mr Broughton: Has a vibrating plough been used in order to liquefy the sand?

Dr Palmer: It has been tried on land and there are controversial views about it. It has not been tried offshore because vibration introduces so many difficulties in operation.

As far as jetting is concerned I think it suffers from a lack of proper analysis as it cannot be that complicated a phenomenon. At present there appears to be alot of folklore about it. I think this is an area that warrants respectable scientific investigation.

Dr L H King (Bedford Institute of Oceanography): I am curious about the sidescan results in relation to iceberg furrows which I should think would be the first hazard in this environment. How does the depth of these furrows relate to the depth of burial of the pipe?

Dr Palmer: We expected to find a lot of evidence of ice scour at the bottom of the bay but there was almost nothing to be seen. Whether this is because it is rather a sheltered area, or because the bottom is so heavily remoulded by ice the whole time that there is nothing to be seen we do not know, but I rather think the former. The pipeline has been in one summer so far. Time lapse photographs were made every three hours during the break-up period in July and August and all that happened was that the ice out in the bay broke up into lumps. It was all very quiet and still. Then in September the ice began to form again and became land fast. We have to wait and see what happens in the long-term.

The pipe is buried out to a depth of 20 metres. It is not just trenched, but it is protected within a core of artificial permafrost. Beyond this it lies on the bottom.

General discussion
Chairman: Dr J B Burland
Panel: Dr K L Taylor
J de Ruiter

Chairman: At the start of his talk Dr Palmer made an interesting point picking up on Mr Muir-Wood's opening remarks. We are discussing the application of site investigation data to design and we must consider very carefully indeed whether the way the site investigation team is organised is adequate. Is it sufficiently integrated in general with the design team?

Mr J de Ruiter: I agree with the point made by Dr Palmer and it applies to offshore site investigations in general and not only to pipelines. The composition of the team and the team leaders are extremely important and can be decisive in the outcome of the investigation.

The worst mistake is to define an investigation programme in the office ahead of time without paying due regard to the possible variability of the soil conditions. The team leader has to ensure that the relatively short time he has at his disposal is spent wisely. Unlike the onshore situation it is not easy to come back again if something is missed.

My advice is therefore to keep the programme flexible, set general guidelines and leave it very much to the discretion of the people in charge, preferably on board, to vary the programme depending on what they find.

There are limitations on the numbers that can be accommodated on board. Sometimes the engineer for the consultant must also represent the oil company. This often works well but it is preferable to have a representative of the oil company on board who is aware of the overall problems involved.

There is a further limitation in that life on board a drilling vessel is none too pleasant. In my own experience after about two years the majority of engineers like to move back to shore leaving it to the new generation to work offshore.

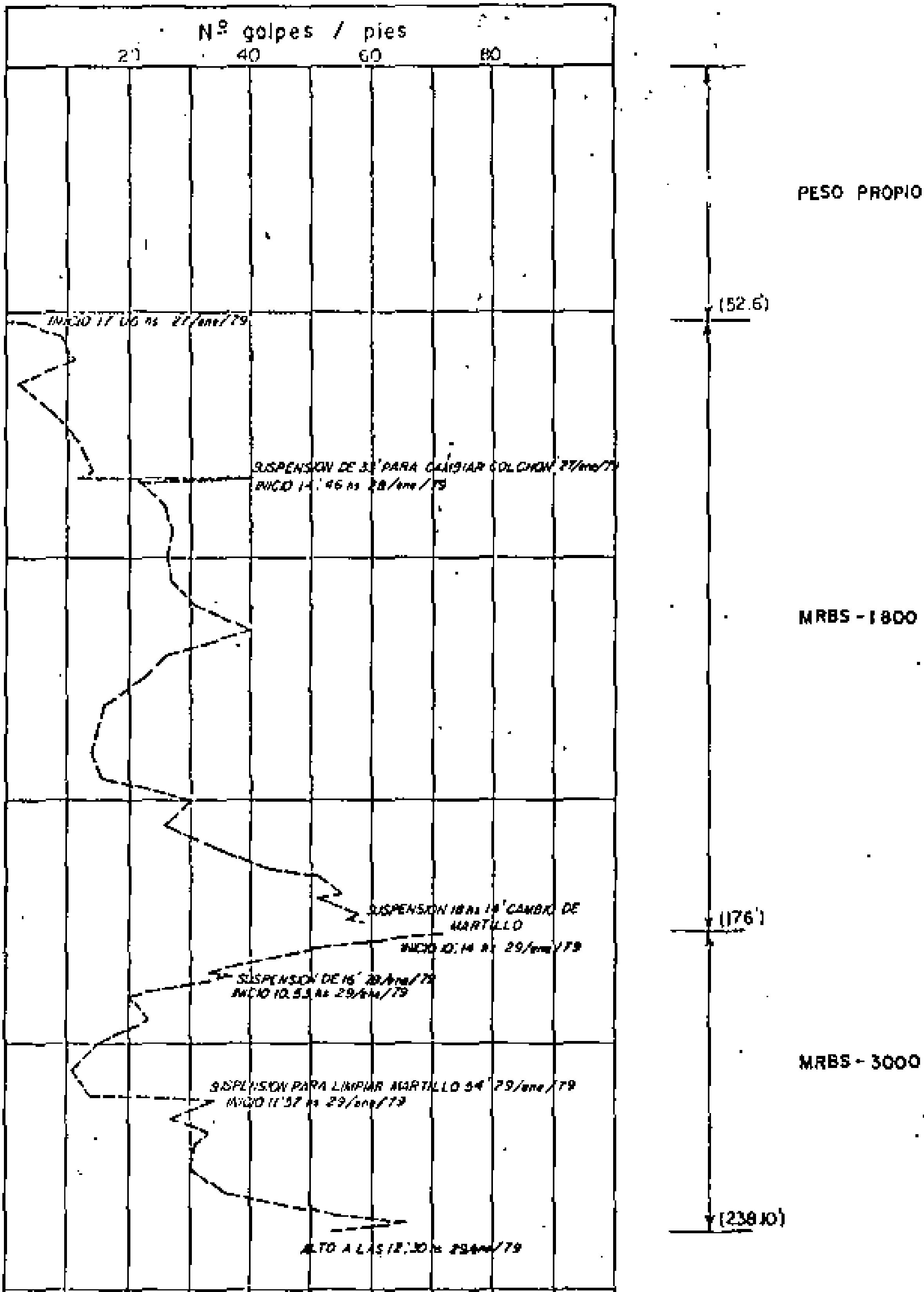
In summary the information collected during the investigation may have an important bearing on the interpretation of the results and a strong team will be an important factor in the overall success of the investigation.

Dr K L Taylor: I agree with all the points made about the field team. I would like to comment on the team doing the design. Take for example the design of a steel jacket structure. There is a real danger that the structural engineer will use numbers for the soil and plug them into an available 'black box', such as a P–Y analysis, without understanding what they are doing. He needs to go back to first principles and examine what the various soil parameters contribute to the analysis, consider over what range of values they might actually lie and how they might affect the performance. At the moment I do not think that the structural engineers really know what they are dealing with.

In reality the foundation design is an interactive process and the structural engineer cannot get the answers without knowing something about the soils.

Mr E Toolan (Fugro (UK) Limited): In my paper I made it clear that the P–Y curve is not a soil property and should not be used blindly. There is certainly a danger in using computer programs where a soil type is plugged in and forgotten about as it generates the answers.

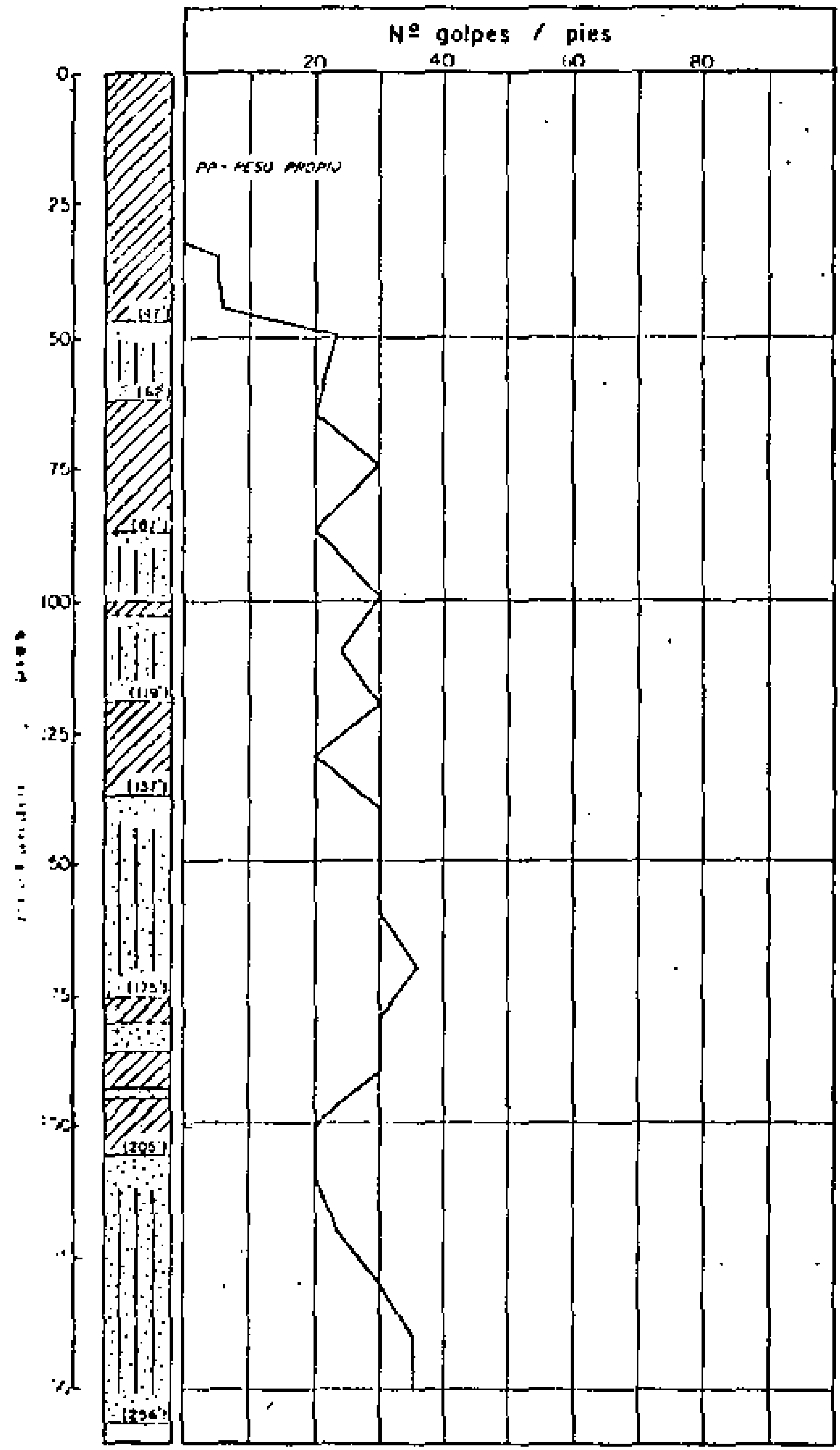
Penetración "Estándar" vs Energía de Hincado
Pilote de Ø 40"



(Plataforma Nohach B)

SEMINARIO INTERNACIONAL
DE
INGENIERIA OCEANICA

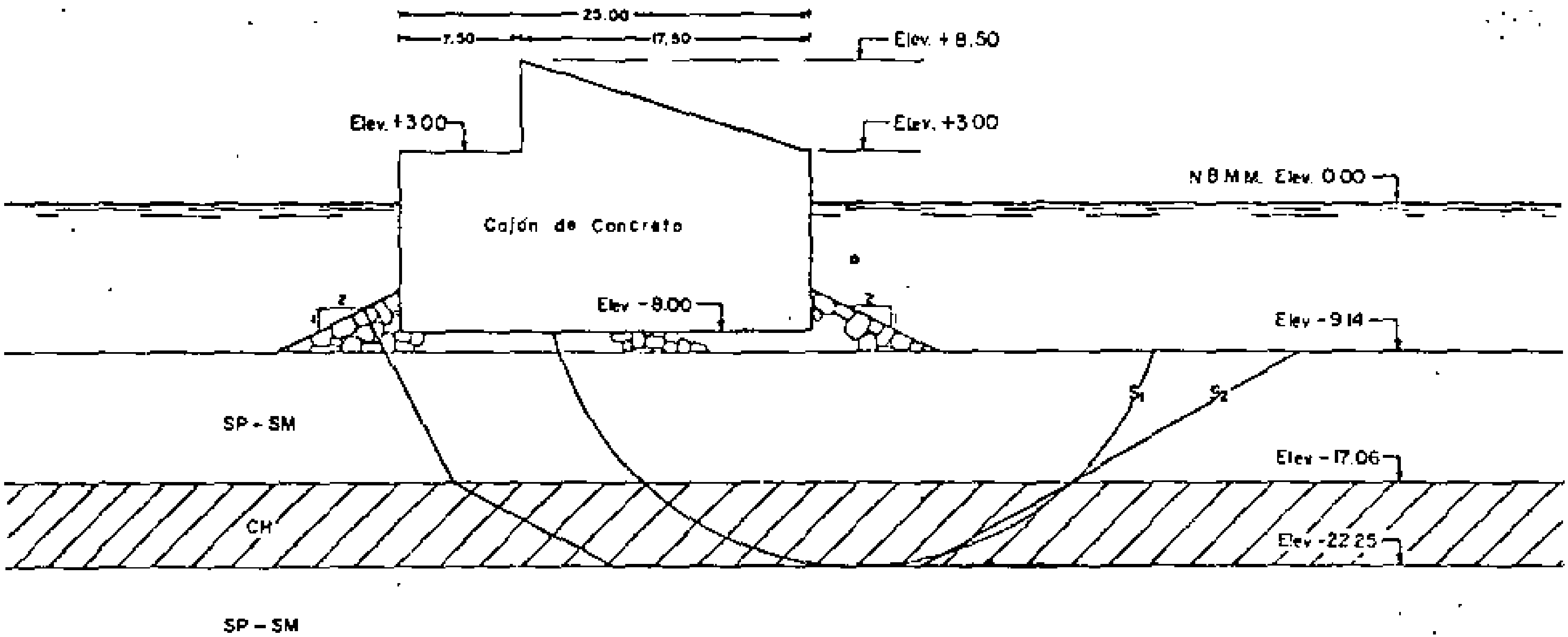
Penetración "Estándar" vs Energía de Hincado
Pilote de Ø 40"



(Plataforma Nohoch B)

SEMINARIO INTERNACIONAL
DE
INGENIERIA OCEANICA

Superficie	Factores de Seguridad			
	A Corto Plazo	A Mediano Plazo	Mediano Plazo c/carga de oleaje	Mediano Plazo c/ sismo
S_1	1.27	1.52	1.42	1.16
S_2	1.35	1.81	1.69	1.15

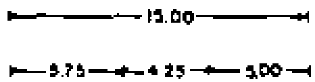


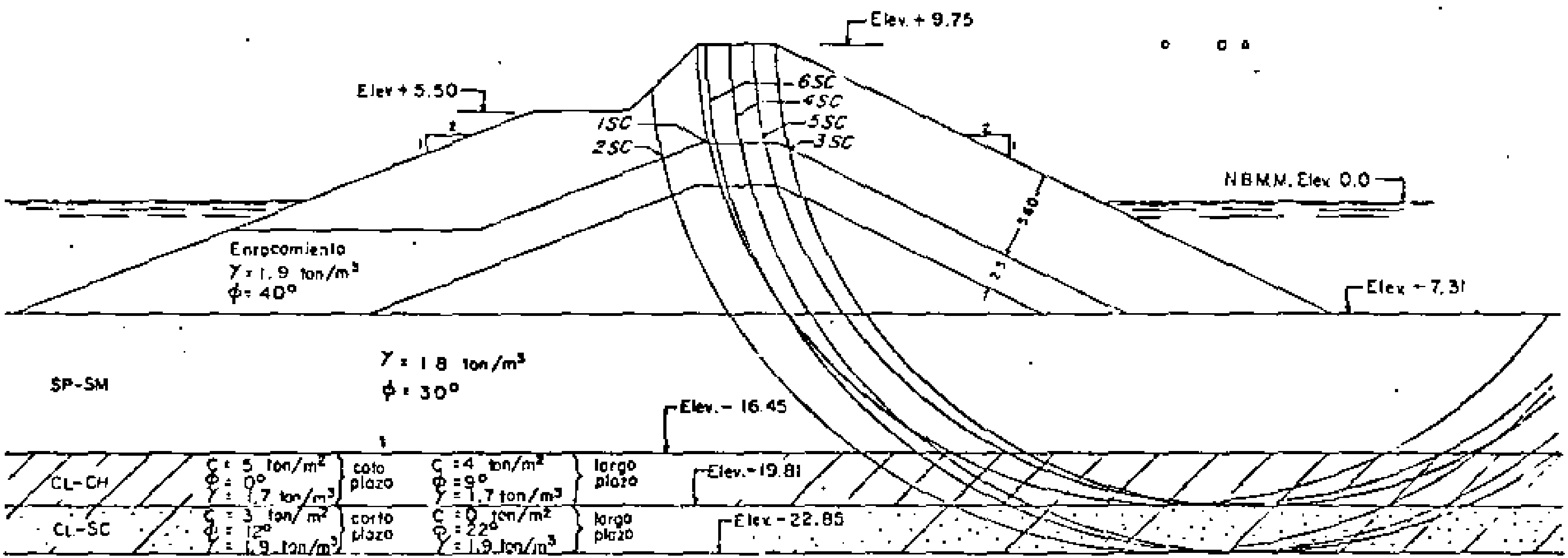
98

NOTA
- Escala 1 : 400
- Acotaciones y elevaciones en metros
- Aceleración Sísmica = 0.123

Fig. 18 Superficies de Falla y Factores de Seguridad Obtenidos para la Alternativa a Base de Cajones de Concreto en la Estratigrafía del Sondeo 12.

Círculo	Factores de Seguridad Bishop		
	Símbolo	Corto Plazo s/sismo	Largo Plazo c/sismo
1SC-A2	o	1.801	3.725
2SC-A2	o	2.057	5.158
3SC-A2	Δ	2.058	4.656
4SC-A2	Δ	1.935	4.054
5SC-A2	□	1.880	3.897
6SC-A2	□	1.934	4.262





ESC 1:400

Fig. 17 Superficies de Falla y Factores de Seguridad Obtenidos para la Alternativa a Base de Enrocamiento.

- Dragado
  - Excavación
  - Conducción
  - Depósito

- Tipo de Material
  - Suelos
    - Rellenos Inadecuados
      - Turba
      - Arcilla
    - Rellenos de Buena Calidad
      - Arena
      - Grava
      - Boleos
  - Roca
  - Coral

DIVISION DE EDUCACION CONTINUA
FACULTAD DE INGENIERIA U.N.A.M.

CURSO:

INGENIERIA OCEANICA

AMBIENTE MARINO

PROFESOR: R. SAENGER

MARZO 23, 1982.

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TRANSPORTE LITORAL

Es de suma importancia hacer análisis del arrastre de material que se efectua en una playa, especialmente, si la zona a considerar es la boca de una laguna o de un estuario, o si se planea la construcción de una obra marítima que modifique la configuración costera.

El movimiento de los granos de arena puede darse en dos direcciones: uno normal y otra longitudinal a la playa.

La forma en que los granos se mueven puede ser:

a) En la zona de lavado y generalmente en zig zag en dirección de la incidencia de oleaje; es por arrastre.

b) Por corrientes litorales provocadas por/hacia el ángulo incidencia del oleaje y se genera desde la primera línea de rompiente hasta la playa. Se produce en el fondo y por suspensión.

c) Por corriente de retorno, es en suspensión y hacia el mar.

La cantidad y dirección del transporte se puede evaluar por medio de:

I mediciones de campo

II fórmulas

III modelos

I) METODOS DE CAMPO

Los métodos directos de medición sirven para ajustar las fórmulas empíricas que se han desarrollado,

Es recomendable efectuar una serie de trabajos previos tales como una batimetría, análisis granulométrico, análisis de vientos locales, observación de oleaje y mareas y medición de corrientes.

Se procura obtener cuantificación del material que se mueve en el fondo como en suspensión a la vez que se toman mediciones de velocidad, graficando líneas de igual concentración y líneas de igual velocidad en perfiles perpendiculares a la costa. Esta medición deberá cubrir como mínimo un año.

a) Muestreadores y trampas

a.1) De material en suspensión

Se usan aditamentos que capturen o muestras de agua o solamente el material en suspensión

Los primeros son recipientes de volumen conocido que se cierran a la profundidad deseada.

Botellas Van Dorn

Botellas de Vacío

o por medio de mangueras de succión.

Los segundos son recipientes que se instalan a una profundidad en la que permanecen un cierto tiempo capturando el material que pasa por una abertura de área conocida.

Muestreador Bambú

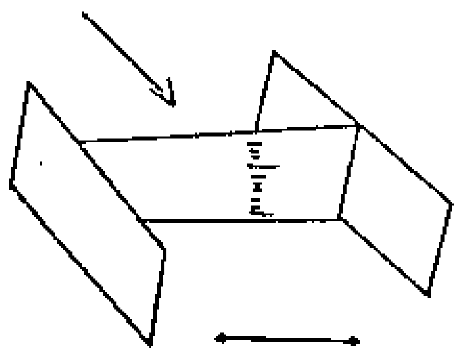
Botella direccional

a.2) Muestreadores de fondo

Son trampas de volumen conocido que se entierran hasta el borde superior el que es abierto a cierto tiempo y se calcula el tiempo en que acumula ciertos volúmenes de material. Pueden ser tambos de 200 lts.

Escolleras submarinas graduadas.

Son pequeñas estructuras que se sitúan perpendicularmente a la corriente y se mide el tiempo en el que acumulan la arena.



b) Mediante alteración del prototipo

Otra manera de cuantificar el arrastre es por medio de construcción de obras en la zona de estudio, ya sea por

Espigones ó
Dragado

b.1) Los espigones deben construirse considerando el ancho de la linea de rompientes. La altura máxima de ola predicha, deben ser impermeables y deben tener una vida útil de un año mínimo.

Se diseñan para que el material transportado, ya sea en suspensión o arrastre de fondo sea detenido.

La cuantificación se logra haciendo batimetrías antes de construirlo y después con lo que se obtienen los volúmenes atrapados.

El metodo altera las condiciones naturales y solo es costeable si se pretende construir una estructura mayor encima.

La cuantificación es a largo plazo.

b.2) Dragado este método es costoso y funciona como una gran trampa, el volumen se cuantifica mediante batimetrías. El inconveniente es que el material llega de todas direcciones. Se recomienda usarlo en obras ya construídas.

c) Mediante trazadores.

Los trazadores son partículas de arena obtenidas del lugar de estudio a las que se marca, para poderlas seguir. Son efectivos para informar la dirección de transporte y algunas veces la cantidad de transporte.

Se pueden marcar mediante métodos

Radiactivos ó
Fluorescentes

La técnica es sembrando los granos marcados y colectarlos a tiempos prefijados, sacando así lineas de concentración.

La recolección puede ser superficial usando tarjetas con algún aditivo que se apoyan en el fondo para fijar el material o haciendo núcleos en los que se analizan los trazadores por capas.

El conteo se hace para los fluorescêntes visualmente, los radiactivos mediante contador geiger.

Con los dos métodos se pueden realizar siembras simultáneas en unos cambiando el color y en otros las longitud de emisión de ondas gamma.

Cuando se recupera con núcleos el gasto Q se puede obtener que:

$$Q = P A V E$$

donde:

P = peso volumétrico

A = Ancho de la zona de transporte se obtiene sacando la distancia del diámetro mayor de la curva de isoconteo de valor menor. Es perpendicular a la trayectoria de los granos.

V = Velocidad de desplazamiento. Se obtiene el centro de la nube de isoconteo de un determinado tiempo. La distancia y el tiempo de la siembra al centro de isoconteo dan la velocidad.

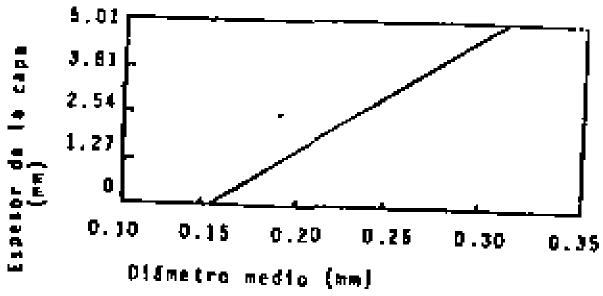
E = Espesor donde se da el transporte. Esta capa de arena se obtiene mediante el conteo en capas de los núcleos. Hay que tener en cuenta que parte de los granos marcados son cubiertos por otros naturales con lo que el espesor de esta capa aumenta.

<u>Espesor de la capa en que se produce el movimiento de fondo</u>.

Si consideramos que solo el material superior de la interface fondo-agua es el que está en movimiento constante se puede suponer que, puede ser más facilmente removido por el movimiento orbital de las olas.

El criterio de Einstein propone una capa límite igual a 2 veces el tamaño medio de la arena. Este criterio es bueno si no se considera la resuspensión.

El criterio de Ingle propone mediante una deducción empírica la siguiente tabla



El criterio de las concentraciones supone que la totalidad del material en suspensión proviene del fondo. La distribución vertical de dicha concentración la da

$$\text{Espesor} = K \sum_{i=1}^{n} \Delta H_i \, C_i$$

K es un coeficiente adimensional

$K = 1$ si C_i está en volumen

$K = \gamma/\gamma_s$ si está en peso

H_i, la distancia para la que la concentración Ci se mantiene

constante. (vertical)

SIMBOLOS

b = Subíndice que indica fondo

b = Distancia entre orogonales contiguas en la zona de rompientes (L)

b_o = Distancia entre ortogonales contiguas en aguas profundas (L)

E_b = Eficiencia de transporte de fondo (adimensional)

g = Aceleración de gravedad (9.81 m/seg^2 = 32.2 pie/seg^2) (LT^{-2})

m = Pendiente de la playa

n = Coeficiente de rugosidad de Manning

o = Subíndice que indica aguas profundas

q_s = Transporte litoral a lo largo de la playa por unidad de ancho (L^2T^{-1})

s = Coeficiente adimensional (fórmula de Castanho)

s = Subíndice que refiere al sedimento

C_n = Velocidad del grupo de olas (LT^{-1})

D = Diámetro del material

E = Componente de la energía del oleaje incidente, paralela y por unidad de playa (FT^{-1})

EC_n = Flujo de energía del oleaje en la zona de rompientes

H = Altura de ola en el punto de observación (L)

H_o = Altura de ola en aguas profundas (L)

I_l = Transporte litoral expresado en peso sumergido (LT^{-1})

K = Factor de proporcionalidad (adimensional)

k = Coeficiente de rugosidad $0.004 \leq K \leq 0.010$ (Castanho)

K_r = Coeficiente de refracción (adimensional) $K_r = (b_o/b)^{1/2}$

L = Longitud de onda en el punto de observación (L)

L_o = Longitud de onda en aguas profundas (L)

P_{tl} = Potencia del oleaje transmitida paralelamente a la playa (E/T)

Q_t = Transporte litoral expresado en volumen (L^3T^{-1})

T = Período de oleaje (T)

U = Velocidad orbital horizontal en el fondo justo antes de romper la ola (LT^{-1})

V = Velocidad de la corriente litoral (LT^{-1})

W_s = Velocidad de caída de las partículas en suspensión (LT^{-1})

α = Angulo de incidencia del oleaje

γ = Peso específico del agua (FL^{-3})

γ_s = Peso específico del sedimento (FL^{-3})

γ_v = Peso volumentrico del sedimento (FL^{-3})

n = Factor de grupo (adimensional) $= 1/2 \left(1 + \frac{4\pi d/L}{\operatorname{sen} h(4\pi d/L)}\right)$

ρ = Densidad del agua (ML^{-3})

ρ_s = Densidad del sedimento (ML^{-3})

ϕ = Angulo de la pendiente natural del sedimento

II METODOS EMPIRICOS

La obtención de una expresión matemática que represente el monto del transporte litoral, ha sido a base de correcciones empíricas de coeficientes.

Las investigaciones de este tipo, así como las formulas obtenidas son factibles de corregir y afinar con datos de campo donde se aplican en una área distinta a la que originalmente fueron aplicados. La correción generalmente es necesaria, ya que las condiciones de tamaño de grano, estadística de la población, densidad y forma de material regimen de oleaje y geomorfología son diferentes.

Las diferentes aproximaciones que se han logrado varían desde las que consideran solamente el oleaje producido localmente, hasta las que consideran corriente litoral, tamaño y forma del grano, oleaje distante etc.

a) Fórmulas que evaluan el transporte considerando solamente la energía del oleaje.

Este conjunto de formulas consideran el transporte total sin diferenciar si es en suspensión o de fondo y relacionan la energía del oleaje por unidad de longitud de playa con el volumen transportado en un intervalo de tiempo.

Esta formulas se desarrollaron para resolver un problema específico, algunas veces son aplicables sin hacer ajustes, pero es recomendable comparar con mediciones directas del campo y ajustar los coeficientes o los exponentes.

a.1) Cadwall

$$Q_s = 210\ E^{0.8} \qquad (1)$$

Q_s = esta dado en yardas cúbicas por día

E = Esta en millones de libras-pie por día y por pie de playa (Fig 1)

Para sistema métrico

$$Q_s = 2068\ \left[H^2\ \frac{L}{T}\ n\ \text{sen}\ \alpha\ \cos\alpha\right]^{0.8} \qquad (2)$$

Q_s en m^3/día

n = 1 para aguas someras

n = 1/2 para profundas

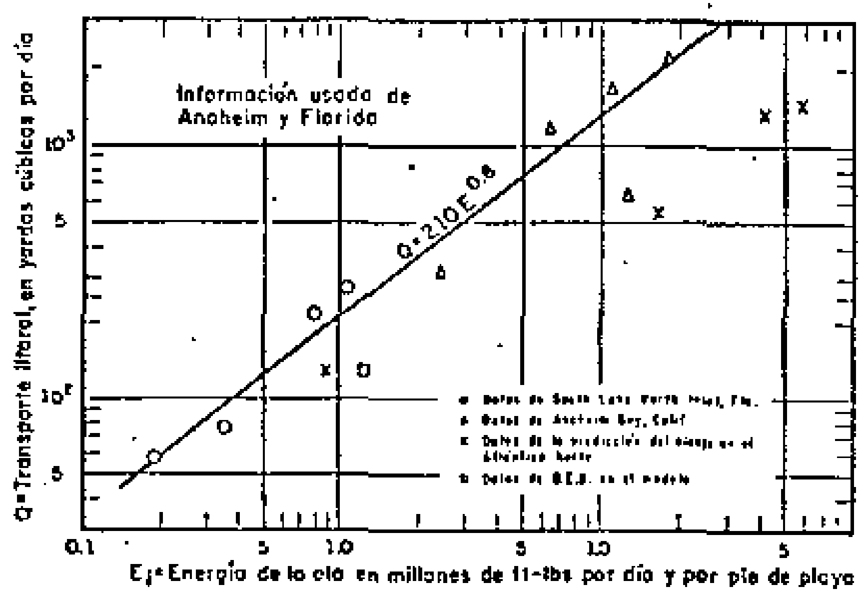


Fig .1 Relación del transporte litoral a la energía de la ola a lo largo de la costa (Cadwell) a.2) Coastal Engineering Research Center

El C.E.R.C propuso originalmente una ecuación similar a la anterior (Fig 2 y 3).

$$Q_s = 125\ E \qquad (3)$$

en la que Q_s y E estan en las mismas unidades.

Para sistema métrico

$$Q_s = 1819\ H_o^2\ T\ K_r^2\ \text{sen}\ \alpha\ \cos\ \alpha \qquad (4)$$

Después propusieron una modificación de la formula quedando

$$Q_s = 7.5 \times 10^{-3}\ P_{ls} \qquad (5)$$

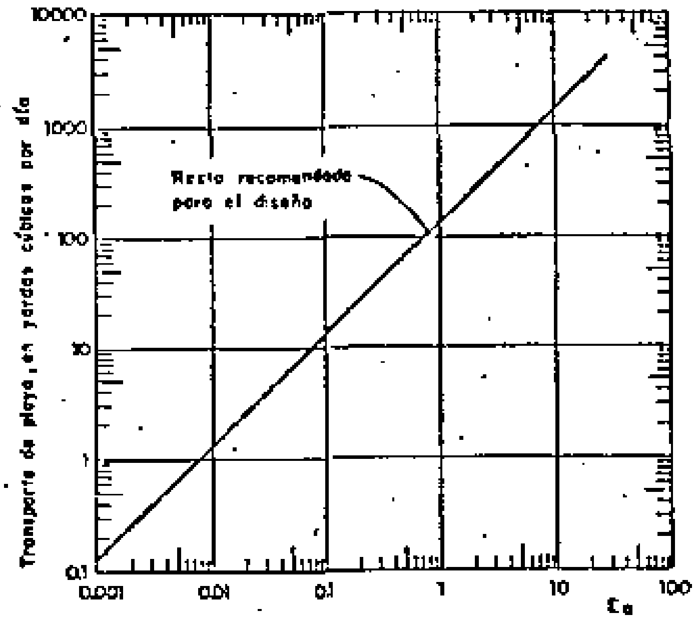
donde P_{ls} es el factor de flujo a lo largo de la costa (en pies-libra/segundo/pie de playa) y es proporcional al flujo de energía P, calculado para una onda periódica simple (teoría senoidal)

$$P = EC_g \qquad (6)$$

donde

E = energía específica de la ola = $\gamma H^2/8$

C_g = celeridad de grupo



E_a, componente de energía paralela a la playa en millones de pies x libra por día y por pie de playa

$E_a = \frac{E_o}{2}$ (número de olas al día)(sen α cos α) K_r^2

E_o, energía de la ola en aguas profundas

$$E_o = \frac{\gamma H_o^2 L_o}{8}$$

α, ángulo entre el frente de ola y playa en la rompiente

K_r, coeficiente de refracción

$$K_r = \sqrt{\frac{b_o}{b}}$$

b_o, distancia entre ortogonales contiguas en aguas profundas

b, distancia entre ortogonales contiguas en la rompiente

Fig 2 Relación entre la componente de energía paralela a la playa y el transporte de arena en la misma

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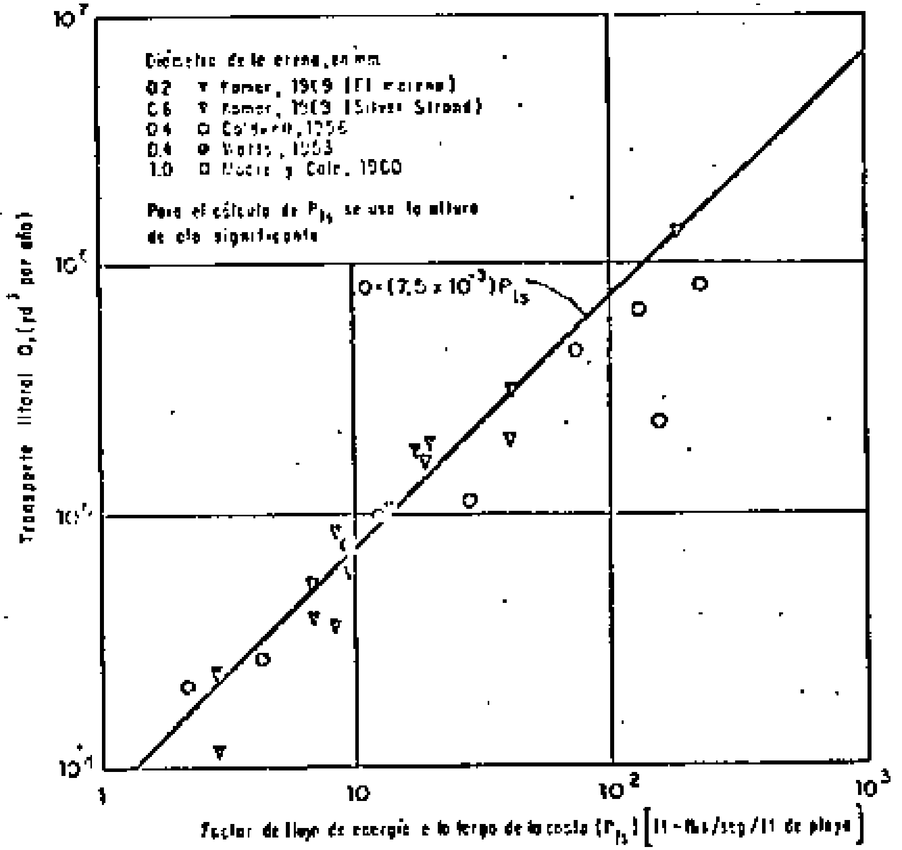

Fig .3a Curva de diseño para la relación del transporte litoral con el factor de flujo de energía. Unicamente se incluyen datos de campo

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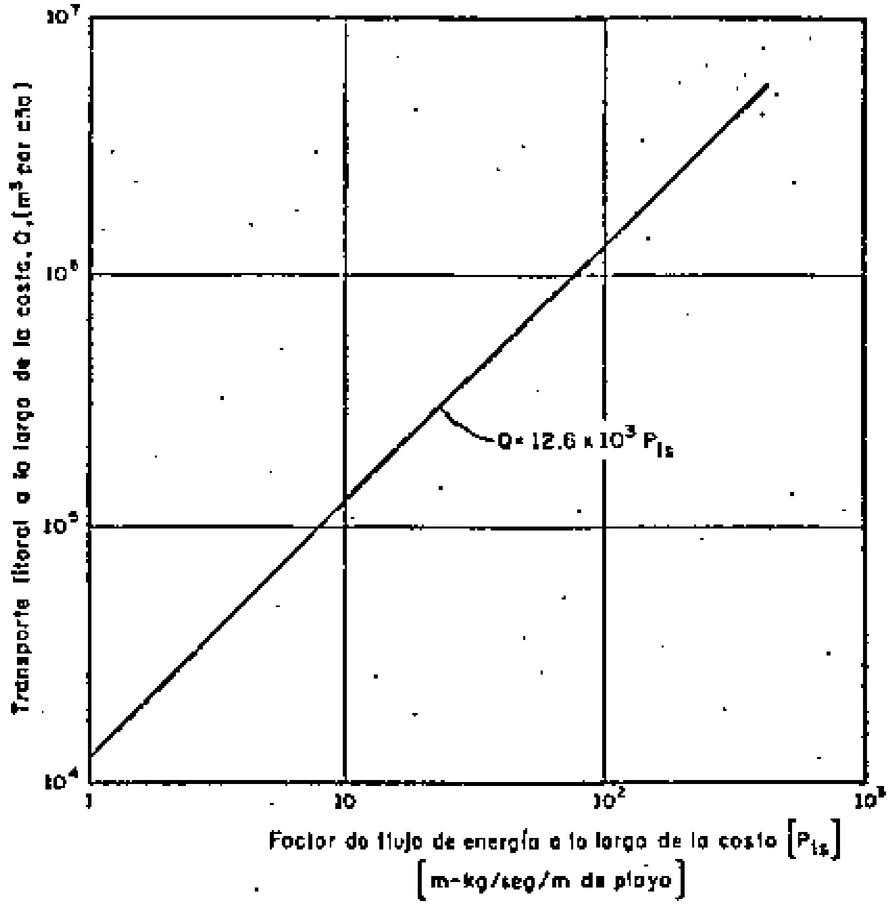
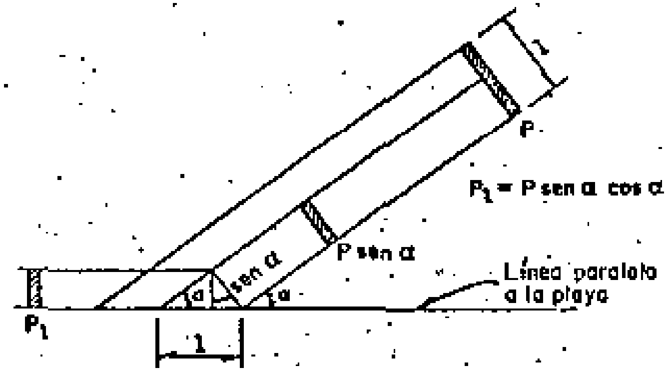


Flg .3b Curva de diseño propuesta por el CERC (1973)

Para el flujo paralelo a la costa P_1 tendremos

$$P_1 = P_{\cos\alpha}\ \text{sen}\ \alpha \qquad (7)$$



Se debe considera el lugar donde se midan las características del oleaje para obtener el valor de P_1 si es en aguas someras (rompientes) o en aguas profundas (¿antes de la rompiente?), para aplicar la teoría senoidal o de la onda solitaria.

La tabla siguiente hace un resumen de la obtención de datos y de la aproximación de teoría de oleaje requerida

Datos medios	P(pie-libra/seg/pie de playa)	
H_b, α_b	$32.1\ H_b^{5/2}\ \text{sen}\ 2\ \alpha_b$	(8)
H_o, α_o	$18.3\ H_o^{5/2}\ (\cos\alpha_o)^{1/4}\ \text{sen}\ 2\alpha$	(9)
$T, H_o, \alpha_o, \alpha_b$	$20.5\ T\ H_o^2\ \text{sen}\ \alpha_b\ \cos\alpha_o$	(10)
T, H_b, α_o	$100.6\ (H_b^3/T)\text{sen}\ \alpha_o$	(11)

Al utilizar las ecuaciones (5),(7), y (10) en sistema metrico tenemos

$$Q_s = 3456\ H_o^2\ T\ K_r\ \text{sen}\ \alpha\ \cos\alpha \qquad (12)$$

Si comparamos (12) con (4) tenemos que el coeficiente es 1.9 mayor en (12)

a.3) Watts

$$Q_s = 280\ E^{0.9} \qquad (13)$$

a.4) Lee

$$Q_s = KE^{-.97} \qquad (14)$$

Estas expresiones (13) y (14) estan en sistema inglés.

a.5) Komar e Inman

$$I_1 = K E \bar{C}_n \operatorname{sen} \alpha \cos \alpha \quad (15)$$

El transporte I_1 esta en peso sumergido transportado en unidad de tiempo.

K es adimensional e igual a 0.77 lo que facilita el uso en sistema inglés o en métrico

Para convertir I_1 a volumen se usa

$$S_1 = I_1/(\gamma_s - \gamma)a' \quad (16)$$

donde $a' = 0.6$

a.6) Castanho

$$Q_s = \frac{P_{t1}}{\gamma_v} \quad \frac{\gamma_s}{\gamma_s - \gamma} \quad \frac{S}{\tan \phi} \quad (17)$$

donde

P_{t1} = se calcula de

$P_{t1} = 2.2\ \gamma\ H^3_b$ sen α [illegible] α_b/T (onda solitaria) (18a)

$P_{t1} = (\gamma/16)\ H^2_o\ L_o$ se[illegible] cos α_o/T (Onda senoidal) (18)b

s se obtiene de

$$s = 1.93\ (107\ E - E^2) \quad (19)$$

$$E = -1.72\ A\ ((1.72\ A)^2 \ [illegible]\ 8)^{1/2} \quad (20)$$

$$A = (m\delta)\ /\ (K \tan \alpha) \quad (21)$$

m = pendiente de [illegible] ya

δ = esbeltez de l[illegible] = Ho/Lo

K = rugosidad 0.0[illegible] ≤ 0.010

o bien s se puede obtener d[illegible] gura 4 obtenido ya el valor de A

a.7) Larras

$$Q_s = Kg\ \ H^2\ T \operatorname{sen} [illegible] \quad (22)$$

Q_s está en m^3/seg

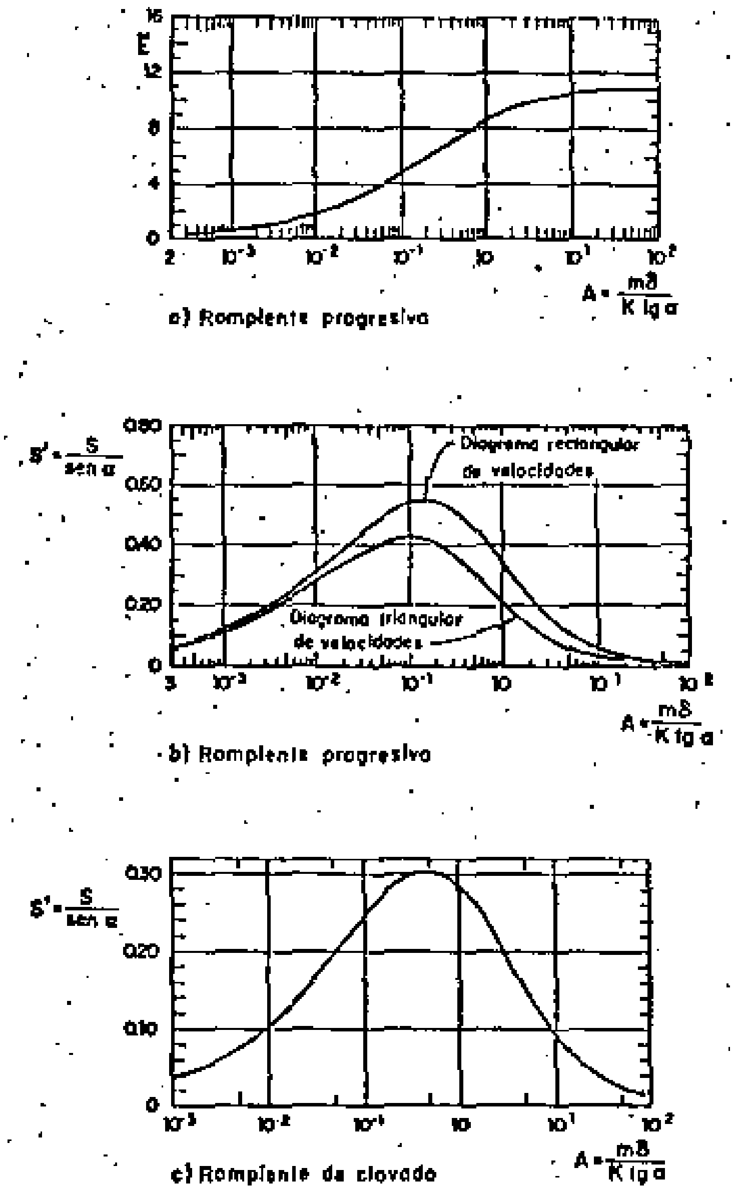


Fig 4

$$k = 1.18 \times 10^{-6}\, D^{-1/2}\, L_o/H_o \qquad (23)$$

dependiendo de la esbeltez de la onda $\delta = H_o/L_o$ expresada en metros y el D diámetro del material en milímetros.

a.8) Pychkine

$$Q_s = 1.2 \times 10^{-6}\, H^2\, L\, (g/D)^{1/2}\, sen\, 2\,\alpha \qquad (24)$$

Q_s = m^3/seg si H, L y D están en metros

a.9) Bonnefille y Pernecker

$$Q_s = K \left(\frac{H_o/L_o}{2.75}\, D \right) \frac{H_o^3}{T}\, \frac{f(\alpha)}{f(20°)} \qquad (25)$$

K depende de δ y del diámetro se obtiene en la figura 5.

b) Fórmulas que implican las corrientes playeras

La aproximación del oleaje a la playa con un cierto ángulo

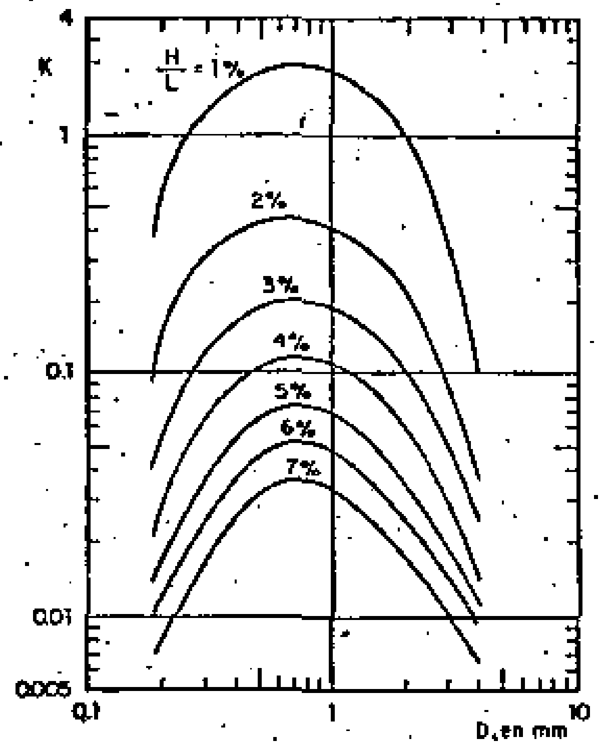


Fig 5 Factor K de la fórmula de Bonnefille y Pernecker, en función de la relación de esbeltez y del diámetro del material

de incidencia genera una corriente a lo largo de esta y con una velocidad relacionada con la energía del oleaje.

La sola velocidad, descontando el efecto de fondo del oleaje, es capaz de provocar un transporte en el fondo y resuspendiendo el material.

Por otra parte, el oleaje, por su movimiento orbital en el fondo también resuspende y transporta las partículas cada vez que pasa una ola.

Las fórmulas siguientes consideran esto:

b.1) Inman y Bagnold

$$I_l = K' (E\, C_u)_b \cos \alpha_b \quad V/v_m \tag{26}$$

K' es adimensional e igual a 0.28 lo que facilita el uso indiscriminado del sistema de unidades.

El coeficiente K' considera

El movimiento orbital es producido por oleaje sin que se genere transporte neto.

Una corriente que se añada al movimiento orbital sí produce arrastre neto.

La energía disipada al poner la arena en movimiento es proporcional a:

> El flujo de energía por longitud de playa y a la velocidad de fricción relativa en el fondo en las rompientes.

La velocidad de fricción es proporcional a la componente horizontal máxima de la velocidad orbital cerca del fondo, justamente antes de romper la ola.

$U_m = gH/2C \cosh (2\pi d/L)$ (27) y se obtiene de la teoría senoidal.

b.2 Einstein corregido

$$Q_t = Q_b \,(1 - I_1 \log_e (33\, h/r) + I_2)\, B$$

El gasto sólido está en m^3/seg y se calcula considerando el arrastre de fondo Q_b

$$Q_b = 5 \pi \, (\mu\tau/\rho)^{1/2} \, \mathrm{Exp}\, (-0.276\, D\gamma/\mu\tau)$$

$$\tau = (1 + (\xi U_0/v)^2/2)\tau_c$$

$$\tau_c = \gamma v^2/C^2$$

$$\xi = 0.0575\, C$$

I_1 e I_2 se obtienen de las figuras 6.a y 6b en función de x y A

- h = profundidad
- r = la semialtura de los ripples o rizos
- b = ancho de la corriente litoral
- D = diámetro de la arena
- μ = coeficiente en el esfuerzo cortante total τ debido a los ripples
- τ = esfuerzo cortante total (oleaje y corriente)
- τ_c = esfuerzo cortante por corriente
- A = densidad relativa

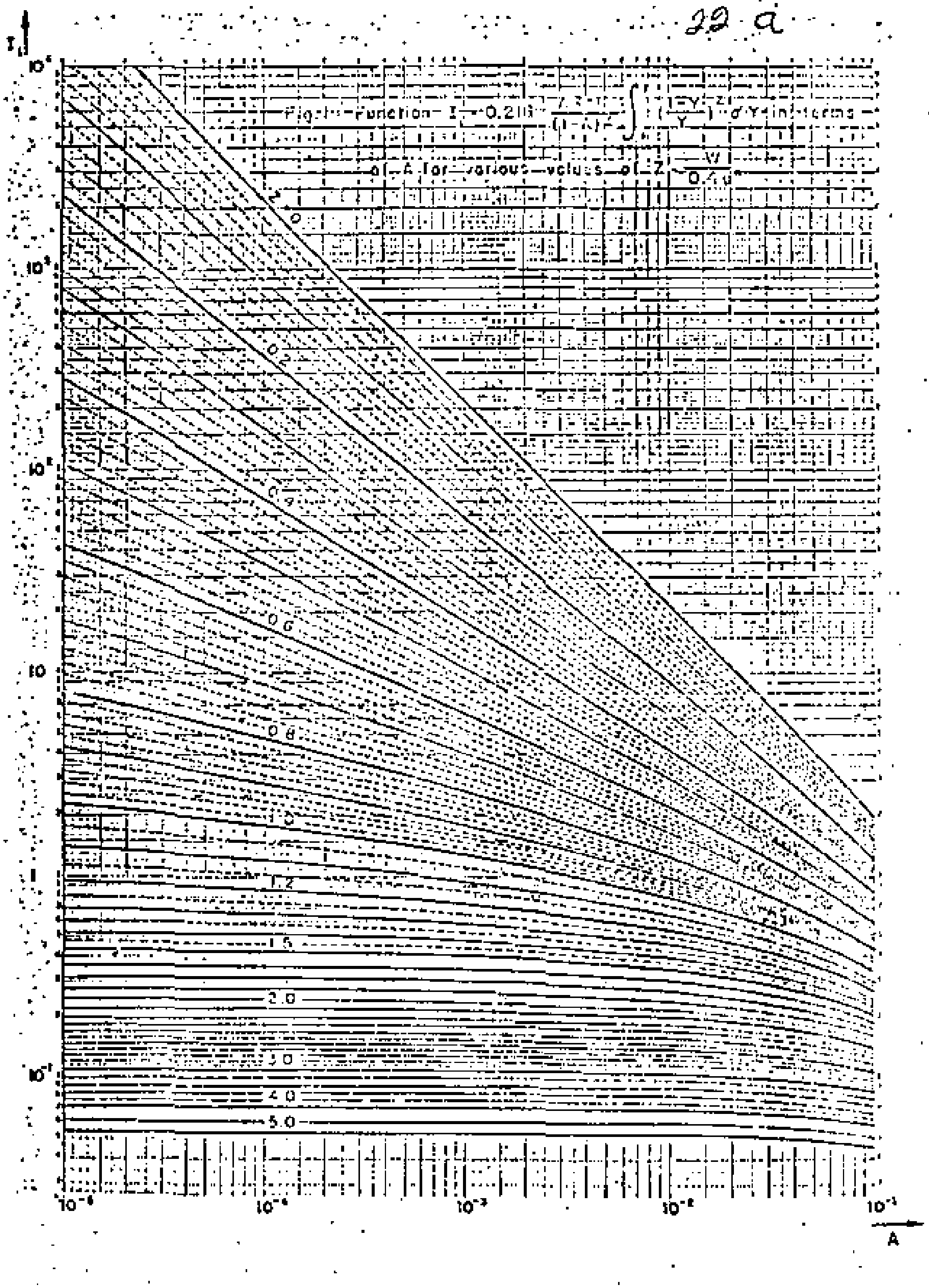


Fig . 6a Valores de I_1 empleados en el método de Einstein

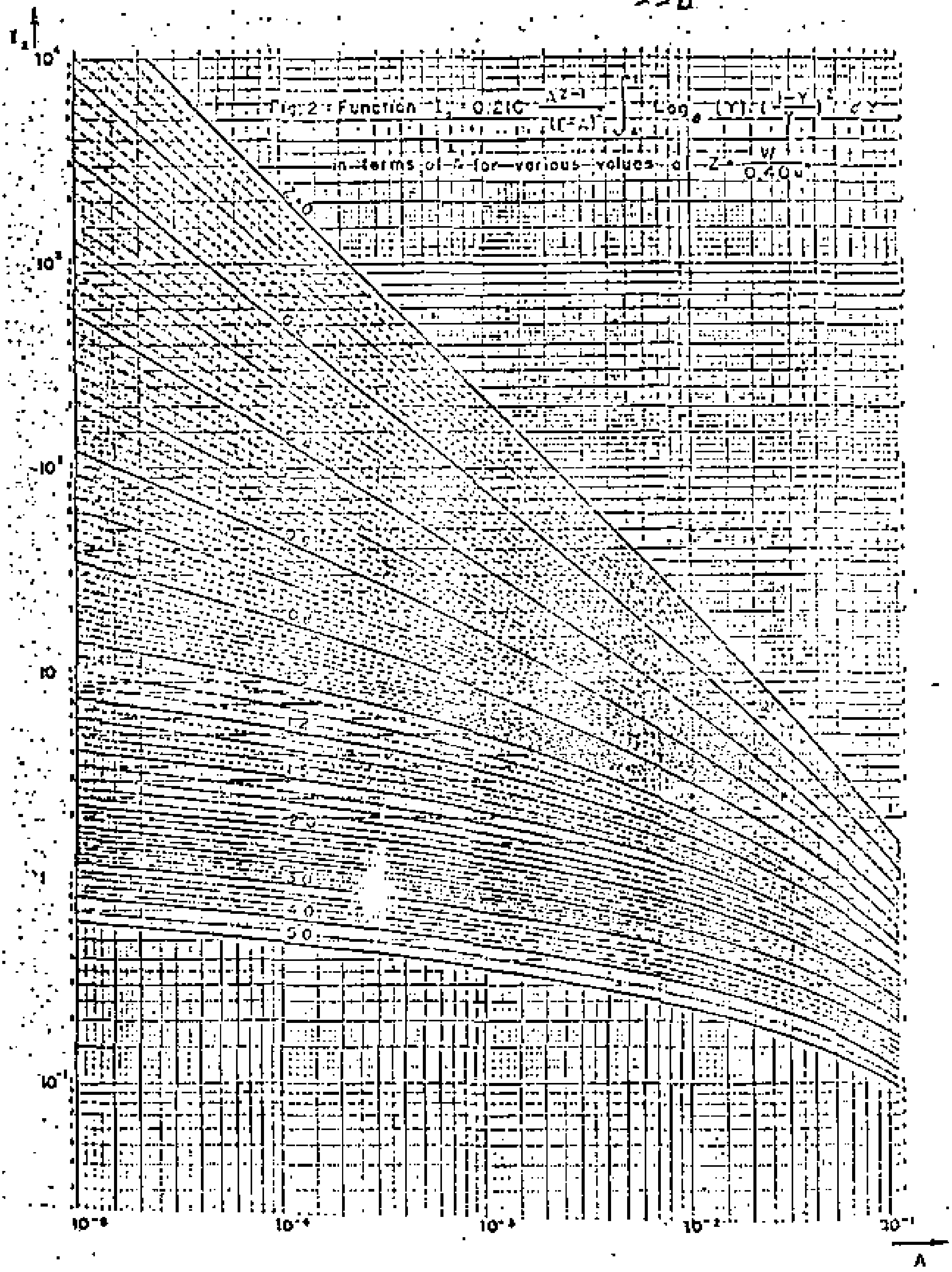


Fig 6b Valores de I_2 empleados en el método de Einstein

ζ = coeficiente determinado del coeficiente C de Chez y

u_o = magnitud de la velocidad tangencial en el fondo

v = velocidad de corriente

K = constante de Von Karman

W = velocidad de caída de las partículas

$Z = \frac{W}{Kv*}$ = exponente de distribución de concentraciones

$v^{*} = (\tau_c/\rho)^{1/2}$ = velocidad del esfuerzo cortante

A = 2D/H = Relación entre el grueso de la capa donde hay arrastre y el tirante H.

c) Formulas que predicen la corriente e incluyen las características de la playa

El grupo anterior de fórmulas supone conocer la velocidad media de la corriente litoral, cosa de difícil obtención

c.1) Eagleson

$$V^2 = 3/8\ (g\ H_b\ M_b)/d\left(\frac{(\text{sen } m \text{ sen}\alpha \text{ sen } 2\alpha)}{f}\right)$$

M_b = factor de grupo (n)

f = coeficiente de Darcy - Weisbach

$= (2 \log_{10} (d/He) + 1.74)^{-2}$

$K_E = D_{65}$ ó 0.00093 si no existe granulometría

c.2) Galvin

$V = K\ g\ m\ T \text{ sen } 2\alpha; \quad K = 1$

c.3) Brenn

$$V = c'_f \left(H_b^{3/2}\ m\ \frac{\text{sen } 2\alpha}{T}\right)^{1/2}\ ; \quad C'_f = 14.3$$

c.4) Putnam

$$V = (6.97\ g\ s/f\ (m\ H^2_b)/T \text{ sen } 2\alpha)^{1/3}$$

c.5) Inman y Quinn

$$V = ((0.25\ R^{-2} + (2.28\ g\ H_b)^{1/2} \text{ sen}\alpha)^{1/2} + 1/2\ R)^2$$

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Appendix E

Use of Tracers in Harbor, Coast, and Ocean Engineering

*Bruun, P. 1970. "Use of tracers in harbor, coastal and ocean engineering." *Engineering Geology*. Amsterdam: Elsevier Publishing Company.

Summary

This appendix describes the use of tracers in various engineering fields including ocean, harbor, coastal and river engineering and some special applications of tracers, e.g., in pollution control. Certain pertinent aspects of tracer technology and methods of analyses to determine drift pattern aspects of tracer technology and methods of analyses to determine drift pattern and quantity are mentioned. A list of references and a special bibliography are included, the latter giving examples of literature on actual tracer projects in various fields of science and technology.

Introduction

For many years methods of tracing sediments have been sought. In the past it was not unusual for scientists to investigate the mineralogic components of the sediments in order to determine the presence of a unique component or ratio of several components. If such a mineral were present, it could be utilized as a natural tracer. As an example several forms of the minerals, hornblende and augite, have been identified and utilized along the Mexican and Californian gulf coasts, respectively. Along the Gulf of Mexico coast, horblende is present in the sedimentary assemblage from both the Mississippi and the Rio Grande rivers. However, the type of hornblende derived from each of these sources is quite distinctive and can be separated readily by microscopic studies. Therefore, possibilities exist for identifying the source of the sediments for the area between the mouths of the two major river systems. In California the presence of augite in the bottom sediments at Santa Barbara indicates a source from rivers north of Point Conception, because the sediments from the rivers east of Point Conception do not contain augite (Trask, 1952). Kamel (1962) uses natural (radioactive) thorium as a tracer.

The study of natural tracer minerals has proven useful in fully deriving a concept of the processes functioning in a given area; however, the results may be misleading. It can be stated unequivocally that the tracer minerals mentioned above are not representative of the total sediments. Usually of all the heavy minerals, hornblende and augite are among these, representing from 1 to 5% of total sediments. The heavy minerals have densities greater than 2.80 grams per cubic centimeter, or greater than that of quartz (2.65 grams per cubic centimeter) and the feldspars (≥2.7 grams per cubic centimeter) which, in most cases, makes up the bulk part of the remaining 95% of the sediments. Because of the greater density, the fall velocity is greater and, therefore, the heavy mineral particles act as a larger particle of quartz or feldspar. The difference in fall velocity illustrates the cardinal rule of sediment tracing: the labeled tracer particle and the natural sediment must possess the same physical and hydraulic characteristics when exposed to wave or current forces. For this primary reason, the heavy minerals are not satisfactory tracers.

Dying of sediments had been tried in the past, but during the middle 1950s, two new methods of tracing sediment transport appeared involving labeling sediments with fluorescent or radioactive material. Both techniques involve the measurement of radiation—one in the visible spectrum and the other in the very short wave lengths. The former requires excitation by ultraviolet light energy of appropriate wave length, while the latter is self-energized, i.e., radioactive. Many different tracers are available for each technology. Reference is made to *Proceedings of the 20th International Navigation Congress,* Baltimore, 1961, Section II, Subject 5, "Methods of determining sand and silt movement along the coast, in estuaries and in maritime rivers. Use of modern techniques such as radioactive isotopes, luminophores, etc."

For the first method, a glue or resin containing a fluorescent material is painted onto the indigenous sediments, (see Newman, 1960). The sediments are injected into the environment, and samples obtained subsequently undergo laboratory analysis to determine the concentration of the labeled particles. Labeling for the latter method requires irradiation of the natural sediments if a proper purity exists (Inman and Goldberg, 1955), painting the sediments with a glue or resin containing an irradiated (excited or energized) isotope (Gilbert 1954), irradiation of a simulated (glass, plastic or concrete) sediment containing a tracer isotope (Putnam et al., 1954) or absorption of a radioactive isotope into the surface and interior of a natural sediment (Krone, 1957). Other special methods, such as forcing of radioactive gases in a carrier, exist. The sediments are injected into the environment, and the concentration of their presence is made in the field by Geiger counters or scintillators. While much has been said and written concerning the hazards of radioactive material, a well-conceived and carefully performed radioactive tracer experiment will not be detrimental to any of the living organisms using, e.g., irradiated Sc_2O_3 (common), Rb_2CO_3 Au 198, Ag 110, Co 60, Cr 51, etc. Table E.1 is a comparison between fluorescent and radioactive tracers, outlining advantages as well as disadvantages related to the practical application of the two different tracer techniques. No actual cost figures are given, mainly because the cost of radioactive tracers varies greatly with the type of tracer to be employed and the character of the specific task to be undertaken.

From Table E.1 it is noted that the main difficulty involved in radioactive tracing lies in the production and transportation of these tracers which, in turn, are responsible for relatively high costs. Often government regulations make it almost impossible to use radioactive tracers. Detection by bottom instrument and analysis is relatively easy but may involve coring. Samples have to be analyzed soon after they are secured because of the fading out of activity.

The main advantage with fluorescent tracing is that it is harmless. Furthermore, it is usually less expensive because it does not require complicated safety measures and development of tracers. Labeling materials are commercially available at reasonable prices (e.g., $0.20 to $0.30 per pound) instead of having to be acquired from radiation laboratories, and no safeguards are required in transporting materials. This method has, however, one main disadvantage—namely that at this stage of the development, it is based on sampling to be followed by laboratory analysis of dry samples inasmuch as scanning of wet samples is not possible. This is time consuming and lacks flexibility because it is difficult to change a sampling program until the results of the sampling(s) already carried out are known. The technique, however, may be improved by preliminary scanning (e.g., on the survey vessel itself), using ovens to dry the samples whether secured as cores, by clamshell or scraper or bound in parafin plates pressed towards the surface of the bottom or by entirely new approaches,

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Table E.1. Comparison Between Fluorescent and Radioactive Tracing

	Fluorescent	Radioactive
Production of tracers	may be produced any place where it is needed; no health hazard	irradiation process at special installation (atomic station); a number of safety measures needed; health requirements by government
Type of tracers	a great variety of colors; a few main colors	γ-emitters preferred; a great variety in half lifetime available
Costs	relatively low, e.g., \$0.5-\$1.00/kg.	relatively high because of cost of isotopes and all kinds of safety measures which are strictly enforced by government agencies
Transport and storage	no problems	numerous safeguards; special containers and storage requirements.
Injection	by seeding; no problems	difficult because of safety precautions
Tracing time	tracers are very durable when a resin-like ureaformaldehyde is used but may contaminate areas of injection for years; some glue tracers have limited lifetime depending upon exposure	a number of tracers exist with half lifetime for γ-ray emission ranging from a few days to half a year or more; it is usually possible to select a tracer with a half lifetime suitable for each particular purpose
Sampling or tracing	surface or core samples have to be picked up, dried and analyzed in laboratory	tracing must be done by a bottom instrument, e.g., geiger counter or scintillator, which is moved over the bottom on a sledge or similar device
Analyses	counting of particles by visual observation using ultraviolet energy-source or by a scanning machine with photoelectric cells; this may be laborious and time-consuming, therefore, a costly procedure	the reading on the geiger counter or scintillator recording instrument gives the result directly; calibration based on probings may be necessary; no further cost on analyses
Accuracy	accuracy depends upon number of samples and length of cores, and upon the accuracy of the scanning procedure itself; it is difficult for the human eye to count more than 3-4 tracers in one sample; samples must be very dry before scanning	accuracy depends upon the depth of disturbance or rearrangements of bottom material; in order to get more reliable data beyond surface readings cores may have to be taken and made subject to laboratory analyses; calibration based on probings possible under simplified conditions.
Scales	best on small scale problems	best on large scale problems

such as underwater tracing using TV cameras, or other photo techniques which, however, apply to the surface of the bottom sediments only.

Fluorescent tracings' great advantage lies in the fact that it can be put in operation with short notice and with little preparation in advance. By varying the color of the tracer label, it is possible to conduct successive tests in the same area, when conditions change quickly, without the waiting period required for radioactive tracers. Also, it is now possible to produce long-lived as well as short-lived tracers of all practical sizes needed for gravel, sand, silt and clay tracing.

Various Methods of Tracing

Tracing of Pattern

Tracing of pattern simply means that the movement of the sediment is followed by dumping and sampling in some grid system allowing a concentration versus time diagram to be drawn indicating the direction and relative distance traveled by the tracer grains. This method suffices for many practical purposes where knowledge of relative magnitudes is of primary importance. Publications on this topic are numerous. The papers published in the *Proceedings of the 20th International Navigation Congress* are typical. Examples of publications of similar nature are Hydraulic Research Station (1956); Zenkovitch, (1958); Russell (1960); Griesseier and Voigt (1965).

Use of Tracers as Velocity Indicators

This method utilizes tracers as velocity indicators only (Bruun, 1967, 1969). The quantity of migrating material is measured in traps installed on the bottom and the average velocity of the migrating grains is measured over a certain time period, which covers the time until maximum concentrations have been passed. The total amount of material is then concentrated in an "imaginary" surface layer with thickness corresponding to the average velocities found. This method can only be operated from permanent sampling stations using special sampling equipment. Using various colors of tracers dumped at different distances and distributing the layer thickness in the ratio between concentrations, one gets a clear picture of the multimotions which take place. This method is in principle similar to the method of "spatial integration" mentioned later.

Quantitative Tracing

Methods have been proposed to arrive at quantitative interpretation of tracer-measurements. The simplest method is the *steady dilution method* which utilizes a constant tracer supply over and assumes that an equilibrium concentration is obtained downstream of the injection point. The technical drawback of this method is that the supply must be given for a long time. This can be overcome by time integration (see Crickmore and Lean, 1962), but both methods are very time consuming.

The *spatial integration method* is based on separate evaluation of the mean particle speed and the depth of movement. The disadvantage of this method lies in the small concentration which may occur resulting in questionable accuracy. Determining the depth of travel may require meticulous surveys and/or corings. Reference is made to Lean and Crickmore (1962) and Hubbell and Sayre (1964).

The most practical and, undoubtedly, most versatile methods are those which apply theoretical dispersion models by which a connection is found between the transport phenomenon and the development of concentration distribution. This mathematical modeling has given promising results. For details the reader is referred to the comprehensive thesis by De Vries (1966).

Practical Use of Tracers

Tracers, radioactive as well as fluorescent, have now been in use for more than 10 years, and usage is still being expanded by improvements in already existing fields as well as new applications. Utilization now covers a wide range of fields, in-

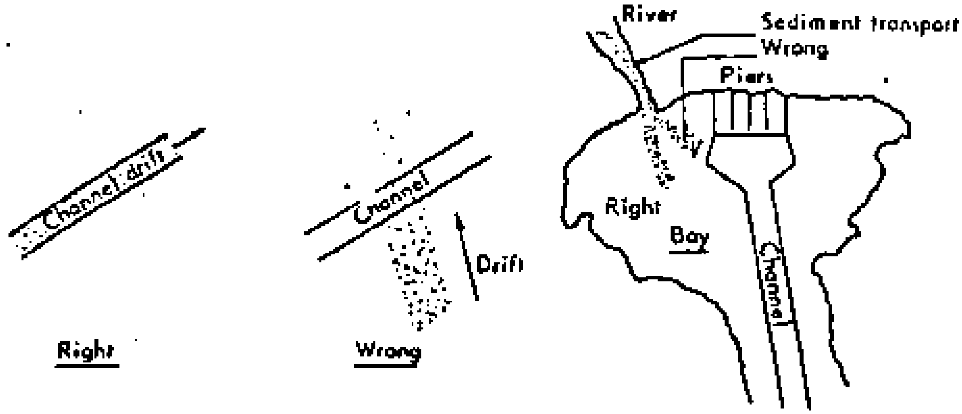

Figure E.1. Tracing pattern of drift at navigation channel; planning navigation channel; investigation of sedimentation at navigation channel.

cluding harbor, ocean, coastal and river engineering, littoral drift and dredging technology, pollution control, etc.

A number of examples of tracer applications in experiments already carried out or suggested applications are given in Figures E.1 to E.19. In the examples of projects already carried out, R indicates radio-active tracing and F, florescent tracing. Because some of the examples given are only described in commercial reports, the list of references includes a special section which gives a number of examples on practical tracing projects.

Tracing of Pattern(s) and Direction(s) of Sediment Transport

This application is of great importance for determining the location of a navigation channel, harbor entrance, pier or dock. Application of tracing technique may result in better planning of future maintenance of navigation channels.

Example: F-tracing on longshore drift in the Black Sea (USSR); R-tracing of bottom drift at Norfolk (England); F-tracing of longshore drift at several places in California and Florida. Special tracing of sediment transport pattern at inlets was carried out by R-tracing on the North Sea coast in Denmark at Thyboroen, by R-tracing in Portugal, e.g., at Figueira da Foz, by R-tracing in India of silt transport (at Bombay), by R-tracing in San Francisco Bay and by F-tracing at several inlets in Florida, e.g., at Palm Beach Inlet, South Lake Worth Inlet and Hillsborough Inlet. F-tracing at Kingston, Jamaica, was undertaken to determine which rivers discharge sediment in navigation channels and harbor basins.

Figure E.1 explains that a navigation channel, if possible, should not be built crosswise on a sediment transport lane (unless it is protected). It is also dangerous to build a navigation channel too close to rivers which discharge much sediment.

Figure E.2 shows how tracers may be used to determine the location of a port on a littoral drift shore the minimum costs to maintenance dredging.

Figure E.3 explains how jetties or breakwaters for a harbor may be built to protect against a known littoral drift pattern and how a confluence of sediment carrying rivers may be arranged to avoid deposition of sediments.

Evaluation of the Relative Magnitude of Drift in Two Opposite Directions

When a harbor on a littoral drift shore, a dredging operation or a coastal protection shall be planned, engineers are often faced with the prob-

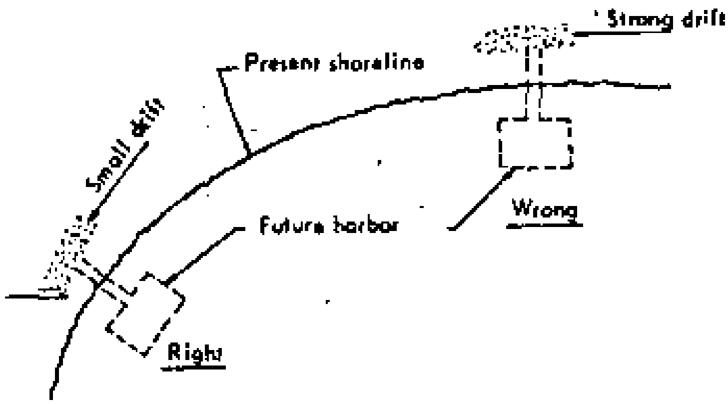

Figure E.2. Tracing of relative magnitude of drift on an open shore; planning location of port on littoral drift coast.

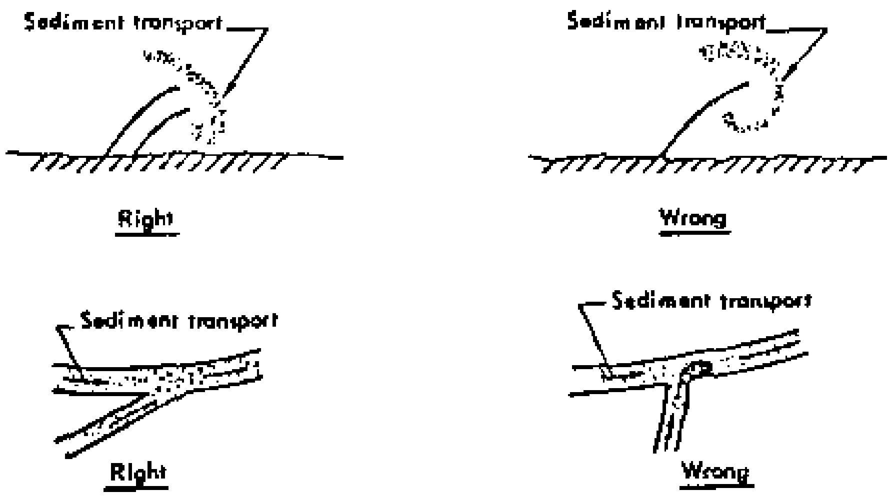

Figure E.3. Tracing pattern of drift at navigation and other channels.

lem of determining the direction of "predominant drift" as well as the relative magnitude of drift in two directions. This may be done by hindcasting of wave action and by computation of longshore wave energy based on meteorological data if wave action is the only responsible factor for material transport. Tidal and other currents, however, often play an important role, and even if the direction of predominant drift may be determined by observation of accumulation at structures extending into the sea, it is still very difficult to evaluate the relative magnitude of drift in two directions. Knowledge hereof is of great importance with respect to determining the location and extension of groin protection, the direction and relative length and shapes of jetties for protection of an inlet entrance, the location of sand traps for a navigation channel, planning of by-passing operations for

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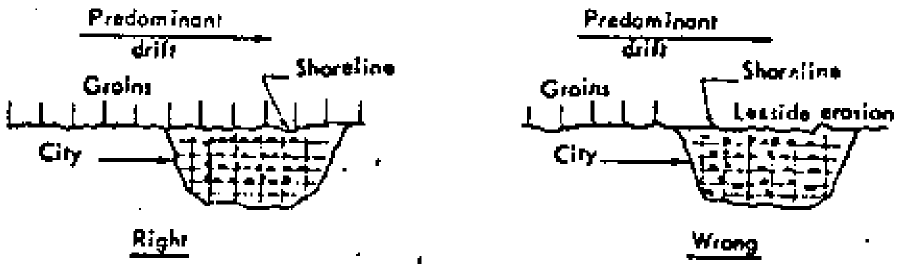

Figure E.4. How far shall a group of groins extend along a shore?

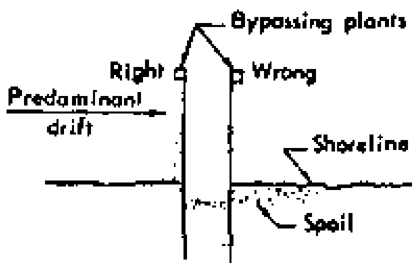

Figure E.5. Which breakwater should have the greatest length?

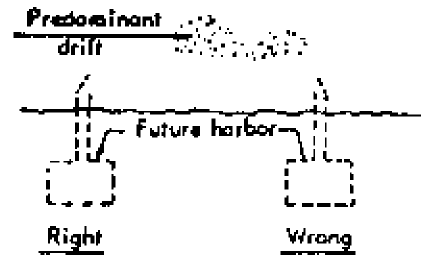

Figure E.6. On which side of an inlet should the by-passing sand plant be installed?

beach nourishment and planning of artificial nourishment and other field operations.

Examples. F-tracing on the Florida Atlantic in the Palm Beach and Broward Counties. Figure E.4 shows how the extension of a groin protection may be planned by tracer tests, Figure E.5 how the configuration of an inlet entrance may be determined (by tracing and by hydraulic model experiments) and Figure E.6 and E.7 how tracing may be helpful in planning a by-passing or trap arrangement.

Determination of the Depth up to Which Sediment Transport Takes Place

This is important with respect to evaluating the depth up to which a navigation channel will be disturbed by sediment transport, evaluating possibilities for erosion and of the amount of over-dredging which should be undertaken in critical areas, evaluating the possibilities for movement of material dumped offshore, evaluating the possibilities for deposit on the beach of material dumped offshore for beach nourishment or for bottom stabilizing purposes, as at Long Branch, New Jersey, evaluating origin of offshore deposits of sediments and minerals, etc.

Examples. F-tracing at Jupiter Island, Florida, to determine the success of offshore scraping for

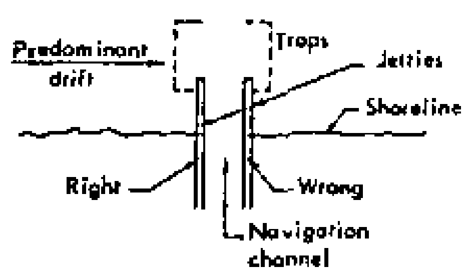

Figure E.7. Shall sand traps be placed symmetrically or asymmetrically at an entrance?

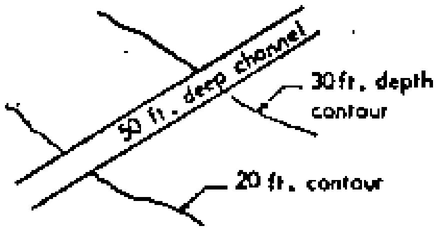

Figure E.8. Is overdepth and maintenance required beyond the 20-foot contour?

beach nourishment, F-tracing at Duluth, Lake Superior, for bottom stabilizing purposes, R-tracing at Japanese ports to determine the limiting depth of material movement, R-tracing at Cape Fear River, North Carolina, to determine source of shoal materials. Figure E.8 explains how tracers may be helpful in planning a navigation channel and the possible extent of maintenance dredging, Figure E.9 how nourishment of beaches from offshore sources may be planned. Figure E.10 refers to drift by currents at deeper waters where some minerals (placers) may tend to concentrate in certain areas carried by current concentrations.

Evaluation of the Possibilities of Effectiveness of a Dredging Operation

This and the performance of a dredging operation, including quantities of redredged material, the efficiency of agitation dredging, the deposit of material from overflow at a hopper dredge, the possibility of sediment deposits by sand, gravel or shell dredges should be evaluated.

Examples. Figure E.11 demonstrates schematically R-tracing in the Thames Estuary when some material dumped by flood current rapidly was carried back to the area when it was dredged. Such situation may be avoided by application of tracers in the spoil area before spoiling starts (Figure E.12). On the other hand natural filling of a dredged area (e.g., a pipeline trench may be desired and the possibility for that may be looked into by tracing as shown in Figure E.13. Figure E.14 shows how dredging in a certain area may

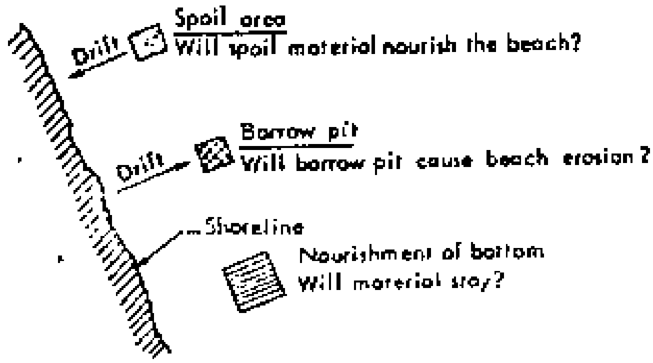

Figure E.9. Beach or bottom nourishment problems.

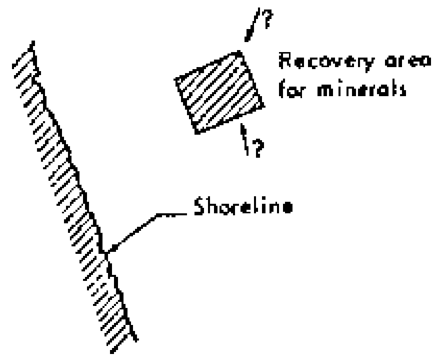

Figure E.10. Where did minerals come from and where are they going?

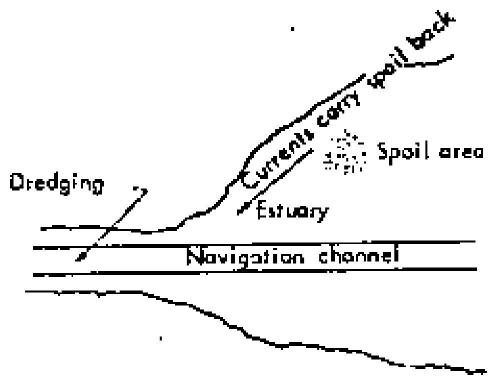

Figure E.11. Will spoil from dredging of navigation channel return to the channel?

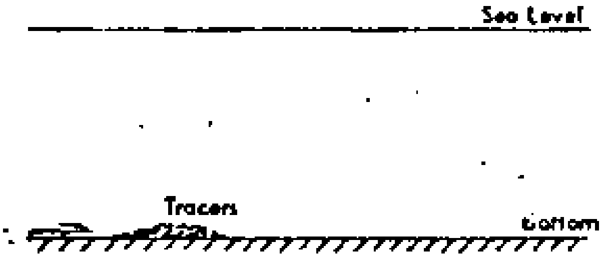

Figure E.12. Are currents able to move material on the bottom? Which way does it go?

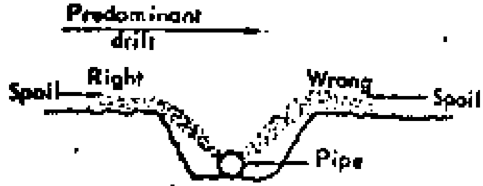

Figure E.13. Will a trench dredged for a pipeline fill in again? Where should spoil be placed?

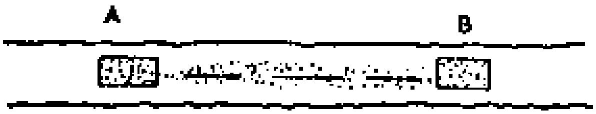

Figure E.14. Will material dredged at *A* by agitation or other dredging operations deposit at *B*?

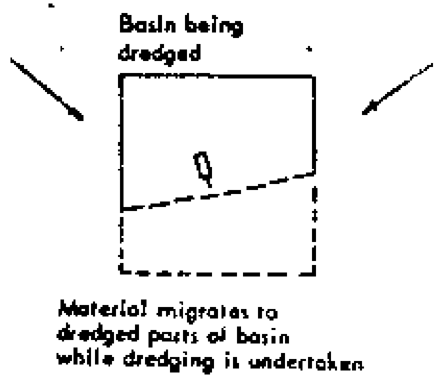

Figure E.15. Problem of siltation during dredging operation.

adversely affect dredging in another area. If dredging of these two areas is done on the same dredging contract, the contractor may face a loss because he redredges too much material. This has particular interest for agitation dredging where possibilities for deposits should be evaluated before dredging starts.

Figure E.15 shows how material from distant areas may be carried down in an area to be dredged. The dredging contractor may face a considerable loss if he must keep dredging material which invades his dredging area from elsewhere.

Figure E.16 demonstrates a sidecasting operation. Sidecasting, as undertaken at the Lake Maracaibo Bar, the Orinoco River Bar, at the Port Everglades (Ft. Lauderdale, Florida), the Oregon Inlet, North Carolina on a minor scale, as a forerunner for hopper dredging, and as undertaken at Gulf Coast inlets by the U.S. Army Corps of Engineers new multipurpose dredge *McFarland*, has proven to be very useful if only a minor part of the material migrates back to the dredged channel. The possibility exists that the material dumped by sidecasting may form submerged "jetties" which decrease material transfer to the channel at the same time as it may be helpful in concentrating tidal flow through the dredged channel, thereby improving flushing of material.

Figure E.17 shows overflow from a hopper dredge, and Figure E.18 overflows from a sand and gravel or shell dredge which may cause deposits in surrounding areas of the bottom, and perhaps adversely affect fish and wildlife. Tracers may help engineering planners and dredge contractors to demonstrate the possible extent of the incident sediment load.

Checking of the Most Effective Location of a Sand Drift Fence

A sand drift fence should be located where it causes maximum accumulation. Tracers may be helpful in determining the best location as well as

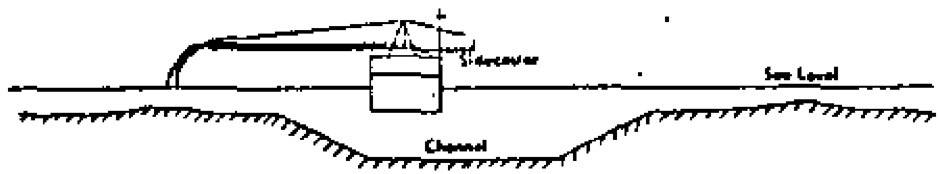

Figure E.16. Sidecasting. Will material return to channel?

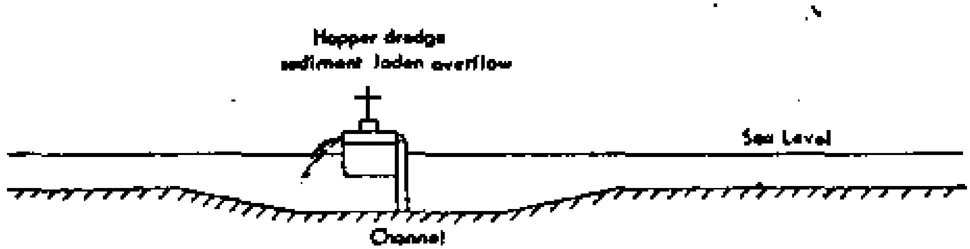

Figure E.17. Overflow of silt-laden water from hopper dredge.

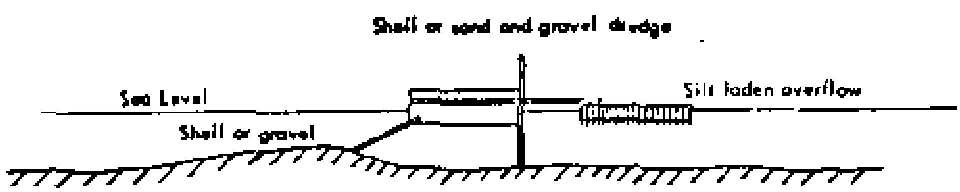

Figure E.18. Overflow of silt-laden water from sand and gravel dredge or shell dredge.

in determining the most effective area on the beach for protection of drift sand.

Example. F-tracing on the Outer Banks (Cape Hatteras National Seashore, National Park Service, U.S. Dept. of the Interior). Figure E.19 explains the application.

Application of Tracers in Pollution Control: Thermal, Biological, Chemical and Nuclear

Tracers in liquid form which could be fluorescent dyes like Fluorescin, Uranium (green) or Rhodamine B may be used for determining the best location of intakes and outlets for cooling water systems and determining the pattern and magnitude of pollution caused by outfalls containing sewage and/or chemical impurities. This problem is becoming increasingly more important because of the still increasing concentration of population and the vast increase in industrial, chemical and nuclear wastes. Examples of this are so numerous that no particular case should be mentioned specifically. The reader is referred to the special list of literature references.

It is unlikely that all applications have been mentioned. When a new technique is introduced, it takes some time before it penetrates all branches of a particular field. So much may be said that there is still much to learn. Radioactive tracers must be made easier to handle and have undoubtedly expanded future application, particularly as tracers in offshore waters and estuaries for pollution control. Fluorescent tracers must be made more flexible. Special and stable dyes with narrow emission bands must be introduced and correlated with the proper resins, glues and hardeners. Some coating should be hard, thereby securing a long

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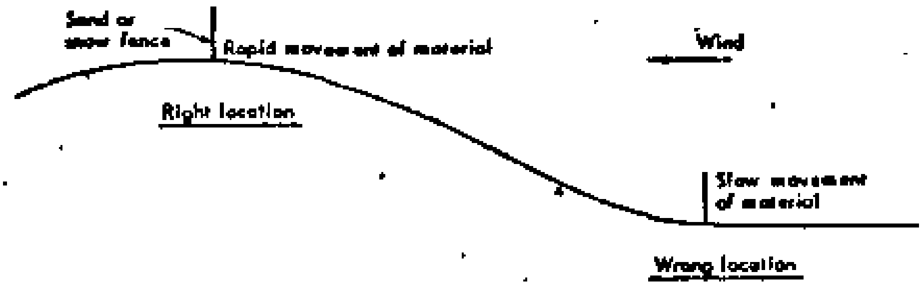

Figure E.19. Where should a sand fence be located to assure highest efficiency? Where does the material come from?

lifetime. Others must be soft so that they wear off in days or weeks, thereby avoiding contamination of lasting character.

Various kinds of sampling equipment exist ranging from paraffin plates (used by the University of Southern California, Los Angeles) to scrapers and cores (used in Florida). As mentioned below, a combination of sampling, recording and analysis technique is desirable.

The method of analysis is probably the technique which, more than anything else, calls for improvement. The human eye is, from an optical point of view, not very satisfactory. Most people are able to count two to three different colors by visual counting. But it gets difficult with four, particularly if red/yellow/orange or blue/green are involved. And it takes much time to count visually. Photoelectric scanning machines developed are helpful but several difficulties have to be overcome including difficulties in correlating exitations and dye-emmision and the corresponding need for filters which often "get tired" and change character after a while. Although much progress has been recorded, we are still in the debugging stage with respect to scanning and the ultimate solution may not be solely optical insofar as direct counting is concerned. Solution and then optical scanning may be a "solution." Much has been tested and much remains to be tested. The development of underwater TV may be very useful for field tracing, particularly if the problem of underwater exitation of fluorescent tracers is solved.

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VARIACION A LARGO PLAZO DE NIVEL DE MAR

Consideremos un plano imaginario, definido por un océano ideal cuya superficie no varía, al cual llamaremos nivel de mar.

Este nivel, en un océano real sufrirá una serie de modificaciones que por su duración clasificaremos en variaciones a corto y largo plazo.

Esta clasificación obedece al agente generador, en el cuadro uno aparecen los tipos de ondas y sus causas.

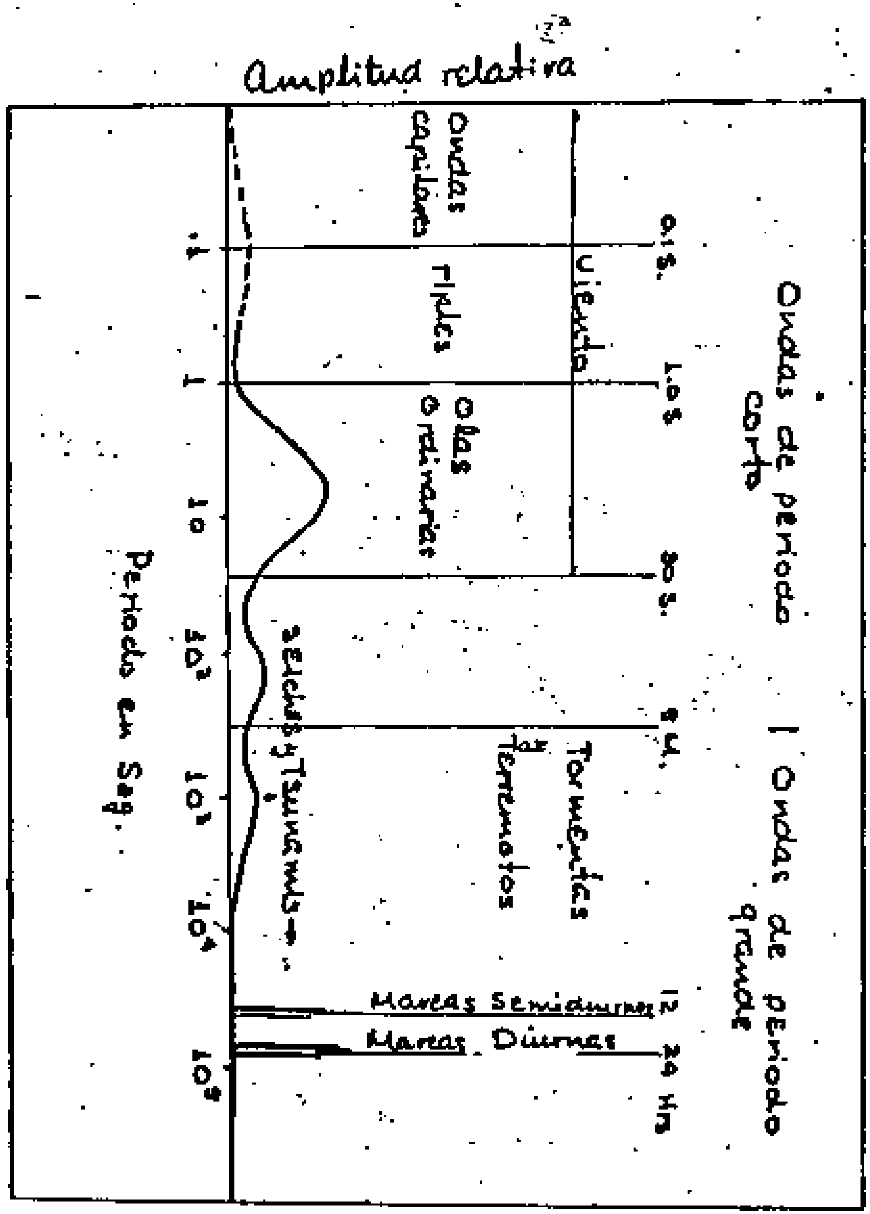
La localización del nivel de mar como plano de referencia requiere del conocimiento de una serie continua de mediciones de la altura del agua, para poder ir filtrando las modificaciones que produzcan ciertos agentes y dejar las modificaciones más constantes y periódicas y sobre éstas estandarizar.

CAUSAS DE VARIACION DEL NIVEL DE MAR

Descartando las variaciones producidas por el viento, fuerza gravitacional, tormentas y terremotos, los factores que intervienen en el cambio estacional o extraordinario del nivel serían:

a) Factores oceanográficos, como el cambio de densidad de la columna de agua a lo largo del año, debidos a diferencias de salinidad y temperatura.

Para filtrar estos cambios de nivel se puede computar un nivel "estérico" y considerar que cualquier cambio de densidad en la columna de agua sin cambiar de masa



esta regido por:

$$z = \frac{1}{g} \int_{p_a}^{p_o} \Delta\alpha \, dp$$

donde z es el nivel de referencia o estérico, en cualquier lugar de la columna. P_o la presión en este nivel, P_a la presión atmosférica y $\Delta\alpha$ la anomalía en el volumen específico.

Para hacer estos cálculos se requiere diseñar un nivel estérico inicial y su P_o, mediante la media de los valores obtenidos anteriormente y de ahí calcular $\Delta\alpha$

b) Factores climatológicos. De este grupo el más importante es la presión barométrica y su efecto se puede considerar que actúa en relación lineal en la superficie del océano.

$$S = a \, (P_a)$$

Si conocemos los valores medios de la superficie $\overline{S}$ y de la presión $\overline{P_a}$ se puede ajustar un coeficiente a y para un caso dado tendremos:

$$(S - \overline{S}) = a \, (P_a - \overline{P_a})$$

c) Cambios eustáticos - Debidos a aumento en el volumen de agua del océano por deglaciación. Estos cambios son notorios, en cuanto a un incremento de nivel, en las zonas en que la disminución de peso de hielo no provoca reacomodo de la corteza terrestre.

d) Cambios de la corteza. Si los continentes se encuentran en movimiento provocaran un cambio de volumen en las cuencas oceánicas adyacentes y en altura relativa desde el centro terrestre.

e) Cambios de las superficies geopotenciales. La asociación del efecto de Coriolis, densidad, localización geográfica y régimen climatológico, produce una superficie geopotencial. Esta superficie puede cambiar de un lugar a otro y por lo tanto se produce una diferencia de niveles de un lugar a otro, por ejemplo el nivel del mar es 0.5 m. más alto en el mar Báltico que en el Mediterráneo.

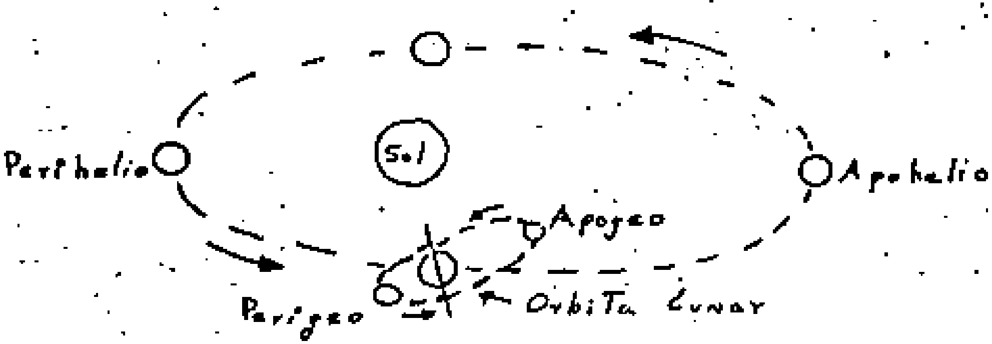
MAREAS

Las masas de agua oceánica responden de una manera compleja a las fuerzas generadoras de mareas de la tierra y los cuerpos celestes.

Los efectos de la topografía costera y submarina, la resonancia en bahías y estuarios cambian las características de la marea. Por esto las mareas debén calcularse con los datos que provengan del propio lugar, mediante un análisis de las armónicas y filtrando las diferencias que provengan de otras causas.

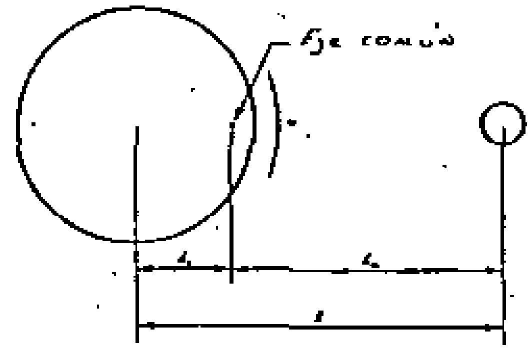
<u>Teoría de Equilibrio de las mareas:</u>

Las variaciones de marea significantes se pueden considerar como producto de la posición y movimiento del sistema sol-tierra-luna.



Darwin presentó una teoría de equilibrio de mareas considerando un solo cuerpo generador actuando sobre una lámina de agua en un cuerpo esférico que no gira y analizó la forma de equilibrio resultante.

El sistema tierra-luna estaría girando alrededor de un eje común.



$$M_T\omega^2\ell_T = \frac{GM_TM_m}{\ell^2}$$

Balance de fuerzas terrestre

$$\frac{GM_TM_m}{\ell^2} = M_m\omega^2\ell_m$$

Balance de fuerza lunar

$$M_T\omega^2\ell_T = G\frac{M_TM_m}{\ell^2} \quad (1)$$

$$M_m\omega^2\ell_m = G\frac{M_TM_m}{\ell^2} \quad (2)$$

la distancia ℓ sería

$$\ell = \ell_T + \ell_m \quad (3)$$

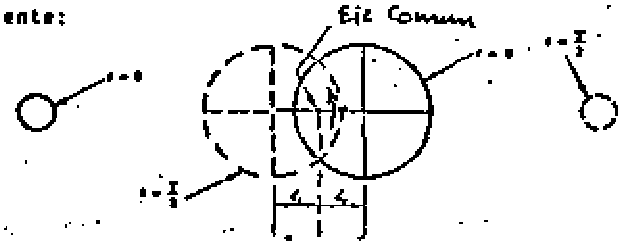
De estas 3 ecuaciones podemos encontrar la distancia ℓ_T a la que se encuentra el eje común del centro de la tierra y el período T del sistema.

$$\ell_T = \frac{\ell}{1 + \frac{M_T}{M_m}} \quad (4)$$

$$T = \frac{2\pi}{\omega} = 2\pi\sqrt{\frac{\ell^3}{GM_m\left(1 + \frac{M_T}{M_m}\right)}} \quad (5)$$

Resolviendo estas ecuaciones se encontró que $l_e = 4{,}666$ Km. o sea menor que el radio terrestre y el período es de 27.3 días.

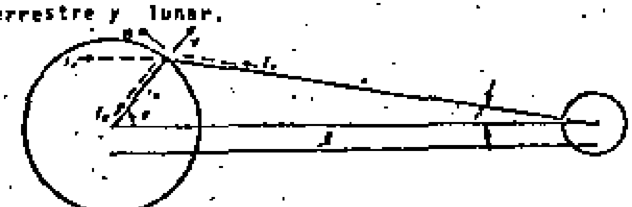
Si este sistema está girando ofrecería un esquema como el siguiente:



A).- La fuerza centrífuga de una unidad de masa de agua dependería de:

$$f_c = \omega^2 l_e$$

y actuaría opuesta a la luna y paralelamente a la línea que uniera los centros terrestre y lunar.



usando (1) la ecuación de fuerza centrífuga será:

$$f_c = \frac{G M_m}{l^2} \qquad (6)$$

y puesto que la fuerza atractiva de la tierra es:

$$g = \frac{G M_e}{r_e^2} \qquad G = g\,\frac{r_e^2}{M_e}$$

la ecuación (6) quedaría

$$f_c = g\left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{l}\right)^2$$

B).- La fuerza atractiva de la luna f_a sería siempre hacia el centro lunar.

$$f_a = \frac{G M_m}{s^2} = g\left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{s}\right)^2$$

S = a la distancia de una partícula de agua hasta el centro lunar.

C).- La fuerza atractiva de la tierra será simplemente:

$$f_g = g$$

Si resolvemos para la última figura en los componentes radial y tangencial tenemos:

$$f_r = -f_g - f_c\cos\theta + f_a\cos(\theta+\beta)$$

$$= -g + g\left(\frac{M_m}{M_e}\right) r_e^2\left[\frac{\cos(\theta+\beta)}{s^2} - \frac{\cos\theta}{l^2}\right]$$

$$f_\theta = f_c\,\mathrm{sen}\,\theta - f_a\,\mathrm{sen}(\theta+\beta)$$

$$= g\left(\frac{M_m}{M_e}\right) r_e^2\left[\frac{\mathrm{sen}\,\theta}{l^2} - \frac{\mathrm{sen}(\theta+\beta)}{s^2}\right]$$

Aproximando y quitando los términos que no tengan un valor representativo.

$$F_r = -g$$

$$F_\theta = -3/2\; g \left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{\lambda}\right)^3 \operatorname{sen} 2\theta$$

Superficie de equilibrio:

La superficie resultante η para cualquier punto estaría dada por:

$$\frac{d\eta}{dx} = \frac{d\eta}{r_e\, d\theta} = -\frac{F_\theta}{F_r} = -\frac{3}{2}\left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{\lambda}\right)^3 \operatorname{sen} 2\theta$$

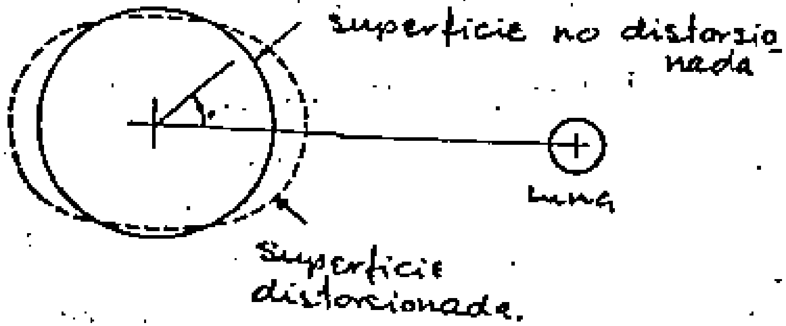
integrando

$$\eta = \frac{r_e}{4}\left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{\lambda}\right)^3 (3 \cos 2\theta + C)$$

C es la constante de integración que determina la cantidad de agua necesaria para encontrar la superficie de equilibrio, como debe de ser la misma cantidad antes y después de la deformación $C = 1$

La ecuación representa un esferoide con su eje mayor hacia la luna.

El valor máximo de η ocurre para
$\theta = 0°$ y $180°$ $\eta = 0.5273$ m.
y el mínimo para $\theta = 90°$ $\eta = -0.1786$ m.



Si la luna gira sobre el plano ecuatorial de la tierra y considerando un punto de observación a una latitud, θ, debemos proyectar el punto de observación al plano ecuatorial.

$$x' = r_e \cos\phi \cos\lambda \qquad x' = \text{proyección}$$

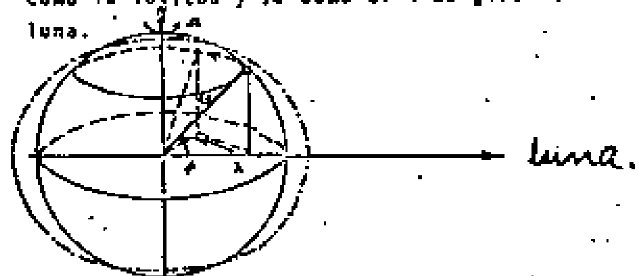
$$\cos\theta = \frac{x'}{r_e} = \cos\phi \cos\lambda$$

$$\cos 2\theta = 2\cos^2\theta - 1$$

$$\eta = \frac{r_e}{2}\left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{\lambda}\right)^3 (3\cos^2\phi \cos^2\lambda - 1)$$

η define cualquier ϕ y λ

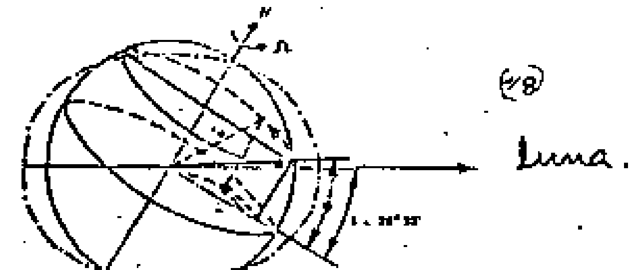
ϕ como la latitud y λ como el ∢ de giro de la luna.



Si la luna gira en un plano diferente al ecuatorial.

$$\eta = \frac{r_e}{2}\left(\frac{M_m}{M_e}\right)\left(\frac{r_e}{\lambda}\right)^3 \left[(3\operatorname{sen}^2\phi \operatorname{sen}^2\delta - 1) + 3/2 \operatorname{sen}2\phi \operatorname{sen}2\delta \cos\lambda + 3\cos^2\phi \cos^2\delta \cos^2\lambda \right]$$

donde δ es la declinación lunar en el plano ecuatorial, siendo la máxima 28°30'.



Las mareas se desplazan en las cuencas oceánicas con una - velocidad dependiente de la profundidad.

Si T_n es el "Período Natural" de una cuenca considerado como el tiempo necesario para recorrer 2 veces la cuenca a una celeridad $\sqrt{g\bar{h}}$

tendremos para el Océano Atlántico

largo $S = 2.2 \times 10^7$ ft
prof. media $\bar{h} = 12{,}900$ ft

$$T_n = \frac{2S}{\sqrt{g\bar{h}}} = 68.3 \times 10^3 \text{ seg} = 19 \text{ horas}$$

Para el Pacífico

largo $S = 5.1 \times 10^7$ ft
prof. media $\bar{h} = 14{,}040$ ft

$\therefore T_n = 42$ horas

Las distancias y profundidades son ecuatoriales.

Elevaciones y Corrientes de Marea en un canal.

El efecto de rotación provoca que una onda progresiva en un canal genere corrientes transversales aparte de las elevaciones y corrientes longitudinales.

Para un canal de geometría regular y aplicando las ecuaciones de continuidad, movimiento, considerando Coriolis y condiciones frontera.

Las elevaciones y las corrientes laterales y longitudinales se regirán por:

$$\left.\begin{aligned} \eta &= Z \cos(\sigma t - Kx) \\ u &= U \cos(\sigma t - Kx) \\ v &= V \sin(\sigma t - Kx) \end{aligned}\right\} \quad (1)$$

siendo Z, U y V funciones de y la frecuencia rotacional σ se considera la de la marea y K se determina de:

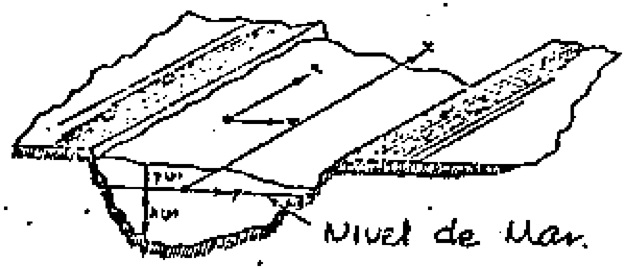
(2) $\frac{\sigma}{K} = \frac{L}{T} = C = \sqrt{g\bar{h}}$ como aproximación

$\bar{h}$ = prof. media

esta aproximación funciona si la relación

$\sigma^2 b^2 / g h_m$ es pequeña.

b es el semi-ancho del canal y h_o es la profundidad de referencia.



Corrientes:

Si de las ecuaciones ③ suponemos que:

$$v = u/2 \qquad \text{y siendo} \qquad x = 0$$

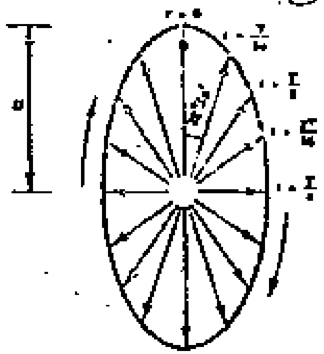
las corrientes transversales y longitudinales

$$\left.\begin{aligned} \frac{u}{U} &= \cos \sigma \tau \\ \frac{v}{U} &= \tfrac{1}{2} \operatorname{sen} \sigma \tau \end{aligned}\right\} ③$$

simplificando

$$\left(\frac{u}{U}\right)^2 + \left(\frac{v}{\tfrac{1}{2} U}\right)^2 = 1 \qquad ⑦$$

esta ecuación representa una elipse.



Esto significa que las corrientes de marea varían con el tiempo de dirección y magnitud.

Asimismo varía la excentricidad elíptica con la profundidad a tiempos iguales en un punto.

Efecto de la Configuración terrestre.

Si se están generando alturas debido a la fuerza atraccional de los cuerpos celestes deben generarse alturas iguales al mismo tiempo. A la vez se puede pensar que habrá lugares donde se presente la máxima altura en tiempos distintos.

Las líneas que unen las líneas de igual altura son líneas de co-rango.

Las de igual tiempo son líneas cotidales.

MAREA DE TORMENTA

Los problemas de elevación de nivel por efecto de tormentas se pueden dividir en varios casos y considerarlos como sigue:

A).- Lagos cerrados y presas.
1. Canal rectangular y de profundidad constante.
2. De forma regular.
3. De forma semiregular.
4. De forma muy irregular.

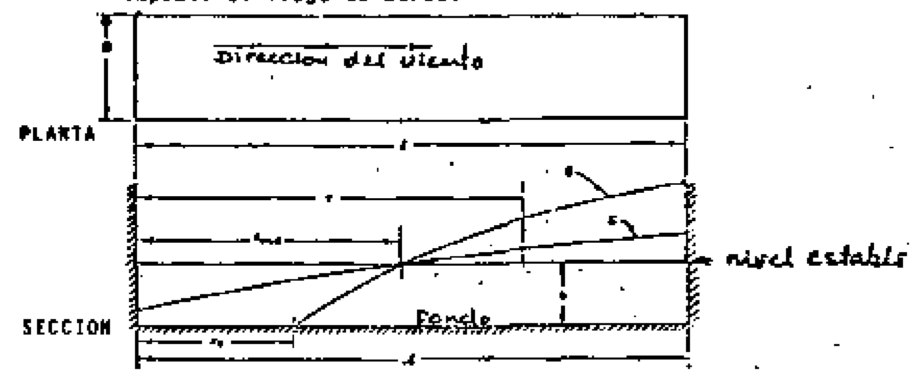
B).- Fuera de la costa o en la Plataforma Continental.
1. Fondo de profundidad constante.
2. Fondo de pendiente constante.
3. Perfil de fondo ligeramente irregular.
4. Perfil de fondo irregular.

C).- En la línea de costa.
1. Línea de costa regular.
2. Línea de costa algo irregular.
3. Línea de costa con salientes.

D).-Entre relieves de la topografía costera.
1. Barreras naturales bajas
2. Barreras naturales medias
3. Barreras naturales altas.

E).- Bahías y Estuarios Abiertos
1. Entrada seguida de un cuerpo de agua grande y con movimiento libre del agua.
2. Entrada seguida de un cuerpo de agua pequeño y con movimiento libre del agua.

3. Entrada pequeña o suficientemente obstruida para impedir el flujo de marea.



A).- Lagos cerrados y Presas.
1. Canal rectangular y de profundidad constante.

Para viento de velocidad constante y paralelo al eje del canal Hellstrom, Langhaar Yenligan, proponen ecuaciones como las siguientes:

(1) $$\frac{ds}{dx} = \frac{(\tau_s + \tau_b)}{\rho g (h+s)}$$ (angulo de la sup. del agua)

(2) $$\int_0^{\ell} (h+s)\, dx = \ell h$$ (conservación de volumen, fondo no expuesto).

(3) $$\int_{x_0}^{\ell} (h+s)\, dx = \ell h$$ (fondo expuesto)

(57)

donde S es el aumento de nivel por viento x la distancia horizontal, τ_s esfuerzo del viento, τ_f esfuerzo en el fondo, h la profundidad del agua, ℓ largo del canal, x_o el largo de la superficie expuesta.

En forma empírica se ha calculado para:

$$\tau_s / \rho g = 3.0 \times 10^{-6} \, u^2 / g$$

y que

$$\tau_f / \tau_s \approx 0.1$$

por lo que la ecuación ② quedaría

$$\frac{dS}{dx} = \frac{K u^2}{g(h+S)} \qquad ③$$

donde $K = 3.3 \times 10^{-6}$ y u la velocidad del viento.

Para cuando el fondo no se expone $x_o = 0$ la ecuación 2a se usa para encontrar la constante de integración para la ecuación 1.

Para el fondo expuesto la constante será x_o mientras que el aumento de nivel entre $x = 0$ y x_o x será $S = -h$

El punto nodal $(x = x_{nodal}, S = 0)$ se regirá por:

$$\frac{x_{nodal}}{\ell} = 1 - \frac{S_{max}^2 + 2 S_{max} h}{2 K u^2 \ell} \qquad ④$$

cuando $S = S_{max}$ en $x = \ell$

(55)

TABLA PARA FONDO NO EXPUESTO

Values of S/h corresponding to x/ℓ and $\kappa U^2 \ell / g h^2$

$\kappa U^2 \ell / g h^2$	x_{nod}/ℓ	x/ℓ = 0	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
.301	.495	-.101	-.082	-.070	-.039	-.018	.003	.021	.011	.050	.079	.092
.309	.493	-.100	-.086	-.063	-.031	-.019	.002	.022	.043	.043	.082	.101
.318	.491	-.114	-.089	-.066	-.013	-.020	.002	.024	.045	.055	.086	.105
.328	.491	-.119	-.091	-.068	-.015	-.021	.002	.025	.047	.059	.089	.115
.339	.490	-.125	-.098	-.073	-.047	-.022	.002	.026	.049	.072	.094	.125
.351	.489	-.132	-.103	-.076	-.049	-.023	.003	.027	.052	.075	.098	.121
-.365	.486	-.139	-.109	-.053	-.051	-.024	.003	.029	.054	.079	.104	.137
.380	.428	-.145	-.115	-.084	-.054	-.025	.003	.031	.058	.081	.109	.144
.396	.430	-.152	-.122	-.089	-.057	-.026	.001	.033	.061	.079	.116	.152
.415	.487	-.167	-.130	-.095	-.061	-.028	.004	.035	.065	.051	.123	.162
.437	.486	-.180	-.140	-.101	-.063	-.029	.005	.038	.070	.101	.131	.172
.461	.485	-.191	-.150	-.108	-.070	-.031	.005	.041	.073	.109	.140	.174
.490	.484	-.211	-.165	-.119	-.074	-.033	.006	.041	.081	.132	.154	.194
.423	.482	-.230	-.177	-.127	-.080	-.035	.008	.047	.088	.126	.164	.157
.463	.480	-.253	-.193	-.140	-.087	-.038	.009	.051	.097	.135	.178	.217
.511	.478	-.286	-.217	-.151	-.095	-.041	.011	.060	.108	.151	.196	.239
.571	.476	-.321	-.245	-.172	-.106	-.044	.014	.069	.121	.173	.219	.256
.648	.473	-.377	-.280	-.195	-.118	-.048	.016	.080	.138	.194	.247	.291
.750	.467	-.415	-.339	-.236	-.134	-.053	.024	.095	.162	.221	.284	.341
.894	.464	-.507	-.408	-.274	-.140	-.059	.032	.115	.193	.263	.334	.374
.930	.458	-.611	-.431	-.278	-.153	-.058	.039	.125	.205	.290	.338	.419
.971	.455	-.659	-.443	-.290	-.161	-.055	.033	.133	.215	.292	.365	.435
1.015	.453	-.715	-.467	-.302	-.169	-.053	.037	.140	.226	.306	.382	.453
1.066	.449	-.791	-.494	-.335	-.174	-.054	.043	.158	.239	.322	.401	.475
1.125	.444	-1.000	-.538	-.379	-.178	-.051	.061	.162	.255	.347	.429	.500

PARA FONDO EXPUESTO

Values of S/h corresponding to x/ℓ and $\kappa U^2 \ell / g h^2$

$\kappa U^2 \ell / g h^2$	x_o/ℓ	x_{nod}/ℓ	0	.1	.2	.3	.4	.5	.6	.7	.8	.9	1.0
1.125	0	.444	-1.0	-.526	-.379	-.178	-.052	.061	.162	.255	.312	.423	.399
1.20	.021	.438	-1.0	-.565	-.345	-.182	-.017	.079	.179	.276	.367	.432	.511
1.40	.070	.438	-1.0	-.713	-.397	-.198	-.019	.097	.219	.328	.429	.524	.613
1.60	.119	.423	-1.0	-1.000	-.466	-.222	-.018	.176	.231	.373	.465	.397	.652
1.80	.143	.423	-1.0	-1.0	-.556	-.253	-.013	.130	.222	.413	.535	.490	.767
2.00	.175	.433	-1.0	-1.0	-.682	-.292	-.020	.141	.305	.450	.582	.704	.822
2.20	.200	.438	-1.0	-1.0	-1.00	-.330	-.043	.148	.336	.483	.624	.755	.874
2.40	.223	.435	-1.0	-1.0	-1.0	-.393	-.070	.153	.315	.513	.664	.832	.931
2.60	.244	.436	-1.0	-1.0	-1.0	-.450	-.099	.155	.351	.510	.701	.867	.981
3.00	.263	.413	-1.0	-1.0	-1.0	-.537	-.171	.155	.374	.566	.735	.890	1.015
3.00	.279	.446	-1.0	-1.0	-1.0	-.623	-.147	.153	.388	.590	.763	.934	1.063
3.20	.294	.450	-1.0	-1.0	-1.0	-.826	-.177	.140	.379	.612	.797	.969	1.121
3.40	.308	.455	-1.0	-1.0	-1.0	-1.0	-.210	.142	.408	.622	.837	1.046	1.159
3.60	.321	.460	-1.0	-1.0	-1.0	-1.0	-.248	.134	.416	.631	.855	1.011	1.229
3.80	.333	.465	-1.0	-1.0	-1.0	-1.0	-.299	.125	.432	.669	.871	1.025	1.251
4.0	.345	.470	-1.0	-1.0	-1.0	-1.0	-.336	.114	.479	.685	.911	1.098	1.290
4.20	.356	.475	-1.0	-1.0	-1.0	-1.0	-.398	.102	.432	.701	.931	1.179	1.337
4.40	.374	.483	-1.0	-1.0	-1.0	-1.0	-.517	.074	.440	.730	.975	1.299	1.397
5.00	.397	.472	-1.0	-1.0	-1.0	-1.0	-.712	.011	.411	.755	1.025	1.255	1.464
5.40	.407	.505	-1.0	-1.0	-1.0	-1.0	-1.0	.0011	.413	.778	1.069	1.307	1.529
6.00	.428	.411	-1.0	-1.0	-1.0	-1.0	-1.0	-.088	.438	.808	1.114	1.331	1.671
7.0	.456	.328	-1.0	-1.0	-1.0	-1.0	-1.0	-.214	.459	.842	1.195	1.482	1.739
8.0	.480	.342	-1.0	-1.0	-1.0	-1.0	-1.0	-.433	.386	.857	1.264	1.591	1.855
9.0	.500	.355	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	.342	.887	1.323	1.681	2.049

2.- De forma regular

Si para el caso presente usamos las fórmulas anteriores se pueden hacer consideraciones como $\bar{h} = h$ como primera aproximación.

Una segunda aproximación sería hacer segmentos y usar las ecuaciones 1 y 2.

Para reducir el rango de error que esto implica se puede incorporar un término de 2o. orden.

$$④ \quad \Delta S_i = h_T\left[\sqrt{\frac{3NKU^2\Delta x}{g\,(h_T)^2} + 1} - 1\right]$$

$$⑤ \quad h_T = \bar{h}_i + \sum_{i=1}^{(i=n-1)} \Delta S_i$$

$$⑥ \quad S_m = \sum_{i=1}^{i=m} \Delta S_i$$

ΔS_i = incremento de altura en la i-sección.
Δx = el largo de la sección.
h_T = la altura original de la sección.
h_i = la profundidad media en la sección.
N = factor de forma plana.
i = número de la sección.

Un valor efectivo de el esfuerzo para esta fórmula se obtiene:

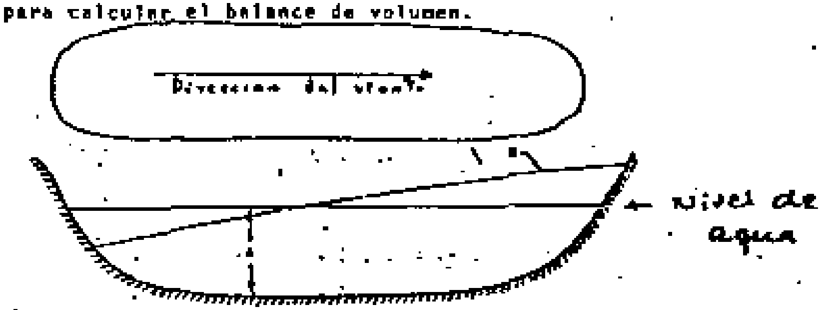
$$⑦ \quad \frac{KU^2 l}{g\,h^2} = \sum_{i=1}^{m}\left(\frac{KU^2\Delta x}{g\,(h_i)^2}\right)$$

de las tablas anteriores, se calcula el punto nodal de donde se empiezan los cómputos.

Se usa

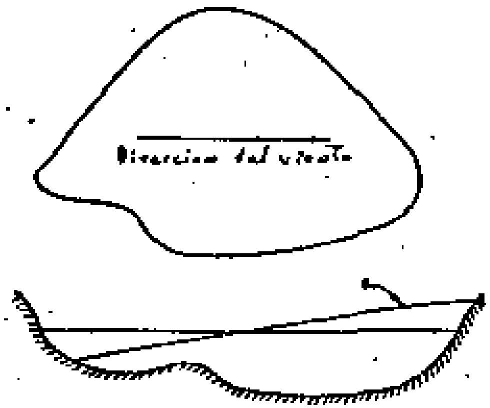
$$⑧ \quad \sum_{i=1}^{m} B_i S_i \Delta x = 0$$

para calcular el balance de volumen.



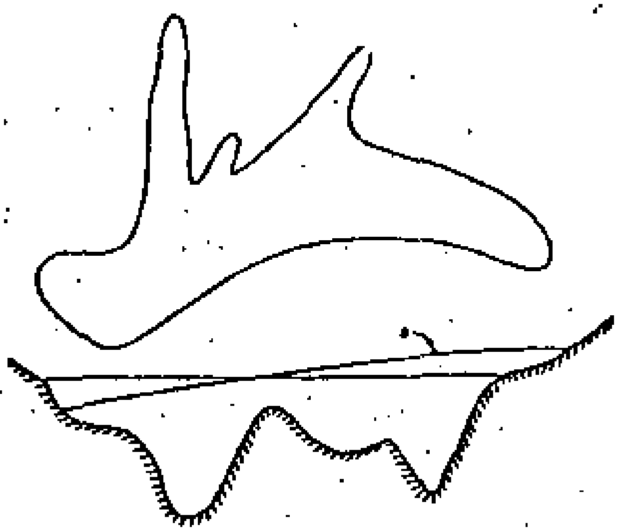
3.- De forma semiregular

Se puede proceder como en el caso anterior.



4.- De forma muy irregular.

Para estos casos no es posible definir una diferencia de alturas especialmente si la dirección del viento es cambiante.



5.- Viento incidiendo en un ángulo al eje mayor del reservorio.

Para vientos incidiendo en un ángulo la fórmula anterior se puede usar reemplazando:

$$U^2 \Delta x \quad por \quad u\, u_x\, \Delta x$$

y resolver para ΔS_x o sea la elevación en la dirección X

Se repite para Y

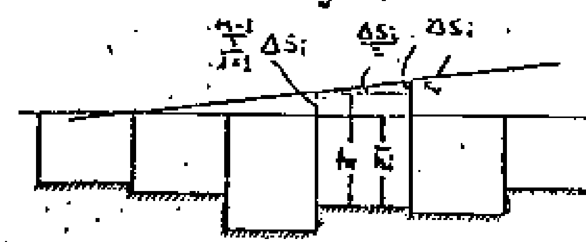
$$u^2 \Delta x \; por \; u\, u_y\, \Delta y$$

obteniéndose ΔS_y o la elevación en Y

Las dos líneas nodales X_{nod}/ℓ y Y_{nod}/ℓ proporcionan el punto nodal en el lugar de su intersepción.

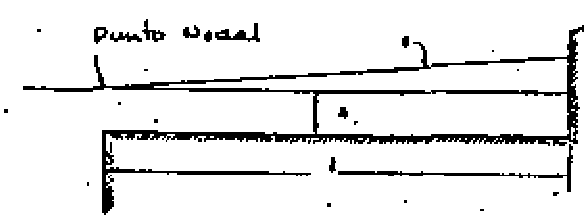
Los cómputos se inician en el punto nodal usando la ecuación ©, donde

$$\Delta S_i = \Delta S_x + \Delta S_y$$



B).- Fuera de la Costa o en la Plataforma Continental.

1. Fondo de profundidad constante y viento perpendicular a la costa.



Si esta condición existiera la sobreelevación por efecto de viento estaría dada por la solución de (1)

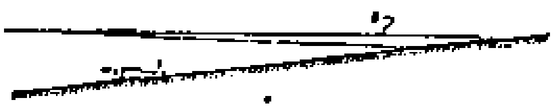
$$S = \frac{K U^2 x}{g\left(h+\frac{S}{2}\right)} \qquad (10)$$

Siendo (10) cuadrática, la raíz positiva proviene de:

$$S = h\left[\sqrt{\frac{2KU^2 x}{g h^2}+1}-1\right] \qquad (11)$$

Las ecuaciones (6) y (11) son muy similares.

2.- Fondo de pendiente constante



El caso se puede resolver usando

$$S_{max} = K\,\frac{T_a}{C_t}\left[\frac{h_1}{h_0}\right]^{1/4} U^2_{max}\,\partial \qquad (12)$$

donde

* K = 3.0 x 10^{-6} coeficiente de esfuerzo por viento.

h_1 = profundidad al margen de la plataforma.

h_0 = profundidad donde Smax efecta (lugar de cómputo)

$C_1 = \sqrt{g h_1}$

$C_0 = \sqrt{g h_0}$

$\bar{C} = \frac{1}{2}(C_1 + C_0)$

$T_a = \ell / \bar{C}$

* K no considera τ en el fondo.

ℓ = ancho de la plataforma continental.

U_{max} velocidad máxima del viento

∂ = factor de repuesta

La ecuación (12) no toma en cuenta el efecto de 2o. orden debida a S cuando h_0 tiende a cero.

Si sustituimos $h_0 = S + h_0$ obtenemos una mejor aproximación en la línea de costa.

Es posible derivar una fórmula para este caso asumiendo.

Usando (5) y de la figura anterior

$$h = h_1 - m_1 x \qquad (13)$$

donde

m_1 es la pendiente del fondo

e integrando de $x_1 = 0$ en h_1

hasta x en h

El término S en el denominador de la parte derecha de la ecuación se puede aproximar en

$$S = m_2 x \qquad (14)$$

donde m_2 es el término de 2o. orden, o sea, la pendiente media de la superficie del agua, y es considerada constante.

Con estas modificaciones (5) quedaría

$$S = \frac{K U^2}{g(m_1 - m_2)}\ln\left[\frac{h_1}{h_1 - (m_1 - m_2)x}\right] \qquad (15)$$

y de (13) y (14)

$$S = \frac{k\,U^2_x}{g\,(h_1 - h - S)}\; l_n \left[\frac{h_1}{(h+S)}\right] \quad (16)$$

Si para un fondo de profundidad constante $h_1 = h$, y usando el primer término

para $h / h+S$ entre 0 y 1

la ecuación (16) queda exacta a (15)

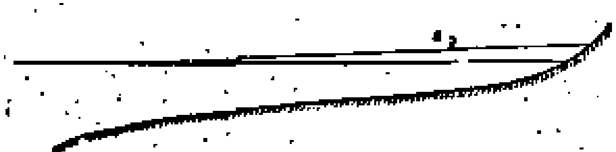
la ecuación (16) en este caso sirve bien como primera aproximación cuando la pendiente del fondo no es muy grande otro sistema de solución es usando (8)

Si el viento es variable en dirección y velocidad se puede aproximar sustituyendo U^2 por U_p^2

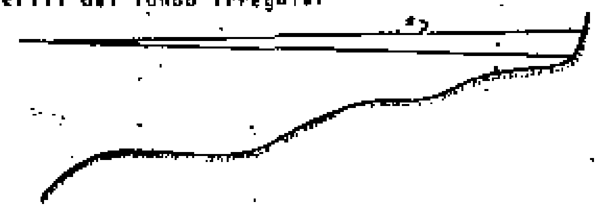
$$U_l^2 = 1/l \int_{x_2}^{x_1} |U|(U_x)\,dx$$

donde $l = x_1 - x_2$

3.- Perfil de fondo ligeramente irregular. Como una buena aproximación se puede utilizar el método anterior o una serie de fondos de pendiente constante.



4.- Perfil del fondo irregular



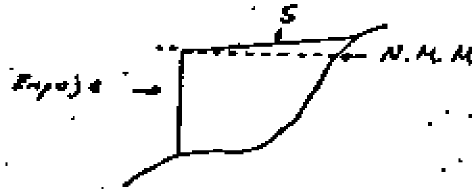
La predicción de altura para este caso no resulta aceptable, al hacerla únicamente por el método propuesto. Si consideramos que dependiendo del lugar de generación de una tormenta podemos "predecir" el comportamiento de ésta y a la vez como se dan familias de tormentas por época del año. Se puede predecir; si se conoce los datos de dirección de aproximación, velocidad y dirección de viento, velocidad de traslación, de tormentas pasadas, el comportamiento de las próximas, al conocer el nivel de agua y los datos de viento en la costa.

5.- Viento en ángulo a la Costa.

En las secciones B.1, B.3 y B.4 se considera el viento normal a la costa y constante.

En B.2 se considera una dirección de viento perpendicular a la costa como primera aproximación y después se descompone en componentes normal y paralelo, para cuando incide normalmente.

Si el viento corre paralelamente al contorno de la costa se produce una marea denominada batistrófica; o sea que esta creada por el empuje del agua o del viento en la plataforma y que crea un prisma en la costa.



La amplitud y de hecho la creación de la marea depende de la topografía submarina.

Realmente esta es una marea debida al efecto de Coriolis.

Si al cambio de nivel de mar S lo consideramos afectado por f o fuerza de Coriolis solamente cuando haya un flujo paralelo a la costa tendremos:

$$\frac{dS}{dx} = \frac{\kappa U U_x}{g(h+S)} + \frac{f\,F_y}{g(h+S)} \quad (18)$$

donde

U es el valor absoluto de la velocidad del viento.
U_x la componente del viento perpendicular a las líneas batimétricas.

$f = 2\,\omega\,\mathrm{sen}\,\phi$ Coriolis

ω velocidad angular terrestre
ϕ la latitud

F_y el flujo paralelo a la costa cuando

$$F_y = \int_0^{(h+S)} v\,dz \quad (19)$$

v la velocidad del agua paralela a las batimétricas.

Si la ecuación para F_y es dependiente del tiempo se representa por:

$$\frac{\Delta F_y}{\Delta t} = \kappa U U_y - \frac{K}{(h+S)^{7/3}} F_y^2 \quad (20)$$

siendo U_y el componente del viento paralelo a los contornos del fondo; para κ el valor es de 3.0×10^{-6} y K debe estar entre un rango de 10^{-3} a 10^{-2} y se ajusta localmente.

Para un intervalo de tiempo de (const.) velocidad constante de tiempo, la solución de (20) quedará:

$$F_{y(2)} = \sqrt{\frac{\kappa U U_y (h+S)^{7/3}}{K}}\ \mathrm{Tanh}\left[\sqrt{\frac{\kappa K U U_y}{(h+S)^{7/3}}}\,(t_2 - t_1) + \mathrm{Tanh}^{-1}\frac{F_{y(1)}}{\sqrt{\frac{\kappa U U_y (h+S)^{7/3}}{K}}}\right]$$

(1) y (2) corresponden a los tiempos inicial y final.

C).- En la línea de costa

1.- Línea de costa recta o regular.

Los métodos anteriores se pueden usar para los diferentes fondos que se presenten. Se hace la consideración que, las barreras son lo suficientemente altas para impedir que el agua las cubra, cuando la zona costera a estudiar, sea una barrera.

Para un fondo de pendiente constante la elevación en la costa para $h = 0$ usamos (7) que quedaría:

$$S = \frac{K U^2 x}{g(h_1 - S)} \ln \frac{h}{S} \qquad (23)$$

En general la pendiente del fondo incrementa rápidamente al acercarse a la playa, entonces para las zonas de playa se usa (23) y para las partes más profundas (26) para cuando $\beta \neq 0$; si el fondo incrementa rápidamente su pendiente -- se puede resolver por secciones.

2.- Línea de costa algo irregular.

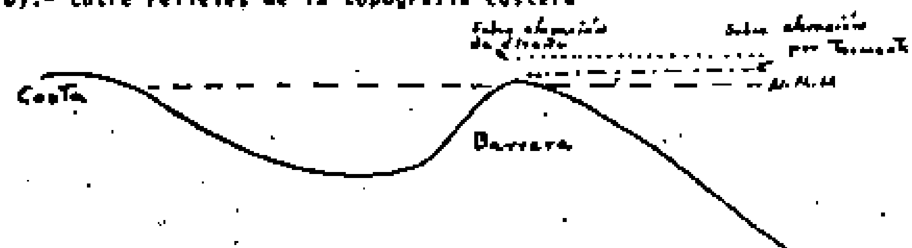
Se necesita hacer calibración de las fórmulas con datos anteriores.

3.- Línea de costa con salientes

En estos lugares para tener un buen diseño se debe enfatizar en recopilar datos anteriores y hacer una buena calibración.

Es necesario que se considere que en la playa y línea costera aumenta el nivel de agua por el oleaje generado por el viento de la tormenta; este incremento puede ser de 10 a 20 porciento del valor de S.

D).- Entre relieves de la topografía costera



1.- Una barrera natural alta será aquella que la sobre elevación de diseño no llegue a cubrir.

2.- La barrera media aquella que la elevación creada - por tormenta no cubre, pero la de diseño si (fig. anterior).

3.- La barrera media aquella que la sobreelevación de tormenta cubre.

El caso D.1. se trata como problema del capítulo C.

El Caso D.3 es difícil de tratar, aunque cuando empieza la elevación de nivel a cubrir la barrera se utilice las fórmulas para vertedores; despues se pierde significancia.

El otro caso se trata como vertedor si se conocen las dimensiones de la cuenca entre la barrera y la costa.

$$Q = C_1 L H^{3/2} \qquad C_1 = 1.893$$

E).- Bahías y Estuarios

Si existe una barra que separe al cuerpo de agua del mar, es necesario calcular como en el inciso pasado si la sobreelevación por tormenta va a cubrirla. También debe considerarse el flujo que aporten los ríos y el tamaño de la cuenca receptora.

El gasto se puede obtener de:

$$Q = C_2 A \sqrt{2g\,|(h_0 - h_1)|} \qquad (24)$$

y el volumen

$$V = \int Q \, d\tau \qquad (25)$$

El área A dependerá de h_o, o sea, la elevación estuaria o bahía y h_1 es la del mar.

El coeficiente C_2 se puede ajustar para cada caso; pero -- como primera aproximación se puede considerar,

$$C_2 = 0.6 \ .$$

2.1.2.- LAS TRES ASINTOTAS

Hasta el momento la teoría estadística ha proporcionado tres tipos de distribuciones extremales asintóticas, de las que a continuación se describen someramente sus características básicas:

1) ASINTOTA - I

Se deduce a partir de distribuciones de la variable cuya cola de interés es ilimitada y tiene una forma exponencial que se puede representar por:

$$F(x) = 1 - e^{-g(x)} \qquad (2.1.2.)$$

donde g(x) es una función creciente de x .

70 La expresión de la primera asíntota es, para el mayor valor:

$$\phi(x) = \exp\left[e^{-\alpha(x-u)} \right] \qquad (2.1.3.)$$

Esta expresión es denominada distribución doble exponencial, y también distribución de Fisher - Tippett, o de Gumbel.

Para el menor valor, la primera asíntota es:

$$\phi(x) = 1 - \exp\left[e^{-\alpha(x-u)} \right] \qquad (2.1.4.)$$

donde α, u son los parámetros de las distribuciones.

Son numerosas las funciones de distribución cuya cola superior tiene la forma exponencial indicada arriba. Entre ellas, pueden citarse:

La distribución Normal

$$F(x) = \frac{1}{\sigma\sqrt{2\pi}} \int_{-\infty}^{x} e^{-\frac{1}{2}\left(\frac{x-u}{\sigma}\right)^2} dx \qquad (2.1.5.)$$

La Logística

$$F(x) = \frac{1}{1+e^{-A(x-B)}} \qquad (2.1.6.)$$

La exponencial

$$F(x) = 1 - e^{-\frac{x-A}{B}} \qquad (2.1.7.)$$

La distribución gamma

$$F(x) = 1 - e^{-x} \sum_{k=0}^{v-1} \frac{x^k}{k!} \qquad (2.1.8.)$$

La de Weibull

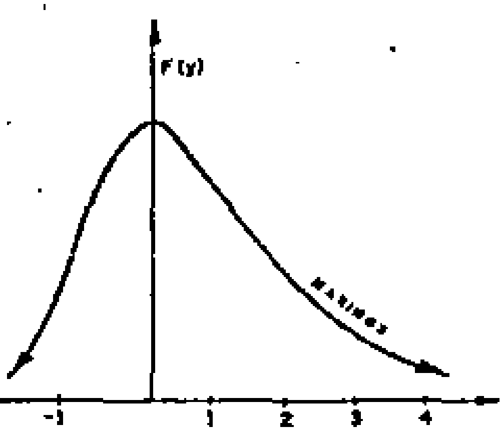
$$F(x) = 1 - e^{-\left(\frac{x-A}{B}\right)^C} \qquad (2.1.9.)$$

La propia distribución doble exponencial pertenece también a esto tipo

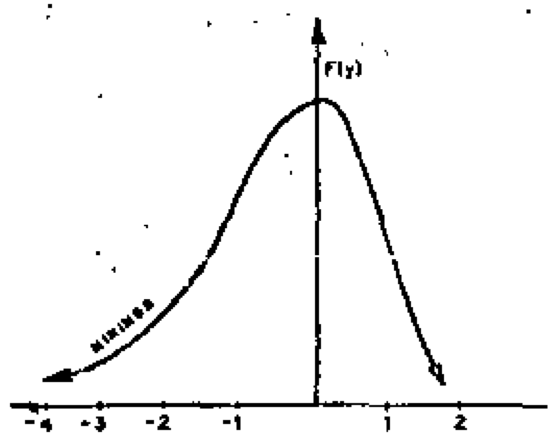
$$F(x) = e^{-e^{-\alpha(x-u)}} \qquad (2.1.10)$$

La Asíntota - I tiene una función de densidad asimétrica, y admite valores negativos de la variable.

La forma de su función de densidad es la siguiente:



Y = VARIABLE REDUCIDA
MAYOR VALOR, EC.2.2.3.



Y = VARIABLE REDUCIDA
MENOR VALOR, EC.2.2.4.

2) ASINTOTA - II

Se deduce a partir de funciones de distribución de la variable cuya cola de interés tiende asintóticamente a la forma:

$$F(x) = 1 - A \left(\frac{1}{x}\right)^{B} \qquad x \geq o \qquad (2.1.11.)$$

En el otro extremo, las distribuciones están limitadas por el valor cero.

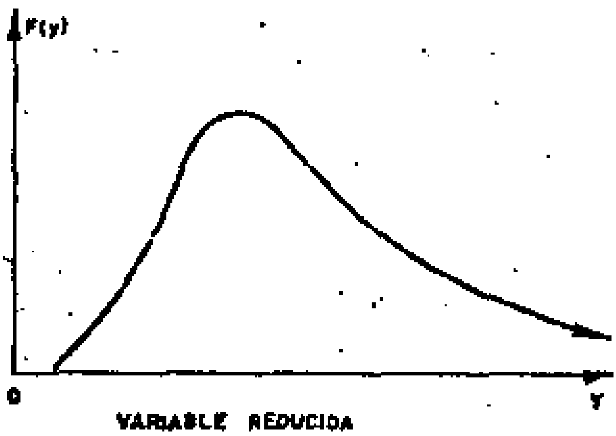
La segunda Asíntota es también denominada distribución de Fréchet. La forma en que la obtuvo este autor es:

$$\Phi(x) = e^{-\left(\frac{\lambda}{x}\right)^{B}} \qquad \begin{matrix} x > 0 \\ B > 0 \end{matrix} \qquad (2.1.12.)$$

E. Gumbel (1.957) propuso usar una expresión más general, que incluye un límite inferior $\underline{\epsilon}$ para la variable:

$$\Phi(x) = e^{-\left(\frac{\lambda-\epsilon}{x-\epsilon}\right)^{B}} \qquad (2.1.13.)$$

La forma de su función de densidad es:



La segunda Asíntota puede obtenerse a partir de la primera, por medio de una transformación logarítmica:

Si $\underline{\ln (x)}$ tiene una distribución Asíntota - I, entonces $\underline{x}$ tiene una distribución Asíntota - II en su forma menos general (Fréchet, Ec. 2.1.12).

Si $\underline{\ln (x-\epsilon)}$ tiene una distribución Asíntota - I, entonces $\underline{x}$ tiene una distribución Asíntota - II en su forma generalizada (Gumbel, Ec. 2.1.13.).

La cola de la Asíntota - II es más larga que la de la Asíntota - I, por lo cual produce pronósticos más elevados. - Por lo demás, la función de densidad es también asimétrica pero no admite valores negativos de la variable.

Un ejemplo de distribución que cumple la condición expresada en la Ec. (2.1.11.) en la cola de interés, es la propia distribución de Fréchet.

3) ASINTOTA - III

Se deduce a partir de funciones de distribución que son limitadas precisamente en la cola de interés.

Cuando el interés reside en los valores mayores, -- existe un límite máximo A. Las funciones que dan lugar a la tercera Asíntota, tienen la forma siguiente en las proximidades de ese límite:

$$F(x) = 1 - b\,(A-x)^{C} \qquad x < A,\; C > 0 \qquad (2.1.14.)$$

Y la expresión correspondiente de la tercera Asíntota para el valor mayor es:

$$\Phi(x) = e^{-\left(\frac{A-x}{A-B}\right)^{C}} \qquad x < A \qquad (2.1.15.)$$

Si se trata de los valores menores, cuyo límite inferior es A, la cola de la función de distribución de la variable debe tener la forma, cerca de A :

$$F(x) = b\,(x-A)^{C} \qquad x > A \qquad (2.1.16.)$$

Y la expresión correspondiente de la tercera Asíntota para el valor menor es:

$$\Phi(x) = 1-e^{-\left(\frac{x-A}{B-A}\right)^{C}} \qquad x > A \qquad (2.1.17.)$$

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A esta forma particular de la tercera Asíntota se denomina generalmente distribución de Weibull, quien la usó por primera vez para el análisis de rotura de materiales cuyo límite inferior de resistencia y de fatiga está representado por el parámetro A.

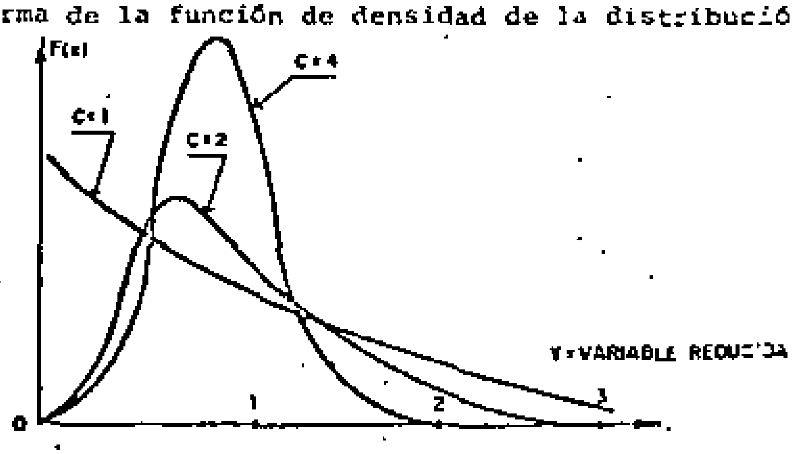
Esta distribución tiene un interés particular debido al uso extensivo que se ha hecho de ella en estudios de oleaje. Curiosamente no se la ha empleado para estudiar las propiedades extremales mínimas sino que, por el contrario, se ha utilizado la cola opuesta de la distribución para extrapolar a valores grandes. Aun más, su uso más extendido no es como función de distribución extremal (en estudios de oleaje máximo, ver por ejemplo C. Petrauskas y P. Aagard, 1971; L.P. Brown et al, 1.975), sino como función de distribución de la variable (también en estudios de oleaje, se pueden citar entre otros a C. Bretschneider, 1.965; N. Nordestrom, 1.969; J. Battjes, 1.970; B. Pedersen, 1.971; K. Holte, 1.973; etc.). Evidentemente, ambos usos están desconectados por completo de la justificación teórica que ha permitido formular la Ec. (2.2.17.) como función de distribución asintótica extremal.

Las distribuciones Asíntota - III del mayor y menor valor están también relacionadas con las correspondientes Asíntotas - I del mayor y menor valor, mediante transformaciones logarítmicas:

Si $\ln(\frac{1}{A-x})$ tiene una distribución doble exponencial, entonces x tiene una distribución Asíntota - III del mayor valor.

Si $\ln(x-A)$ tiene una distribución Asíntota - I del menor valor, entonces x tiene una distribución de Weibull.

La forma de la función de densidad de la distribución de Weibull es:



(75)

Un ejemplo de distribución de la variable que conduce a la Asíntota - III del menor valor (Weibull), es la distribución Gamma en su cola inferior.

En cuanto al parámetro n, una hipótesis básica implícita en las deducciones de las tres Asíntotas es n=constante. Por otra parte, la buena convergencia de las Asíntotas con sus correspondientes distribuciones extremales depende de que n tome un valor suficientemente alto (además, naturalmente, de que la cola de la distribución F(x) converja a su vez con suficiente rapidez hacia la forma característica indicada para cada Asíntota).

2.1.3.- ESTIMA DE LA FUNCION DE DISTRIBUCION A PARTIR DE UNA MUESTRA EXTREMAL.

El ajuste de una muestra extremal puede servir a dos propósitos distintos: Por un lado, constituye el elemento básico para realizar un contraste directo de la validez del tipo de distribución extremal propuesto. Por otro, una vez decidido el tipo de distribución que va a utilizarse (mediante el contraste anterior o siguiendo otros criterios), proporciona la estima de los parámetros de esta distribución, con la que se realizarán las extrapolaciones. En general (B. Kimball, 1.960) los métodos considerados como más adecuados a estos dos propósitos no coinciden necesariamente.

La estima de la distribución a partir de una muestra extremal tiene dos etapas sucesivas con problemáticas específicas. La primera se refiere a asignar a cada valor muestral una determinada probabilidad, denominada generalmente "frecuencia de representación" ya que se utiliza para representar el valor en los (comunmente utilizados en el análisis de funciones de distribución) papeles probabilísticos. Los papeles probabilísticos son sistemas coordenados que linearizan las funciones de distribución, permitiendo contrastes gráficos sencillos y rápidos. En el Apéndice I se trata sobre la construcción y uso de los papeles probabilísticos, y se incluyen las funciones de distribución que aparecen a lo largo de este trabajo. En cuanto a la segunda etapa, cor

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Revista de OBRAS PUBLICAS

Año CXXIII | JUNIO 1976 | Núm. 3131

UN METODO PRACTICO DE PREVER OLEAJE EXTREMAL PARA EL CALCULO DE ESTRUCTURAS MARITIMAS(*)

Por ENRIQUE COPEIRO DEL VILLAR

Dr. Ing. de Caminos, Canales y Puertos

La determinación de la altura de ola de cálculo es básica para el proyecto de todo tipo de estructuras marítimas. Los métodos habituales se basan en fórmulas empíricas poco adaptables a los estudios económicos que dependen de la vida previsible de la obra. En el artículo se presenta un método para la determinación del régimen extremal de oleaje, a partir de la distribución frecuencial de alturas de ola significante en el año medio. Para ello se usa un parámetro que depende exclusivamente de la duración media de los temporales y la frecuencia de presentación del oleaje en el vector direccional escogido. Finalmente se aplica a la zona NE. del Atlántico y el litoral español en San Ciprian (Lugo), zona de ubicación de una gran planta industrial.

1. Descripción del tema.

Durante los últimos años, ha sido publicada una cantidad considerable de datos de oleaje referentes a todos los océanos. La gran mayoría de ellos consisten en distribuciones frecuenciales de alturas de ola y períodos significantes estimados visualmente por observadores a bordo de barcos, bien en posiciones estables (barcos meteorológicos), o en ruta (marina de guerra o barcos mercantes voluntarios). Tres buenos ejemplos de compilaciones de datos visuales son las "Sea and Swell Charts" (U.S. Hydrographic Office), "Ocean Wave Statistics" (Hogben and Lumb, National Physics Lab., U.K.) y el excelente trabajo de H. Walden "Die Eigenschaften der Meereswellen in Nordatlantischen Ozean" (Deutscher Wetterdienst Seewetteramt Hamburg). Recientemente están siendo publicados, según un programa de la Organización Meteorológica Mundial, resúmenes anuales de datos de oleaje correspondientes a los barcos meteorológicos y a determinadas áreas marítimas de particular interés (por ejemplo, "Marine Climatological Summaries" del Atlántico Norte, publicados por Gran Bretaña, y el Mediterráneo, a cargo de Holanda).

Es harto frecuente que el proyectista de una obra marítima no disponga de más fuentes de datos de oleaje que las mencionadas arriba. Como mucho, en algunos proyectos de gran envergadura se ha podido disponer de un año para medir las características del oleaje *in situ* con un registrador.

Sin embargo, ni los resúmenes de unos cuantos años de datos de barcos en ruta ni las hipotéticas medidas instrumentales durante un año, suministran directamente el régimen extremal de oleaje.

Este régimen es, por otra parte, imprescindible para un diseño adecuado de las obras. La dificultad de su obtención hace que el proyectista acuda, con demasiada frecuencia, a viejas fórmulas empíricas que suministran una "ola de cálculo". La estructura determinista de esas fórmulas las hace inadaptables a los métodos habituales de cálculo económico, que de-

(*) Se admiten comentarios sobre el presente artículo, que pueden remitirse a la Redacción de esta revista hasta el 30 de septiembre de 1976.

penden de la vida previsible de la obra. El sentido físico de esta "ola de cálculo", que no está vinculada a período de retorno alguno, es muy problemático. Por otra parte, resulta desconocida la eficiencia intrínseca de aquellas fórmulas fuera de las áreas específicas en que fueron taradas a partir de observaciones.

En este trabajo se propone calcular el régimen extremal de oleaje, para cualquier sector direccional, a partir de la distribución frecuencial de alturas de ola significante en el año medio. Para ello se usa un parámetro que depende exclusivamente de la duración media de los temporales y la frecuencia de presentación del oleaje en el sector direccional escogido.

La duración media de los temporales que sobrepasan una cierta altura de ola sólo puede estimarse directamente a partir del análisis de la curva de estados del mar, tal como es suministrada por observaciones hechas en un punto fijo y con un intervalo de tiempo reducido. Por lo tanto, la información de los barcos en ruta no es de utilidad a este respecto. Sí lo es, en cambio, la suministrada por los barcos meteorológicos (tres a una hora de intervalo) o los registradores de oleaje colocados *in situ*.

Conocida la ley de variación de la duración media de temporales con la altura de ola en varios puntos de la costa, algunas de las leyes correspondientes a puntos intermedios pueden hallarse por interpolación. A este respecto, la información que suministra la red de registradores de oleaje existente en la costa española (descrita por P. Suárez Bores, 1973) podría ir rindiendo ya resultados útiles, aunque el número de años registrado no sea suficiente para confeccionar directamente regímenes extremales fiables. Esa red de registradores no discrimina direcciones y, por tanto, su información sería únicamente aplicable al régimen escalar. Para el uso correcto de las estimas de duración de temporales hechas a partir de un número reducido de años y, por tanto, no muy exactas, es necesario conocer la forma general de la relación duración-altura de ola, punto que se discutirá más adelante. De este modo, se puede ajustar la ley general a los datos usando, por ejemplo, unos ejes coordenados graduados en forma conveniente.

Las duraciones de temporales, direccionales o escalares, no siempre son directamente interpolables entre lugares distintos de la costa. Para juzgar la conveniencia de las interpolaciones es necesario tener conocimiento de las característicos generales de la generación y desarrollo de los temporales en el mar. Las cartas de densidad de borrascas son un instrumento muy valioso a este respecto; en un apartado posterior se muestran dos de tales cartas correspondientes al Atlántico norte.

Estos criterios de cálculo se han aplicado a una zona del NE. del Atlántico, y utilizado en el diseño de un nuevo puerto que va a construir en San Ciprián (Lugo) Alúmina Española Sociedad Anónima, y Aluminio Español, Sociedad Anónima.

Las leyes de variación de duración de temporales en varias direcciones han sido halladas en algunos barcos meteorológicos de esa zona del océano. Se ha comprobado así que las interpolaciones entre direcciones adyacentes, para un mismo lugar, y entre lugares distintos, para un mismo sector direccional o para el régimen escalar, son perfectamente válidas cuando se utiliza un criterio adecuado.

Las duraciones medias de los temporales no han sido halladas directamente, a partir de la curva de estados del mar. Para cada dirección se han calculado las duraciones a partir del régimen extremal y la distribución frecuencial absoluta de H_s (altura de ola significante), "volviendo del revés" el modelo estadístico. La bondad de los resultados obtenidos constituye, pues, además, una comprobación satisfactoria del modelo mismo.

Un número de barcos meteorológicos mayor que el existente en la actualidad, proporcionaría una red de regímenes de duración de temporales a partir de la cual, por interpolación y extrapolación, podrían obtenerse los parámetros de cálculo necesarios en todos los puntos deseados de la plataforma costera. La red actual de barcos es pobre, aun en zonas privilegiadas como el Atlántico norte, pero aún así ha podido suministrar información utilizable para la costa noroccidental española.

La escasez de información disminuye, obviamente, la exactitud de los resultados obtenidos de la aplicación de estos criterios. Existen otras fuentes de incertidumbre, además, cuyo peso es, a veces, muy difícil de precisar. Por ejemplo, la precisión de las observaciones visuales individuales y el efecto de los cambios generales de clima.

Indudablemente, a medida que se van estableciendo métodos adecuados de cálculo, irán también siendo puestos a punto sistemas de toma de datos que, en su día, alimentarán aquellos métodos. Sin embargo, mientras esos datos llegan habrá que seguir proyectando obras marítimas. El trabajo que se presenta aquí describe un caso en el que se muestra cómo se puede extraer utilidad práctica para el diseño, de los datos que hay disponibles "ahora".

2. El modelo estadístico.

El modelo se basa en el cálculo de la distribución del valor máximo que una variable X toma en un cierto intervalo de tiempo.

Sea X una variable aleatoria con una función de distribución:

$$P(X \leq Y) = F(X)$$

Si el suceso X se presenta, aleatoriamente, n veces dentro de un intervalo de tiempo, la distribución del máximo valor que toma X en ese intervalo es:

$$\Phi(X) = [F(X)]^n \qquad (2.1)$$

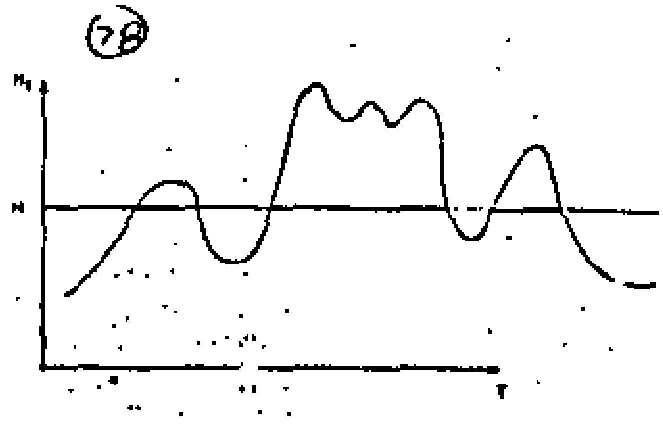
Esta es una regla probabilística elemental, según la cual la probabilidad de que un suceso aleatorio se presente todas las veces durante n "pruebas", es n veces la probabilidad de que aquel suceso ocurra en una "prueba" cualquiera.

Al aplicar esta teoría al oleaje aparecen varias dificultades.

La primera es determinar cuál es el "número de pruebas", n, que ocurre en la unidad de tiempo. Esta última debe tomarse igual a un ciclo meteorológico, o sea, un año. El problema está en el hecho de que la función de distribución de H_s no suministra información alguna sobre la duración del "estado del mar" caracterizado por H_s. Es necesario, pues, buscar una conexión entre H_s y duración, problema que quizá fue apreciado por primera vez por J. A. Battjes (1970).

La conexión buscada puede ser facilitada por el hecho de que H_s varía con el tiempo, a lo largo del año, describiendo una curva continua ondulada.

Se pueden definir "temporales" con $H_s \geq H$. Durante cada uno de estos "temporales" la curva de estados del mar permanece por encima de H, como se muestra esquemáticamente en



el dibujo anterior. La duración de esos temporales es variable y susceptible de medida. Se puede hablar de la duración media de los temporales que superan cierta altura de ola, $t(H_s)$. Es obvio que esa duración media aumenta al disminuir H_s.

La situación descrita arriba pude reducirse a un fenómeno con dos sucesos posibles: la presentación de un temporal ($H_s \geq H$), o su no aparición. El número total de veces que el temporal "podría" presentarse en un año, es la duración del año dividida por la duración media del temporal correspondiente. Ese es el "número de pruebas". La probabilidad de presentación de un temporal ($H_s \geq H$) en una "prueba" cualquiera, es precisamente el porcentaje del tiempo total en que se sobrepasa H, o sea, $[1 - F(H)]$.

La probabilidad de que el temporal ($H_s \geq H$) no se presente en el año es:

$$\Phi(H_s) = [F(H_s)]^n \qquad (2.2)$$

donde:

$$n = \frac{8760}{t(H_s)}; \quad (t(H_s) \text{ en horas}) \qquad (2.3)$$

$\Phi(H_s)$ es la probabilidad de que H_s sea el máximo estado del mar en el año, es decir, la distribución extremal de H_s.

El período de retorno de H_s es:

$$T(H_s) = \frac{1}{1 - [F(H_s)]^n} \qquad (2.4)$$

Para valores de $T(H_s)$ iguales o superiores a unos diez años, la expresión anterior puede ser sustituida por el período de retorno de las excedencias de H_s:

$$T_e(H_s) = \frac{1}{n[1 - F(H_s)]} \qquad (2.5)$$

El segundo problema es la aleatoriedad de las "pruebas". La teoría que permite formular la ecuación (2.5) requiere que todas las "pruebas" sean estadísticamente independientes, lo que significa que los temporales aparecen aleatoriamente. Esta es una hipótesis dudosa para valores medios y bajos de H_s, pero parece correcta para los valores elevados de H_s, que sólo ocurren muy raramente. Esos valores son precisamente, los que interesan a efectos de la descripción estadística de valores extremos. A este respecto, E. Gumbel (1957) comenta que, en distribuciones formadas con valores mutuamente dependientes, la influencia de esa independencia se anula para valores grandes de la variable. El mismo autor presenta una demostración, debida a G. S. Watson (1954), de aquella afirmación, demostración que es válida para la distribución asintótica del mayor valor de una variable ilimitada.

La hipótesis de aleatoriedad en la presentación de temporales excepcionales puede, por tanto, ser adoptada con confianza.

3. Caracterización de otros modelos mediante el parámetro "n".

El cálculo del régimen extremal de oleaje por medio del parámetro "n", tal como se define en la ecuación (2.3), es aplicable a otros modelos estadísticos sugeridos previamente.

K. G. Nolte (1973), en un estudio comparativo exhaustivo, analiza cinco modelos que predicen el período de retorno de alturas de ola individuales. Después de hacer correcciones en algunos de ellos, este autor encuentra que todos dan resultados muy similares. La aplicación de los modelos a alturas de ola significantes es inmediata. Sin embargo, el tipo de datos que deben ser introducidos en los modelos corregidos les restan considerablemente aplicabilidad práctica.

Los llamados por Nolte "modelos de temporal" se adaptan particularmente bien a ser caracterizados mediante el parámetro n. Estos modelos consideran las alturas de ola agrupadas en "temporales", como se ha hecho en la sección 2. Los modelos tienen básicamente la misma estructura. Un modelo de este tipo puede ser descrito como sigue:

Supongamos que el año medio contiene λ temporales que sobrepasan un cierto límite inferior, H_1. La función de distribución que utiliza en el modelo, $S(H_s)$, es la probabilidad de presentación de una altura de ola significante mayor que H_s en el temporal medio. El período de retorno es definido como el intervalo medio de tiempo entre temporales que superan H_s:

$$T_s(H_s) = \frac{1}{\lambda \cdot S(H_s)}$$

Se demuestra fácilmente que el período de retorno de H_s no cambia, cualquiera que sea el límite inferior H_1 escogido, siempre que $H_1 \leq H_s$. Por tanto, podemos escoger $H_1 = H_s$, con lo cual todos los temporales inferiores a ... quedan excluidos de la estadística.

Según esto, $S(H_s) = 1$ y λ = número de temporales superiores a H_s en el año medio.

$$T(H_s) = \frac{1}{\lambda}$$

λ puede ser hallado a partir de la duración media de los temporales.

$$\lambda = \frac{8760 \cdot [1 - F(H_s)]}{t(H_s)} = n \cdot [1 - F(H_s)]$$

Por lo tanto:

$$T(H_s) = \frac{1}{n[1 - F(H_s)]}$$

Que coincide con la ecuación (2.5).

4. Estimación de n.

La duración media de temporales extraordinarios no puede ser caracterizada adecuadamente promediando los valores de esas duraciones según fueron registradas, debido precisamente a la rareza de su presentación. Sin embargo, si se pudiera establecer la ley de variación de $t(H_s)$ con H_s, a partir de los temporales más abundantes en los registros, se podría extrapolar esa ley a valores superiores de H_s.

La estima de la duración media de temporales puede abordarse de dos formas:

4.1. *Estima directa.*

Se ha indicado ya cómo puede estimarse H_s a partir de la curva de estados del mar.

Las observaciones visuales suministradas por barcos en ruta dentro de una cierta área, no pueden ser utilizadas con este propósito, ya que su situación e intervalo varían aleatoriamente y no se facilitan con los datos de oleaje publicados.

Si se realizan medidas instrumentales *in situ*, el tiempo disponible es necesariamente corto y las duraciones medias estimadas constituyen sólo (especialmente si se pretende discriminar direcciones) una aproximación grosera.

La utilidad de los valores obtenidos así, aumenta considerablemente si se conoce de antemano la forma de la relación de $t(H_s)$ con H_s. Para caracterizar esa relación, se ha utilizado información de los barcos meteorológicos aplicando el método de estima indirecta como se expone a continuación.

4.2. *Estima indirecta.*

Si se dispone de una muestra extremal de H_s suficientemente amplia, en un punto dado, la función de distribución extremal $\Phi(H_s)$ puede ser estimada a partir de aquélla y la ecuación (2.2) permite calcular n con la expresión:

$$n = \frac{\log \Phi(H_s)}{\log F(H_s)} \qquad (4.1)$$

La aplicación de esta ecuación a varios sectores direccionales, en los barcos meteorológicos del Atlántico NE, proporciona una relación aproximadamente exponencial entre n y H_s, para valores altos de H_s, como se mostrará en un apartado posterior.

5. Curvas de diseño.

El uso de la función de distribución extremal, $\Phi(H_s)$, para el diseño de obras marítimas, no consiste en calcular directamente con la altura de ola correspondiente a un período de retorno igual a la vida prevista de la obra. La función que debe usarse en el diseño es la que da la probabilidad de que una cierta altura de ola H_s no sea superada en la vida de la obra, N. Esa función es:

$$\Phi_N(H_s) = [F(H_s)]^{nN} \qquad (5.1)$$

La función $\Phi_N(H_s)$ puede introducirse en los cálculos económicos usados habitualmente para determinar la altura de ola óptima. Se puede demostrar fácilmente que el uso de $\Phi_N(H_s)$ es equivalente a la aplicación del criterio de riesgo, tal como ha sido definido, por ejemplo, por A. Court (1953), a la distribución extremal $\Phi(H_s)$.

Dada una vida previsible de la obra N, y un riesgo admitido R de que la altura de ola significante de cálculo H_c sea sobrepasada durante aquella vida, el período de retorno correspondiente a H_c es:

$$T(H_c) = \frac{N}{-\ln(1-R)} \qquad (5.2)$$

Para valores altos de N y bajos de R, como es habitual, la expresión anterior converge en:

$$T(H_c) = \frac{N}{R} \qquad (5.3)$$

Por otra parte, utilizando la ecuación (2.5) para el período de retorno de la distribución extremal $\Phi(H_s)$ (válida, como se advirtió, para valores de N superiores a unos diez años):

$$T(H_c) = \frac{1}{n[1 - F(H_c)]} \qquad (5.4)$$

Igualando las dos expresiones anteriores, se obtiene:

$$F(H_c) = 1 - \frac{R}{nN} \qquad (5.5)$$

Por otra parte, cuando se utiliza la función $\Phi_N(H_s)$, la probabilidad correspondiente a H_c, R, es:

$$R = 1 - [F(H_c)]^{nN} \qquad (5.6)$$

Esta expresión puede sustituirse de modo análogo y con la misma condición que se usó al reemplazar (2.4) por (2.5):

$$R = nN[1 - F(H_c)] \qquad (5.7)$$

De donde:

$$F(H_s) = 1 - \frac{R}{a N}$$

que es la misma expresión (5.5).

6. Aplicación del método a la costa NW española.

6.1. *Datos disponibles y criterio de cálculo.*

Las únicas fuentes de datos de oleaje referentes a la costa NW española que han podido encontrarse, son resúmenes de observaciones visuales realizadas en barcos en ruta. De entre ellos, tiene especial interés la publicación de la Organización Meteorológica Mundial "Marine Climatological Summaries for the Eastern North Atlantic" (Met. Inst., U.K.). En ella se presentan datos referentes a determinadas áreas reducidas de especial interés. Una de estas áreas, denominada número 4, está situada próxima a la costa N gallega (fig. 20), y proporciona la serie de datos más completa y específica que puede obtenerse, hasta el momento, de la zona relativa al proyecto de San Ciprián.

Los datos de la O.M.M. sirven para confeccionar regímenes de altura de ola significante en cualquier sector direccional escogido. Sin embargo, las duraciones medias de los temporales no pueden ser obtenidas de esta información por razones expuestas anteriormente. La obtención directa del régimen extremal, a partir de observaciones, no ha sido empleada por dos motivos: Primero, el número de años publicados es excesivamente corto. Segundo, las alturas de ola mayores no quedarían adecuadamente representadas, debido a la costumbre de los barcos de desviar su ruta para evitar el encuentro con el núcleo de los temporales. La importancia de este último punto ha sido subrayada por N. Hogben (1974).

La solución adoptada fue el calcular las duraciones de temporales correspondientes a algunos puntos del Atlántico NE, donde están situados permanentemente los barcos meteorológicos I, J, K (ordenados de N a S). El barco K está a la misma altura que el área 4 y unos 7° desplazado hacia el W. Los resultados obtenidos han servido para estimar el tipo de relación entre H_s y $t(H_s)$, y para ver la medida de la variación de las duraciones entre los lugares correspondientes a los tres barcos. De ella se han extraído conclusiones en cuanto a las duraciones que deben aplicarse al área 4. Con estas duraciones y los regímenes de H_s, se han calculado las curvas de diseño $\Phi_N(H_s)$.

Las distribuciones $F(H_s)$ han sido obtenidas, para cuatro sectores direccionales, del resumen de diez años publicado por H. Walden (1964).

Los datos de H. Walden constituyen "la distribución del sistema de oleaje dominante en cada instante". Esto quiere decir que, cuando una determinada observación de un barco diferencia entre dos o más trenes de ola presentes simultáneamente en varias direcciones, sólo se tiene en cuenta el tren con mayor H_s. Este tratamiento de los datos permite dividir el tiempo total del año en cuatro partes, cada una correspondiendo a un sector direccional. Por tanto, estas distribuciones direccionales de H_s no representan la verdadera probabilidad de ocurrencia de los valores inferiores de H_s (especialmente en las direcciones con menor energía de oleaje). En cambio, el rango superior de valores de H_s está descrito adecuadamente, y sus probabilidades no van referidas al tiempo total del año. Esta disposición del conjunto de datos es favorable a la aplicación del método extremal propuesto, como se verá más adelante. Los datos de los "Marine Climatological Summaries" son del mismo tipo.

En las figuras 1, 2, 3, 4 y 5 se muestran los regímenes direccionales y escalar correspondientes al barco K, ajustados por una distribución de Weibull. Esta distribución es, según ha demostrado J. A. Battjes (1970) con datos instrumentales, la más indicada para caracterizar los regímenes de altura de ola significante (en el Atlántico N.E., al menos). Debe puntualizarse que los regímenes trabajados por Battjes eran escalares y se desconoce, por tanto, si algunos regímenes direccionales pueden presentar desviaciones peculiares. De hecho, si bien todos los regímenes escalares utilizados en este trabajo se ajustan con una precisión sorprendente a la distribución de Weibull, en algunos regímenes direccionales el conjunto de puntos más altos parecen desviarse de la tendencia general. Es cuestionable si esto es una peculiaridad real del clima para esa área y dirección o si es una desviación accidental debido a la baja caracterización estadística de [illegible] puntos, que han sido calculados con un número [illegible].

En las figuras se señala con "Cambio de Código" a los puntos correspondientes a $H =$ $= 4,75$ m y $H =$ [illegible] m. Estos puntos deben desecharse de la [illegible] (N. Hogben, [illegible]), pues se ha demostrado que, al utilizarse un código diferente cuando H_s supera esos valores, el observador sufre una influencia subjetiva que se traduce en una distorsión apreciable de la frecuencia atribuida a esos valores.

Los regímenes extremales de H_s han sido obtenidos a partir de los datos publicados en *Taglicher Wetterbericht* (Met. Inst., Alemania Occidental). Estas publicaciones suministran la información de oleaje de los barcos meteorológicos con intervalos de tres horas. Para la confección de los regímenes, se han examinado los partes correspondientes a diecinueve años. Ocasionalmente aparecen errores de codificación en los partes, por lo cual se realizó un contraste de aquella información con la publicada en *The Daily Weather Report* (Met. Inst., Gran Bretaña), que tiene seis horas de intervalo. Las cartas meteorológicas que figuran en los partes fueron también consultadas en los casos dudosos.

En las figuras 6, 7, 8, 9 y 10 se muestran los diecinueve máximos valores de H_s registrados en el barco K, para las cuatro direcciones y su conjunto. La probabilidad de representación escogida es la recomendada por E. Gumbel (1975).

6.2 *Estima de las duraciones de temporales en los barcos meteorológicos.*

Utilizando la ecuación (4,1), se han calculado las duraciones de temporales para los cuatro sectores, W., N., E., S., y su conjunto, en el barco K. Para comparar las duraciones de temporales en las distintas direcciones, n debe ser dividido por la frecuencia de presentación, f, de la altura de ola dominante en cada dirección, ya que:

$$t(H_s) = \frac{8760 \cdot f}{n}$$

En la figura 11 se han representado las curvas H_s-duración (esta última representada por: $\frac{n}{f} = \frac{8760}{t(H_s)}$).

La duración de los temporales aumenta, en el barco K, según el orden S., E., N., W. El régimen escalar ocupa una posición intermedia. Para caracterizar las distribuciones extremales se han utilizado las curvas que mejor ajustan el conjunto de datos, trazadas a sentimiento. El grado de precisión de las observaciones visuales no aconseja procedimientos más refinados.

Para efectuar una comprobación de la conveniencia de utilizar interpolaciones cuando se quiere caracterizar direcciones intermedias, se ha utilizado un sector de 45º centrado en el NE. El interés de ese sector direccional estriba en su papel fundamental dentro del proyecto de San Ciprián. Debido a la peculiar configuración de los fondos en torno a la ensenada de San Ciprián, el oleaje de esta dirección experimenta una concentración que incrementa notablemente su altura en la zona de los futuros diques de abrigo.

En las figuras 12 y 13 se muestran las distribuciones absoluta y extremal de H_s para el NE. (45º) en el barco K. La curva de duraciones correspondiente se ha señalado a trazos en la figura 11. El resultado es muy satisfactorio, teniendo en cuenta el orden de exactitud de los datos.

Un segundo grupo de cálculos se refiere a la variación de los regímenes de duración con el cambio de lugar en el océano. Las figuras 14 y 19 muestran las distribuciones de H_s para las direcciones W. y E., y para el régimen escalar, en los barcos I y J. En las figuras 20 a 25 aparecen los correspondientes datos extremales. En las figuras 26, 27 y 28 se comparan las duraciones del régimen escalar y de las direcciones W. y E. en los barcos I, J, K.

Según puede apreciarse, para el régimen escalar y la dirección W. las duraciones aumentan en el sentido I, J, K, lo que no ocurre en el oleaje del E.

La elección de este conjunto de direcciones obedece a su importancia en San Ciprián. Los resultados obtenidos dan ocasión a apreciar cómo el orden de variación de las duraciones, tanto en las distintas direcciones del barco K como en una misma dirección para los tres barcos, I, J, K, es coherente con el régimen característico del desarrollo de los temporales en el Atlántico Norte.

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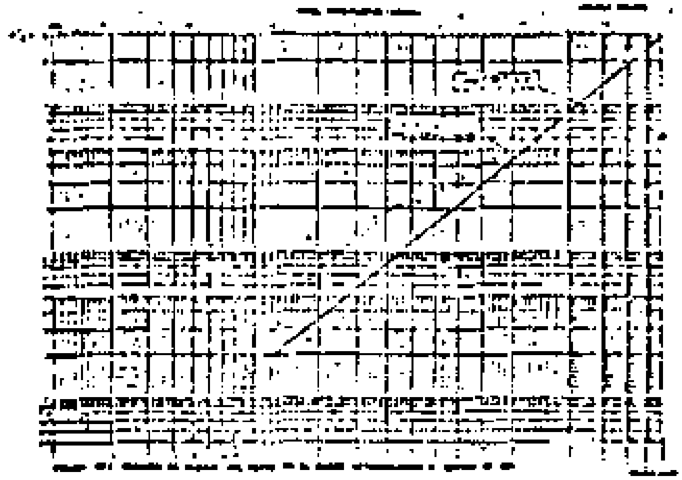

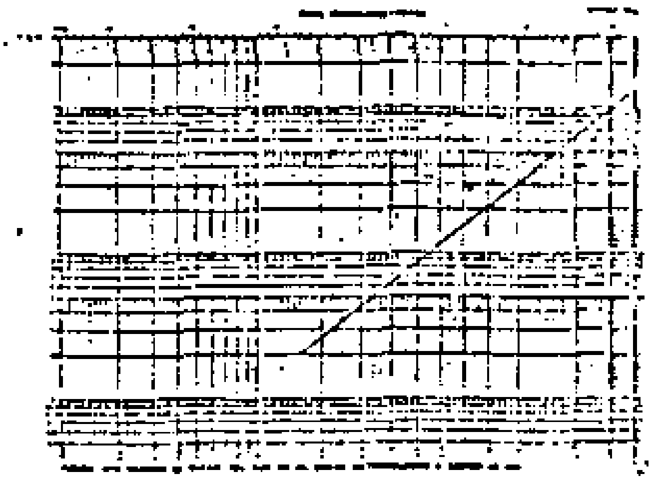

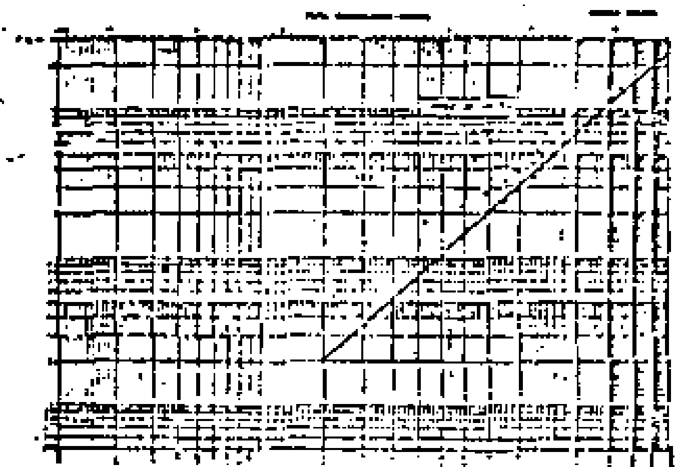

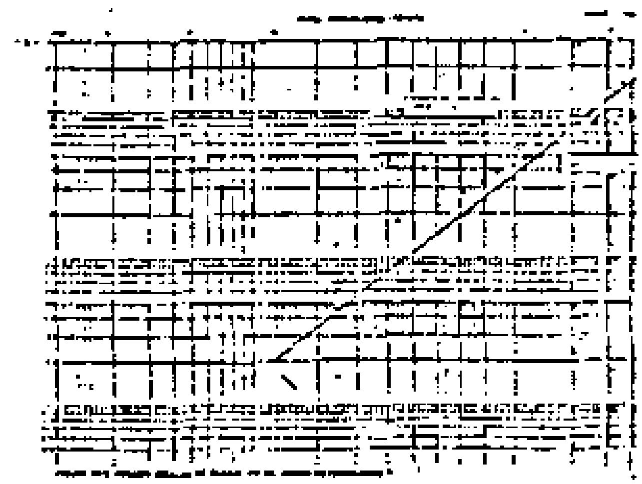

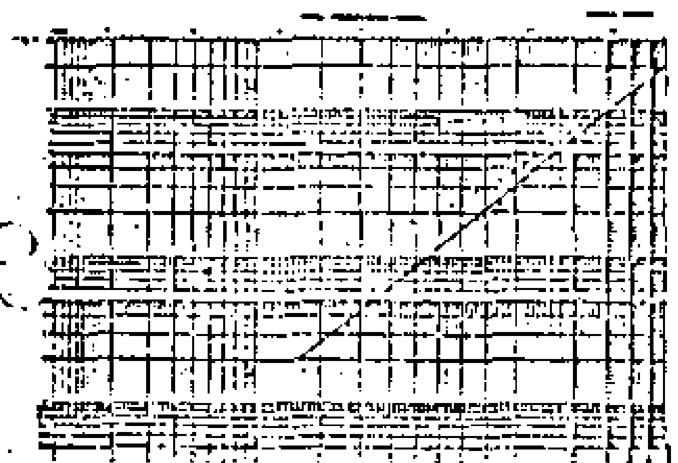

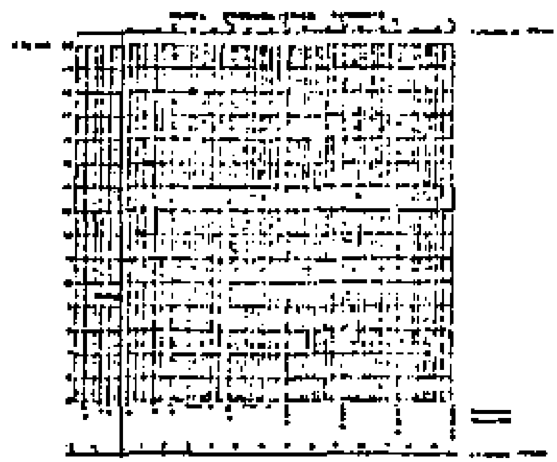

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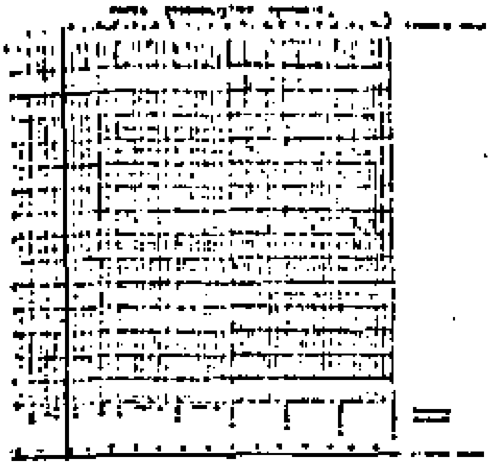

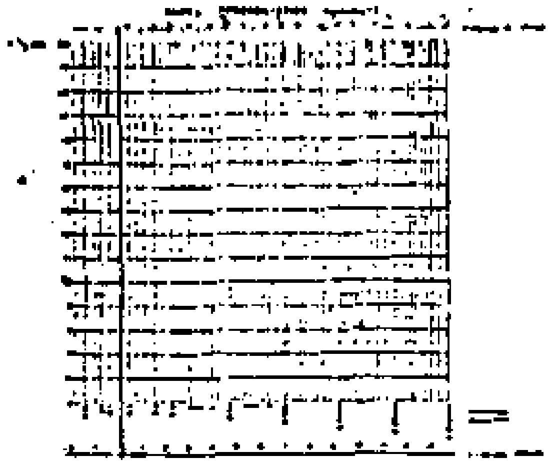

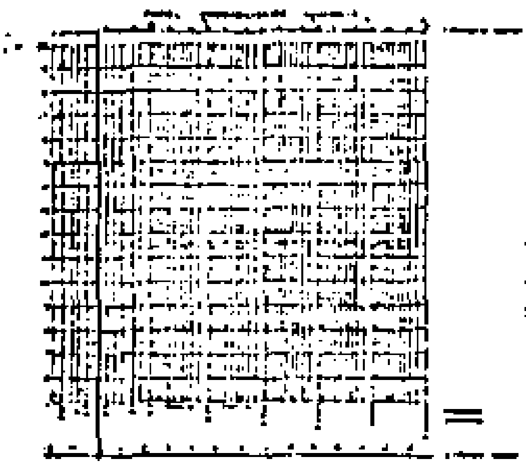

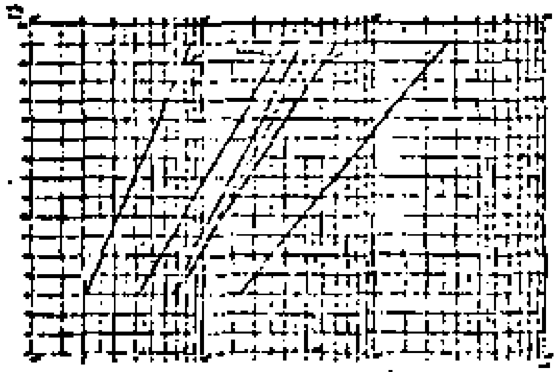

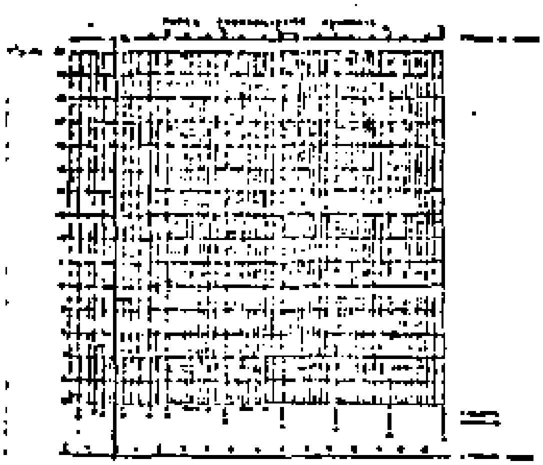

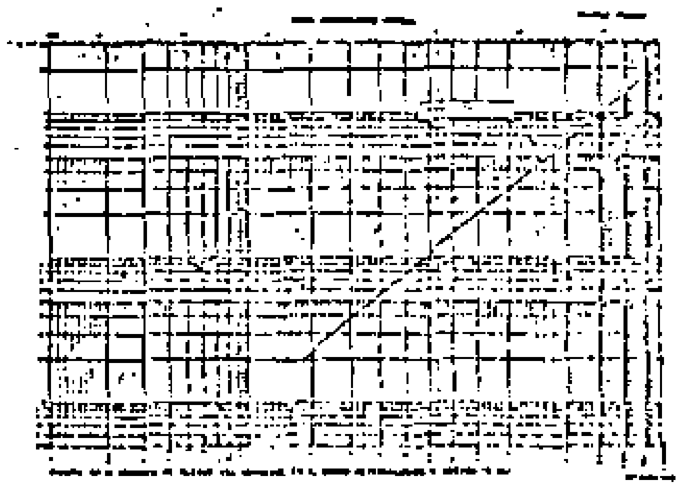

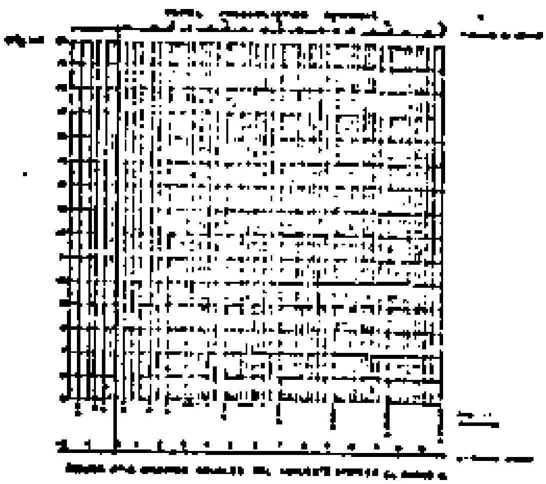

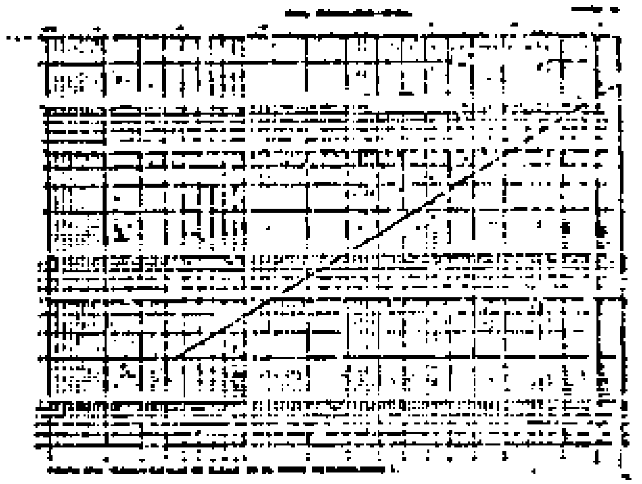

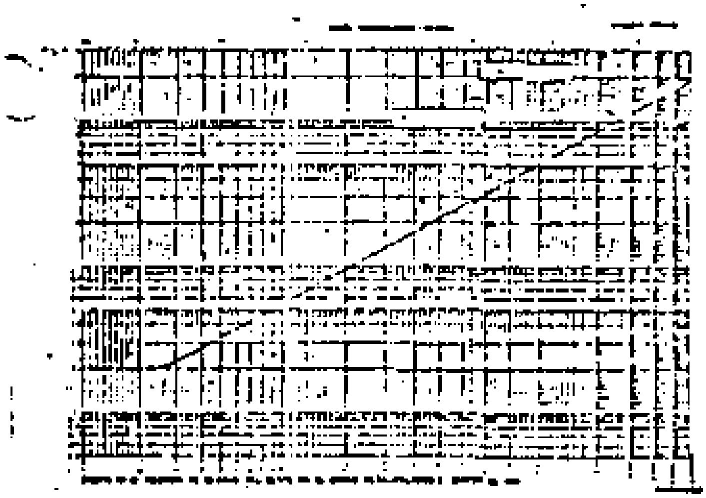

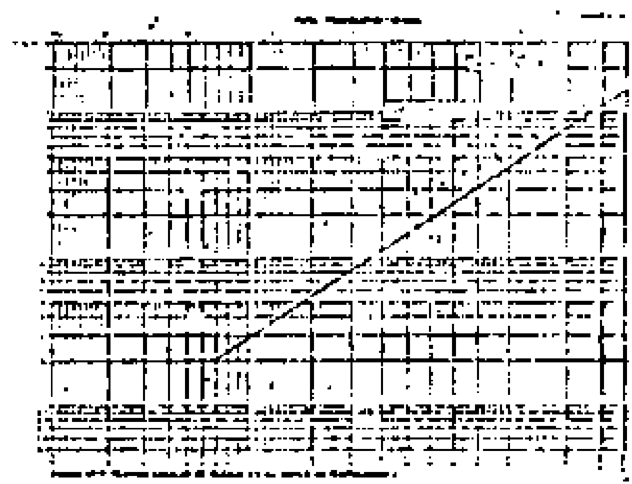

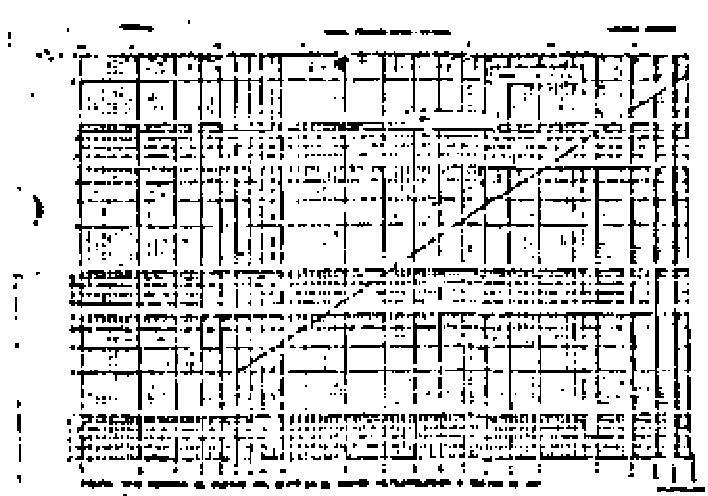

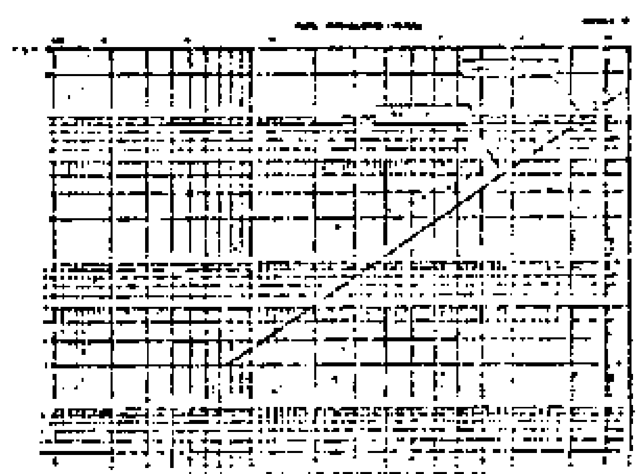

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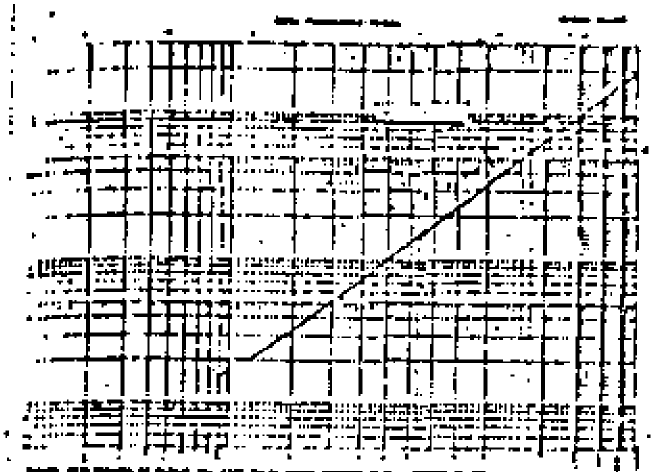

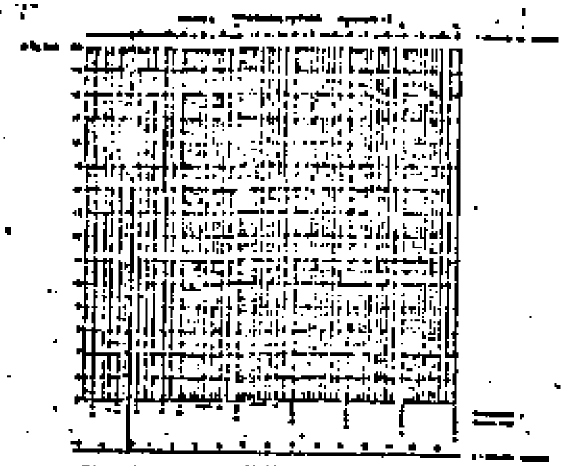

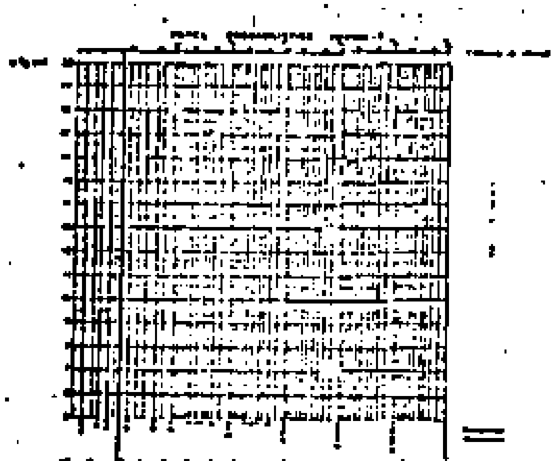

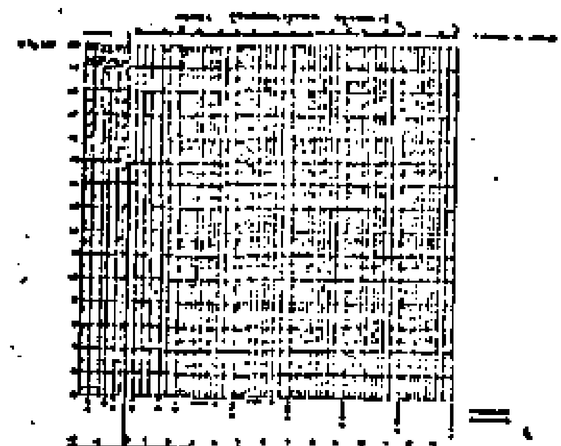

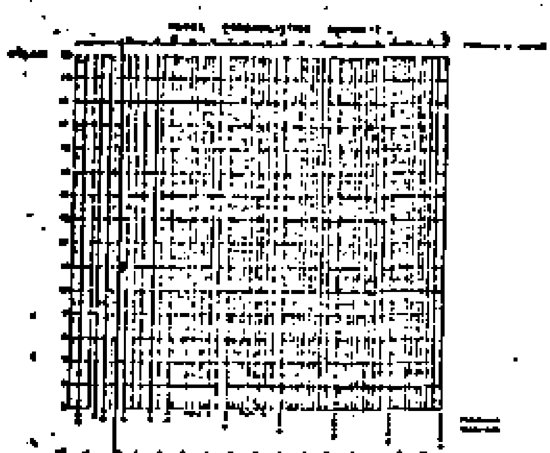

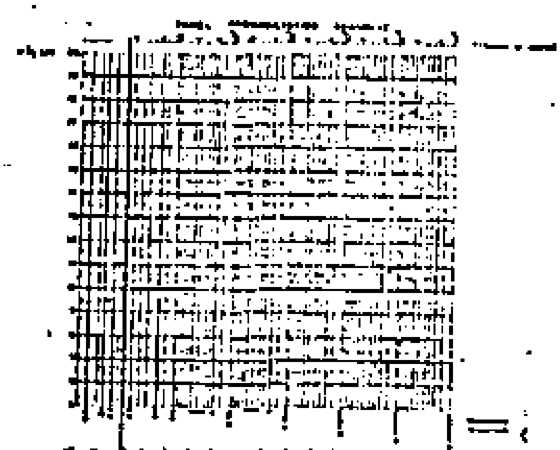

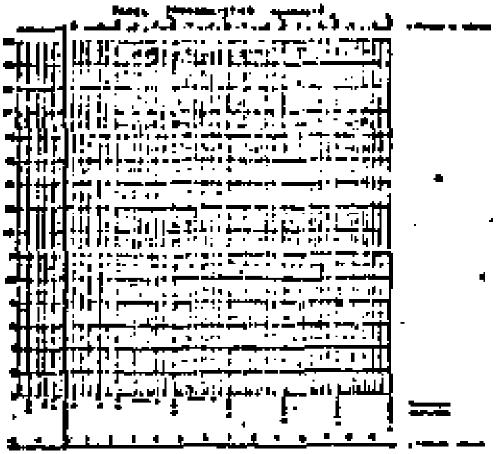

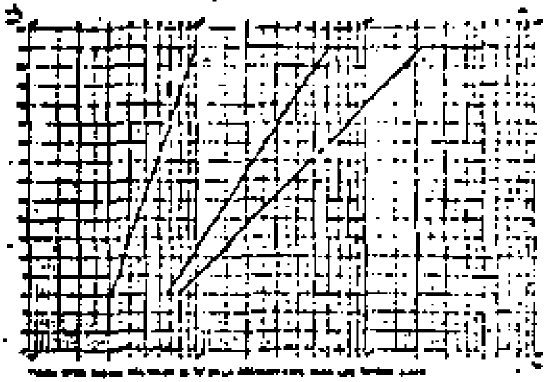

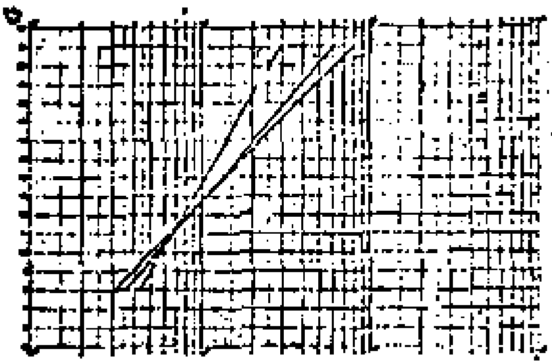

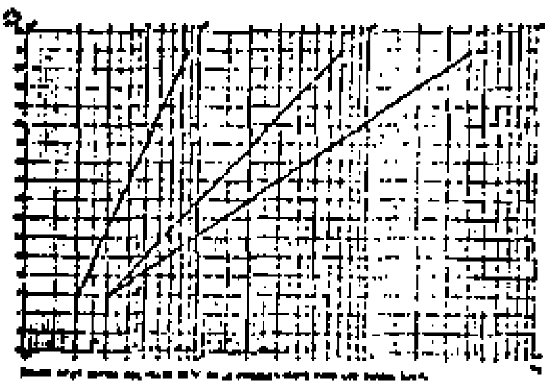

Las cartas de densidad de borrascas son un instrumento de gran utilidad a ese respecto. En las figuras 29 y 30 se han dibujado dos cartas, correspondientes al Atlántico Norte. En ellas se ha representado el número de centros de borrascas que han aparecido, durante cuarenta años, en los cuadrados de una red de 5º. Los datos han sido tomados de L. W. Pollak (1951). La segunda carta muestra las depresiones más intensas, por debajo de 990 mb. A partir de esos datos se han trazado isolíneas de centros de borrascas, mostrando áreas de concentración de esos centros y, por tanto, indicando la magnitud relativa de frecuencia e intensidad de vientos en todas direcciones. El uso de cartas de densidad de borrascas fue sugerido por R. Silvester (1956), con el fin de determinar la dirección predominante de transporte de arena en las costas.

Los centros principales de concentración de borrascas están, como puede verse, junto a Islandia, Groenlandia y la costa W. canadiense. Las isolíneas decrecen, en general, hacia el S y algo también hacia el E.

Las duraciones de los temporales están ligadas directamente a la longitud de *fetch* ocupado por el viento, y la persistencia de este último. Cuando las borrascas se mueven siguiendo ciertas trayectorias sobre los océanos, la duración de los temporales es superior hacia el final de esos caminos. La disposición de las isolíneas sugiere ya las características generales de estas trayectorias. En el Atlántico Norte los centros de baja presión se dirigen de W. a E., a veces con componente Sur; desde las áreas de concentración, donde se forman generalmente, hacia el continente europeo. La figura 30 es especialmente clara al respecto.

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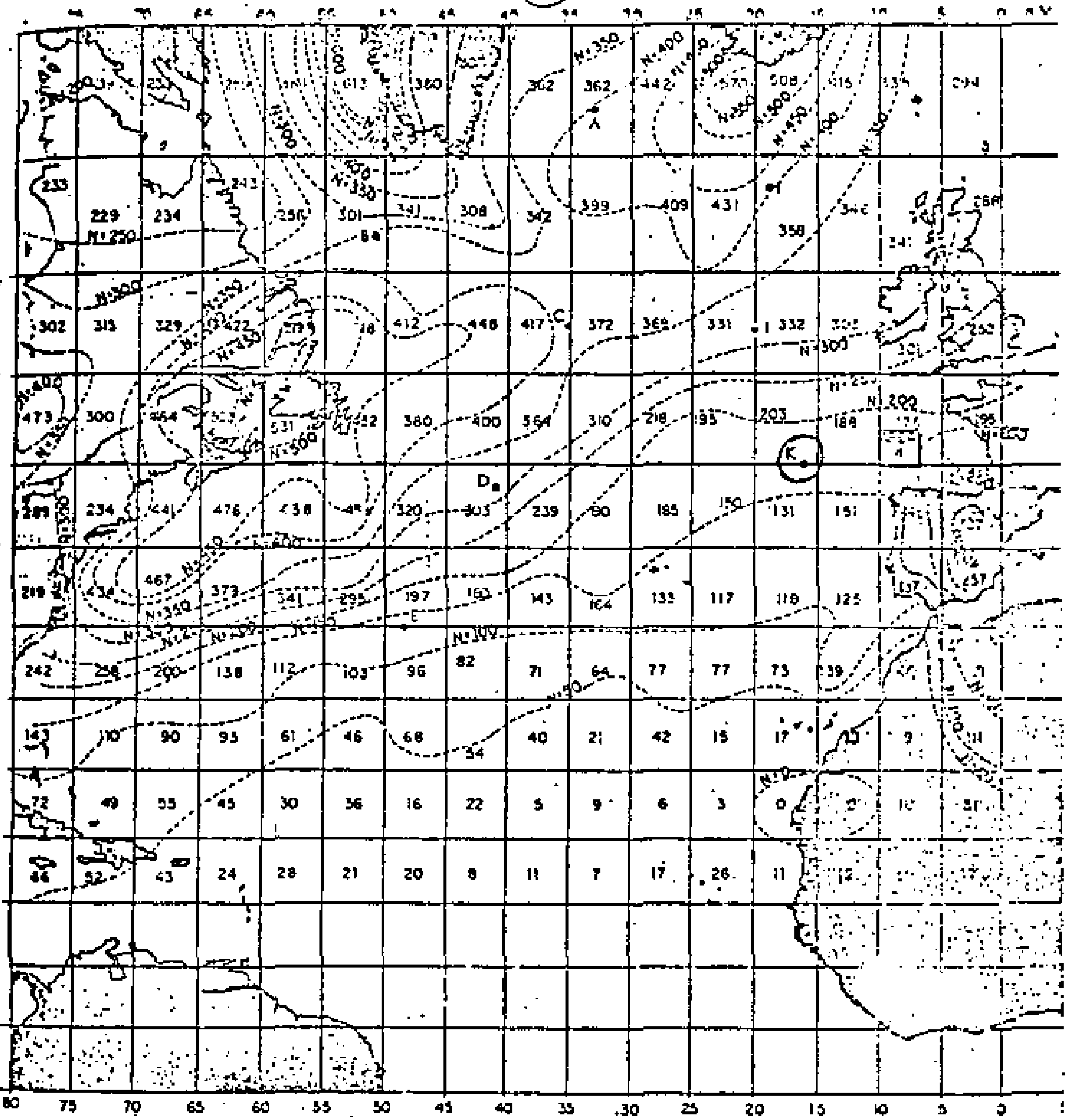


Fig. 29. — *N*, número total de centros de borrascas. Barcos meteorológicos: A, B, C, D, E, I, J, K, M. Número de centros de b: cas, 1899-1938. (Datos tomados de L. W. Pollak, 1951.)

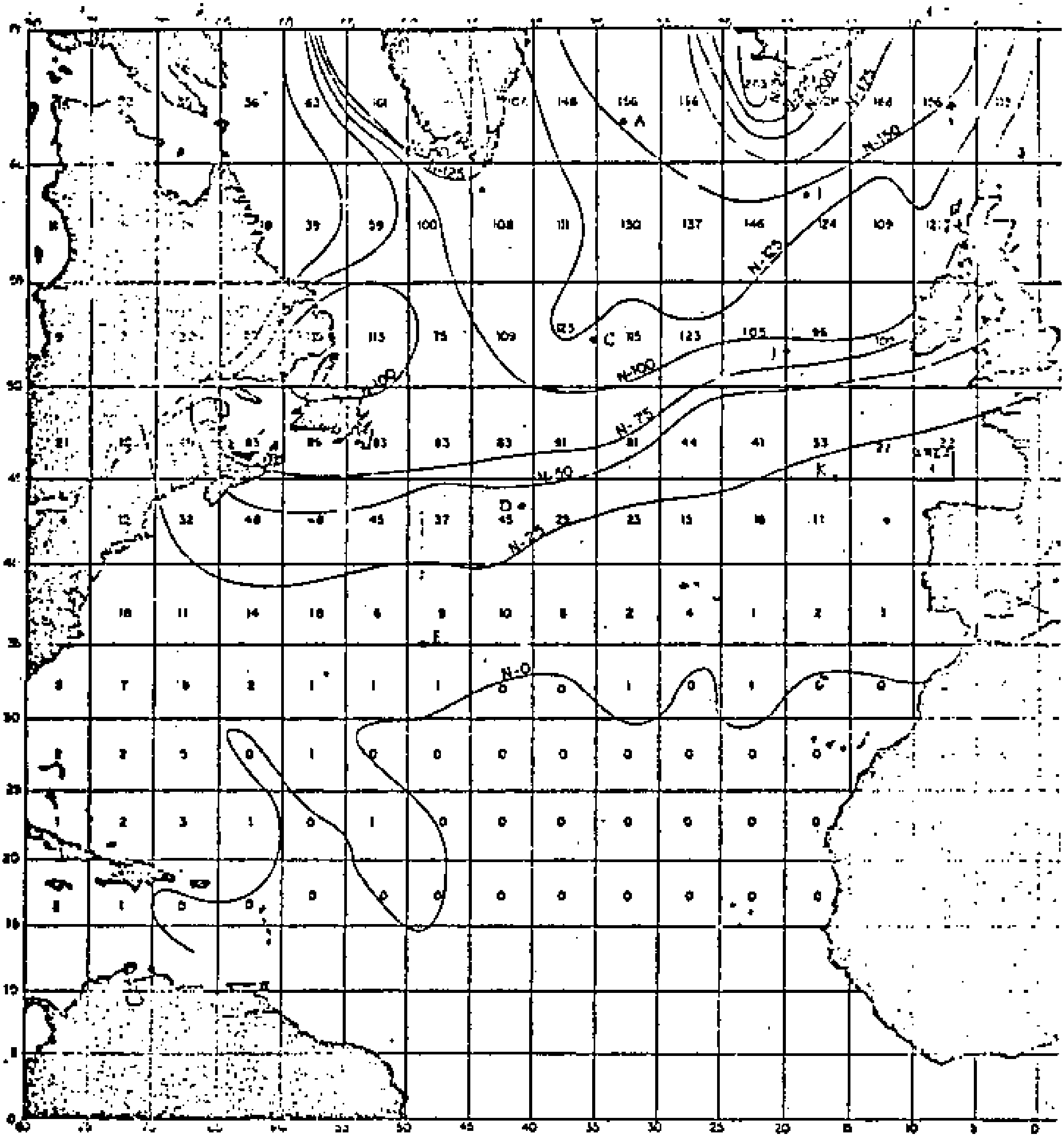


Fig. 30. — Número de centros de borrascas con presión en el centro inferior a 990 mb.

De estas consideraciones se desprende que la duración de los mayores temporales (régimen escalar) que afectan a los barcos *I*, *J*, *K*, estarán precisamente en esta secuencia creciente, obedeciendo al comportamiento general de los centros de baja presion. Esto se ve confirmado en los resultados de la figura 26. Los temporales del W. participan, obviamente, de las mismas características aún más acentuadas, como aparece en la figura 27 consecuentemente.

El comportamiento de los temporales del E. tiene explicación observando que, en la figura 29, aparece un área de concentración de borrascas sobre España. Esto hace incrementar la frecuencia de vientos de componente E. en el barco K (15,0 por 100) por encima de lo correspondiente al barco *J* (9,8 por 100) y casi tan alta como en *I* (16,5 por 100).

Se da mayor intensidad de vientos fuertes en *J* que en *K* y en *I* que en *J*, como se deduce de la figura 30 y se refleja en regímenes extremales más elevados en el orden *K*, *J*, *I*. Sin embargo, en la figura 28 las duraciones están en orden *I*, *K*, *J*. La mayor persistencia del viento del E. en *K*, junto con un *fetch* más largo confiere mayores duraciones a los temporales allí que en *J*, a igual altura de ola significante. Esta secuencia no podía preverse claramente de antemano, con sólo la información de las figuras 29 y 30, pero ésta sí indica que podían esperarse anomalías respecto al orden de los regímenes escalares del W. Siguiendo estos razonamientos, la secuencia de duraciones para las distintas direcciones, en el barco *K*, queda explicada inmediatamente a la vista de las cartas.

6.3. *La ley de variación de* $f(H_s)$.

Se ha visto en las figuras anteriores que los valores de $\frac{n}{f}$, representados en un papel semilogarítmico contra la altura de ola, describen líneas rectas. Esto supone una relación exponencial entre ambas variables.

En realidad, las rectas han sido trazadas sustituyendo, con una gran aproximación, líneas que presentan una ligera curvatura. La razón de que la relación entre *n* y H_s sea muy aproximadamente exponencial para valores altos de H_s (el factor constante *f* no altera la forma de la relación), está en dos peculiaridades de las distribuciones $F(H_s)$ y $\Phi(H_s)$. Una de ellas es que los regímenes extremales $\Phi(H_s)$ trazados a través de la nube de datos, en papel doble exponencial, se aproxima notablemente a la línea recta en todos los casos; en muchos de ellos, como puede apreciarse claramente, la línea recta da un ajuste totalmente satisfactorio.

La distribución doble exponencial (o asíntota *I*) es:

$$\Phi(H_s) = e^{-e^{-\frac{H_s - a}{b}}}$$ [6.3.1] *a*, *b*, parámetros de la distribución.

Por otro lado, se han caracterizado las distribuciones de H_s mediante la ley de Weibull, que tiene la forma:

$$F(H_s) = 1 - e^{-\frac{(H_s - A)^C}{B}}$$ [6.3.2] *A*, *B*, *C*, parámetros de la distribución.

Ambas expresiones son relacionadas mediante la fórmula (2.1):

$$\Phi(H_s) = [F(H_s)]^n$$

Para valores elevados de H_s, $F(H_s)$ es cercano a la unidad y la expresión anterior converge en:

$$\Phi(H_s) = e^{-n[1 - F(H_s)]} \qquad (6.3.3)$$

Sustituyendo las distribuciones $F(H_s)$ y $\Phi(H_s)$:

$$\ln(n) = \frac{(H_s - A)^C}{B} - \frac{H_s - a}{b} \qquad (6.3.4)$$

Ahora bien, en todas las distribuciones de Weibull ajustadas, el parámetro *C* resulta ser muy cercano a la unidad. Aparece la siguiente relación aproximada:

$$\ln(n) \simeq \frac{b - B}{bB} H_s + \frac{aB - Ab}{bB} \qquad (6.3.5)$$

El utilizar esta aproximación está perfectamente justificada dentro del orden de precisión en que los datos disponibles nos permiten movernos. Por un lado, el ajuste de las distribuciones de Weibull se hace "a sentimento", por lo cual el valor exacto de sus parametros es discutible, siendo solamente constante la aproximación de C a la unidad en todos los casos. Por otro, la precisión que puede esperarse de las observaciones extremales visuales no justifica el usar un método matemático de ajuste de la nube de puntos para caracterizar $\Phi(H_s)$.

En el apartado siguiente se discute la influencia en los resultados de utilizar una expresión aproximada para n.

De la relación aproximadamente exponencial hallada se deriva una interpretación interesante: la rápida disminución de la duración de los temporales al aumentar H_s implica la existencia de un cierto límite físico al crecimiento de la altura de ola significante.

6.4. *Sensibilidad de las predicciones extremales a los cambios del parámetro* n.

El período de retorno de H_s viene dado por la expresión (2.5) siempre que valga más de unos diez años. Introduciendo en ella la distribución de Weibull (6.3.2) obtenemos:

$$n\,T(H_s) = e^{\frac{(H_s - A)^C}{B}}$$

Recordando que C es muy cercano a la unidad, se puede escribir:

$$H_s \simeq A + B \ln [n \cdot T(H_s)]$$

Utilizando dos valores diferentes de n (n_1, n_2) se obtienen, para un mismo período de retorno T, dos valores de H_s (H_1, H_2):

$$H_1 \simeq A + B \ln (n_1 \cdot T)$$

$$H_2 \simeq A + B \ln (n_2 \cdot T)$$

Restando:

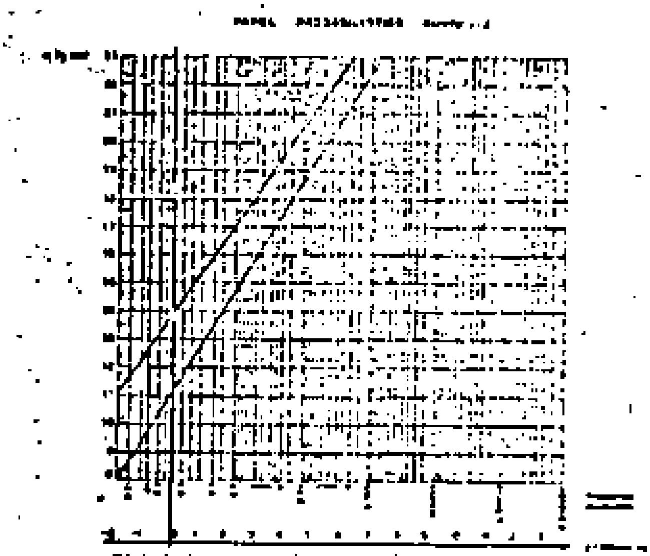
$$H_1 - H_2 \simeq B \ln \frac{n_1}{n_2}$$

El error en el valor de H_s que se predice cuando, en la ecuación (2.1), se usa 1,65 veces n en lugar de n es, para el mismo período de retorno, igual a $\frac{B}{2}$. Si se toma tres veces n, el error es casi igual a B. B es el valor de H_s con probabilidad 0.63. Los valores de B en los regímenes $F(H_s)$ obtenidos para el barco K son 1,65 m, 1,25 m, 1,70 m, 1,15 m y 1.24 m para los regímenes escalar, E., W., N. y S., respectivamente.

Se concluye que la sensibilidad de las predicciones a cambios pequeños en n es baja. Esta conclusión es de importancia [illegible] siva en la aplicabilidad del método, tanto autorizar el uso de expresiones aproximadas para n como para utilizar interpolaciones o extrapolaciones de n entre distintos puntos o direcciones.

6.5. *Cálculo de las curvas de diseño.*

Las curvas de diseño para agua profunda en San Ciprián han sido calculadas a partir las distribuciones de H_s pertenecientes al área (figura 29), mediante la ecuación (5.1). Las duraciones de temporales han sido estimadas partir de las halladas para el barco K. En la figura 31 se muestran las curvas correspondientes al régimen escalar y al sector NE. (45°).



Las duraciones correspondientes al régimen escalar han sido tomadas ligeramente mayores que en el barco K, y para el NE., algo inferiores en consecuencia, con la posición relativa ambos lugares.

6.6. *Limitaciones del método.*

Es conveniente pasar revista brevemente las limitaciones propias de la aplicación de sistemas de cálculo expuestos al conjunto datos utilizados.

Las fuentes de incertidumbre están basa[illegible] miento en la calidad y cantidad de los datos con que se trabaja.

El número de observaciones con que se confeccionan las distribuciones extremales es un primer factor a tener en cuenta, cuando se obtienen las duraciones de temporales por el método indirecto utilizado aquí. Cuando el número de observaciones cambia, existe una variación sistemática de la pendiente de la recta ajustada. Números progresivamente mayores producen pendientes en disminución, lo que implica predicciones más conservadoras a mayor escasez de datos. Esto no deja de ser tranquilizador, pero da valor al esfuerzo empleado en la obtención de una mayor cantidad de datos. W. Potter (1949) verificó este hecho utilizando una enorme cantidad de registros de lluvias en U.S.A. A partir de un cierto número, la variación de la pendiente se hace despreciable.

El número de años utilizado en este trabajo, diecinueve, es bueno dentro de los estándares visuales (ver, por ejemplo, H. Thom (1970), o P. Suárez Bores (1973)). Sin embargo, debe ser mejorado a medida que haya mayor número de datos disponibles.

El segundo punto a observar se refiere al uso de la distribución de Weibull. Al contrario de lo que afirmaba recientemente B. Pedersen (1971), las extrapolaciones producidas por la ley de Weibull son altamente sensibles a la variación del parámetro A, ajustado a sentimiento. Por esta razón, es sumamente importante el establecer un criterio que homogeinice la calidad de los ajustes. El criterio adoptado en este trabajo ha sido ajustar el rango de H_s medias y bajas (con un límite inferior de 1 m, aproximadamente), y dar menos peso a las más escasas observaciones de los valores altos de H_s.

La bondad de este procedimiento, como se comentó en la sección 6.1, está bien comprobada para regímenes escalares (en el E. del Atlántico Norte al menos). Sin embargo, no se ha hecho hasta el momento una comprobación sistemática semejante, con datos instrumentales, para las distribuciones direccionales. La calidad de los resultados dependerá de la adecuación de aquella hipótesis a esos regímenes direccionales. Podemos argumentar que, de momento, no disponemos de una hipótesis mejor.

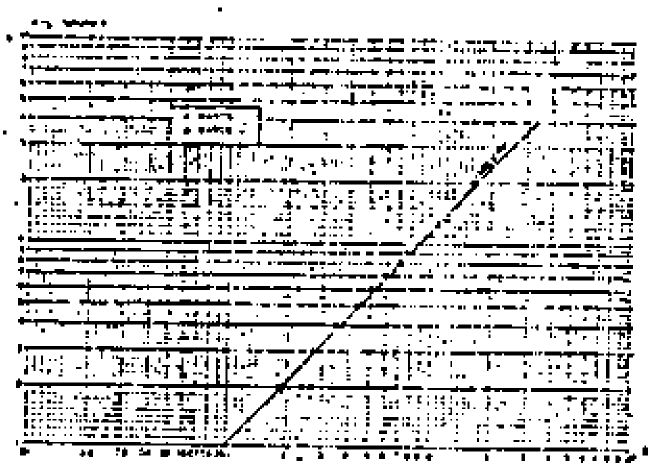
La calidad de las observaciones visuales de altura de ola significante constituye otro tema de discusión. El tema ha sido abordado en varias publicaciones. N. Hogben (1969), que ha trabajado intensamente en la verificación de la fiabilidad de las estimas visuales de H_v, llega a conclusiones francamente optimistas al respecto, cuando los observadores han sido convenientemente entrenados. Partiendo de la hipótesis de una homogeneidad de resultados cuando se promedian un gran número de observaciones visuales, N. Nordestrom (1969) ha buscado una correlación sistematica entre la H_v estimada visualmente y la $H_{1/3}$ real. El resultado obtenido por este autor es la relación:

$$H_{1/3} = 1{,}68\, H_v^{0{,}73}$$

Esta relación no ha sido confirmada por la comprobación que hemos hecho en los barcos *I* y *J*. Los datos instrumentales publicados por L. Draper and M. Whitaker (1965) y por L. Draper and E. Squire (1967) han sido representados, junto con la serie de datos visuales compilada por H. Walden (1964), en la figura 32. La relación resultante resulta ser:

$$H_{1/3} = 1{,}02\, H_v^{1{,}03}$$

Esta ecuación debe preferirse para su uso en el barco *K*, por razones de similitud (es también un barco meteorológico y usa sistemas idénticos) y proximidad geográfica.



En cuanto a la precisión de las observaciones visuales de valores extraordinarios de H_v, se puede aducir una comprobación satisfactoria de N. Hogben (1969). En este punto, como en los anteriores, los proyectistas no podemos adoptar otro punto de vista que el pragmático: mientras no haya otros datos, aquellos son "los mejores" y tendremos que diseñar con ellos.

Será sumamente interesante el comprobar, con medidas hechas sobre la curva real de estados del mar, la relación "aproximadamente exponencial" entre n y H_s para valores altos de esta última. Es un tema que piensa abordarse en un futuro próximo por su importancia en el proceso de cálculo.

El conocimiento de los factores que aportan incertidumbre a la aplicación de estos métodos, permite ir mejorando el uso de estos últimos a medida que haya mejores datos disponibles. Por el momento, algunas comparaciones hechas con los resultados obtenidos en San Ciprián son satisfactorias:

En la dirección NE., un estudio realizado para San Ciprián por L. Draper (Institute of Oceanographic Science, U.K.) a partir del régimen extremal de vientos (elaborado por el British Meteorological Office) proporciona los resultados siguientes, para un período de retorno de cincuenta años: Aplicando el método de previsión de Bretschneider (revisado en 1970), H_s = 11,5 m; aplicando el método del I.O.S., 13,5 m. El resultado obtenido en este trabajo ha sido (fig. 31, probabilidad 0,37), H_s = 11,1 m.

En el estudio efectuado por L. Draper, a los inconvenientes que plantea la calidad de los datos se une la problemática delimitación de *fetches* y asignación de duración a los vientos. Resulta, pues, confortante, ver cómo los resultados son comparables a pesar de todas las fuentes de inexactitud que han sido señaladas.

7. Conclusiones.

7.1. La distribución extremal de H_s puede ser calculada a partir de la distribución de H_s, para cualquier sector direccional, cuando se conoce la duración media de los temporales y la frecuencia de presentación del oleaje dominante en ese sector.

7.2. Se demuestra que esas mismas condiciones sirven para caracterizar otros modelos extremales de oleaje establecidos anteriormente.

7.3. Las duraciones medias de temporales, $t(H_s)$, pueden ser estimadas directamente, a partir de la curva de estados del mar, o indirectamente cuando se conoce la distribución extremal. Conocidos los valores de $t(H_s)$ en varios puntos del océano o en varios sectores direccionales de un mismo lugar, las duraciones correspondientes a lugares o sectores direccionales intermedios (o adyacentes) pueden estimarse por interpolación (o extrapolación).

7.4. Para juzgar la conveniencia de interpolaciones o extrapolaciones, es necesario conocer las características generales del desarrollo de los temporales en el mar. Las cartas de densidad de borrascas son un instrumento útil a este respecto.

7.5. La relación entre H_s y $n = \frac{8760}{t(H_s)}$ es, aproximadamente, exponencial, para valores altos de H_s, en las direcciones y barcos meteorológicos del E. del Atlántico Norte, que han sido utilizados en este trabajo. Esta relación, extrapolada, constituye un límite físico al crecimiento de la altura de ola significante (la duración disminuye rápidamente al crecer H_s).

7.6. La sensibilidad de las distribuciones extremales calculadas, a cambios pequeños en el parámetro n, es muy baja. Este hecho da valor práctico a las interpolaciones y extrapolaciones.

7.7. Se recomienda el uso de las curvas de diseño $\Phi_x(H_s)$ para efectuar los cálculos económicos que proporcionan la ola de cálculo óptima.

INTERVENCIONES EN EL TRABAJO

La oportunidad, nada corriente, de haber dedicado un tiempo y esfuerzo en desarrollar un método original de cálculo del oleaje extremal, aplicable en San Ciprián, se debe al punto de vista progresivo de Victoriano Fernández Dupuy y William Brennan, ingenieros responsables del proyecto por parte de Intecsa. La publicación de los resultados ha sido posible igualmente gracias a su entusiasmo profesional. Las discusiones de William sobre el contenido del artículo han sido también de gran valor práctico.

El profesor R. Dorrestein (KNMI, Países Bajos) ha revisado el trabajo original, sugiriendo comentarios interesantes. Asimismo, las conversaciones mantenidas, durante el período de búsqueda de datos, con N. Hogben (Nat. Phys. Lab., U.K.), L. Draper (Inst. Ocean. Sc. Wormley, U.K.) y M. Tann (Inst. Ocean. Sc., Taunton, U.K.), proporcionaron una información valiosa sobre varios aspectos del tema.

REVISTA DE OBRAS PUBLICAS. Mayo 1979. Pág. 401 a 420.

Función de distribución extremal en fenómenos de tipo meteorológico (*)

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Se discute la metodología en uso para la determinación de función de distribución extremal en fenómenos de tipo meteorológico. Se analizan las deficiencias y se proponen procedimientos alternativos desde el punto de vista de su utilidad práctica en Ingeniería.

1. INTRODUCCION

El análisis estadístico extremal de variables de tipo meteorológico (o geofísicas) como la velocidad del viento, la precipitación, el caudal fluvial o la altura de ola, viene representando cada vez más uno de los elementos básicos en los proyectos de obras civiles que se ven afectados por aquellas variables.

El primer texto dedicado a la estadística de valores extremos fue debido a E. Gumbel (1958). Aunque anteriormente se habían publicado algunos análisis extremales, puede decirse que la aparición de este libro marcó, al menos en los países tecnológicamente avanzados, el comienzo del uso generalizado del análisis extremal en la práctica ingenieril. El texto de Gumbel sigue siendo hoy la referencia básica del análisis extremal aplicado.

Sin embargo, no puede decirse que estas técnicas hayan sido aún desarrolladas de forma satisfactoria en todos sus aspectos. A pesar del tiempo transcurrido y de la enorme importancia práctica de este tema, una revisión de los resultados a que han llegado los numerosos autores que han realizado aplicaciones de la teoría extremal descubren un extenso desacuerdo. *En lugar de haberse dado un proceso de homogeneización progresiva de los criterios, como suponía el propio Gumbel en su libro, en realidad el tiempo ha ido acentuando las divergencias:* Cada vez se proponen más funciones distintas para la distribución extremal de una misma variable y hasta dentro de un mismo clima, funciones que al ser extrapoladas para su uso práctico divergen ampliamente implicando impactos sumamente diferentes en los proyectos.

Al propio tiempo son varios los autores que, al tratar este tema, explícitamente aseguran no existir un criterio con fundamento que permita discriminar la conveniencia del empleo de unos u otros métodos en el proceso de cálculo. Es decir, *podemos hablar de una situación de ambigüedad reconocida y grave.* La gravedad de esta situación la experimentan fundamentalmente los ingenieros que deben tomar la decisión de elegir por una de las posibilidades existentes, sabiendo que estas posibilidades pueden representar serias variaciones en el precio y la seguridad de las obras que proyectan, y sabiendo (o lo que es peor, sin saber) que no disponen de un criterio reconocido de elección.

En un artículo anterior (14) se trató de las fuentes de ambigüedad existentes en los criterios habituales de ajuste de función de distribución a partir de muestreos aleatorios. En el presente artículo se van a aplicar los resultados de aquel análisis a la elección de función de distribución extremal, con objeto de llegar a criterios de facilidad práctica desde un punto de vista ingenieril.

(*) Se admiten comentarios sobre el presente artículo, que pueden remitirse a la Redacción de esta Revista, hasta el 31 de julio de 1979.

2. ESTADO ACTUAL DE LA METODOLOGIA

El sistema habitual de determinar qué tipo de función de distribución extremal corresponde a una determinada variable, se basa en obtener una o más muestras extremales de la variable y comprobar las bondades de los ajustes conseguidos con algunas funciones de distribución. *Respecto a qué funciones conviene probar, son numerosos los autores que han utilizado con exclusividad las denominadas "Distribuciones Extremales Asintóticas" (o Asíntotas I, II y III).* Estas tres distribuciones son el producto final de la teoría de distribuciones asintóticas de valores extremos, popularizada en el tratado de Gumbel, y los autores aludidos han venido suponiendo, explícita o implícitamente, que su fundamento teórico como distribuciones extremales hace de las Asíntotas los candidatos óptimos para la solución de cada caso (ver, por ejemplo, 21). Sin embargo, como se indicó anteriormente, se ha llegado a una gran heterogeneidad de resultados en su uso práctico. A título de ejemplo se citan algunos casos significativos:

Precipitación caída en un intervalo de tiempo.

S. Nag y N. Dutta (1951) usaron la Asíntota-II en un estudio de precipitaciones máximas diarias, en la estación del monzón, dentro de la cuenca del río Saraker. Sin embargo, E. Gumbel (1958) probó que la Asíntota-I ajustaba mejor la misma serie de datos. A. Jenkinson (1955), por otra parte, consiguió los mejores ajustes con la Asíntota-II para la máxima precipitación en períodos de una hora, un día y cuatro días en Marsella, Nápoles y Trípoli. Puede señalarse que las Asíntotas-I y II producen extrapolaciones que divergen muy ampliamente, la segunda de ellas alcanzando valores considerablemente más elevados. En la figura 2.2.2 de la referencia (12) se puede ver cómo la Asíntota-I ajusta muy satisfactoriamente a las precipitaciones diarias máximas de Madrid, y en la figura 5.3.2 de la misma referencia se observa que una Asíntota-I también puede ajustar bien a las máximas precipitaciones mensuales en Los Llanos (Cuenca).

N. Barricelli (1943) utilizó la Asíntota-I para el ajuste de las *máximas presiones atmosféricas* anuales en Bergen, Noruega. Pero para la misma variable medida en Marsella, A. Jenkinson (1955) encontró más ajustada la Asíntota-II.

A. Court (1953) utilizó con éxito la Asíntota-I al estudiar las *velocidades máximas del viento* (medias en cinco minutos) en 25 estaciones de Estados Unidos. J. Bell (1961) usó también la Asíntota-I para la velocidad máxima del viento en Hong-Kong, y H. Willingham (1964) para máximas rachas del viento en Australia. A. Davenport (1960) propuso emplear esa función de distribución como método estándar de análisis extremal del viento en las normas canadienses. Pero H. Thom, en una serie de estudios de velocidad máxima del viento en distintas partes del globo (1954, 1960, 1963, 1968, 1969, 1973) ha empleado *siempre* la Asíntota-II. A. Jenkinson (1955) empleó la Asíntota-III para la máxima velocidad media horaria del viento en Trieste. Por otra parte, en la figura 5.3.9 de la referencia (12) puede verse que una Asíntota-I ajustaría bien a las máximas velocidades medias diarias en Valladolid.

E. Gumbel (1940) utilizó por primera vez la Asíntota-I para el análisis extremal de *avenidas fluviales*. Después, la misma función de distribución ha sido empleada en una multitud de casos referentes a ríos americanos por W. Potter (1949), S. Rantz y H. Riggs (1954), R. Carter (1951), y E. Gumbel (1958) entre otros. A. Benham (1950) también la aplicó con resultados satisfactorios a ríos de Nueva Zelanda, y Shuh (1962) a China. Sin embargo, J. Bernier (1956) encontró que la Asíntota-II proporcionaba mejores ajustes para las máximas avenidas diarias del Rhin en Rheinfelden, del Colorado en Black Canyon y del Durance en Archidiacre. A. Jenkinson (1955) empleó también la Asíntota-II para Little River, pero prefirió una Asíntota-III para el río Connecticut.

M. St. Denis (1959) propuso utilizar la Asíntota-I al análisis extremal de la *altura de ola significante*. P. Suárez Bores (1974) empleó esta distribución en el ajuste de varias muestras extremales de altura de ola significante estimadas para puntos de la costa española por medio de estudios de previsión a partir de cartas barométricas y regímenes de viento. H. Thom (1970) prefirió, en cambio, la Asíntota-II para ajustar las observaciones máximas anuales de barcos meteorológicos situados en el Atlántico y Pacífico. El autor del presente trabajo (E. Copeiro, 1976) obtuvo, en algunos barcos meteorológicos del Atlántico Norte, muestras extremales más largas que las utilizadas por Thom y consiguió ajustes satisfactorios con la Asíntota-I. Por otra parte, en un artículo posterior al reseñado arriba M. St. Denis (1973) se declaró partidario de la Asíntota-III por estimar que, según su experiencia, la Asíntota-I da predicciones excesivamente altas.

Como puede verse, se ha llegado a todo tipo de resultados para una misma variable. Es verdad que, entre los usuarios de las Asíntotas, han sido sólo una exigua minoría los que han intentado justificar que las condiciones en que fue deducida la Asíntota escogida como distribución de extremos se satisfacen en el caso concreto en que se la está empleando. Estos intentos de justificación, por lo demás, no han sido acompañados por el éxito debido (como se verá más adelante) a una interpretación errónea de la aplicabilidad de la ecuación extremal a las variables geofísicas.

La generalidad de autores que han empleado las Asíntotas se han limitado a elegir entre ellas de acuerdo con el criterio estrictamente empírico de su mejor o peor ajuste a la muestra extremal disponible. *Esto ha terminado por colocar a las tres Asíntotas en el campo del puro empirismo, donde se las alinea en igualdad de condiciones con otras funciones de distribución a la hora de probar cuál de mejor ajuste a las muestras.* Puede afirmarse que este punto de vista es el dominante en la actualidad. Como ejemplo puede citarse un comentario de J. Bernier, que en 1956 (6) había publicado un artículo sobre la aplicación de la Asíntota-II a avenidas fluviales máximas y en el año 1933 (7) pasa a ser de la opinión que es preferible hacer cada elección en función del ajuste conseguido, para lo cual lo más conveniente es disponer de un amplio arsenal de funciones de distribución que probar. Concluye que no debe concederse un papel especial a las Asíntotas, ya que otras funciones, como por ejemplo la Log-Normal, dan en ocasiones mejores ajustes.

En esta línea, puede mencionarse (5) el estudio comparativo realizado por el Grupo de Trabajo Federal sobre métodos de frecuencias de avenidas (Water Resources Council, U.S.A.), en que se compararon las distribuciones Asíntota-I, Asíntota-II, Log-Normal, Log-Pearson-III, Gamma-II y Hazen, con ayuda de 10 muestras extremales. En el campo de estudios extremales de oleaje, dos ejemplos recientes de la misma tendencia pueden verse en las referencias (24) (Asíntota-I y Weibull) y (9) (Normal y Weibull).

En estas selecciones de distribución extremal en base a los ajustes conseguidos con muestras de valores extremos, los distintos autores han optado por uno de los dos criterios siguientes:

1. *Selección a partir de varias muestras extremales de la variable en cuestión.*

Se escoge aquella función de distribución que mejores ajustes dé al conjunto de muestras, y esa función se emplea en todos los casos correspondientes a aquella variable. Obviamente, este punto de vista parte implícitamente de suponer que cada variable específica posee un mismo tipo de distribución extremal en cualquier localización.

2. *Selección para cada muestra extremal.*

Este criterio, estrictamente casuístico, parte de suponer que una misma variable puede en principio adoptar tipos totalmente diferentes de distribución extremal en localizaciones distintas. Se escoge en cada caso aquella función que da un mejor ajuste a la muestra extremal disponible.

A continuación se discute la conveniencia de estos criterios.

3. DISCUSION DE LOS CRITERIOS EN USO

3.1. *El último de los dos criterios vistos (la elección casuística)* es indudablemente cómodo, puesto que se presta muy bien a reducir el análisis extremal de cada caso a una rutina de tanteos relativamente fácil de mecanizar utilizando ordenadores y sin necesidad de acudir a consideraciones generales. Sin embargo, un somero análisis de los resultados a que puede llegarse con esta técnica muestra que su uso es a la vez insatisfactorio desde un punto de vista teórico y altamente peligroso desde la perspectiva ingenieril.

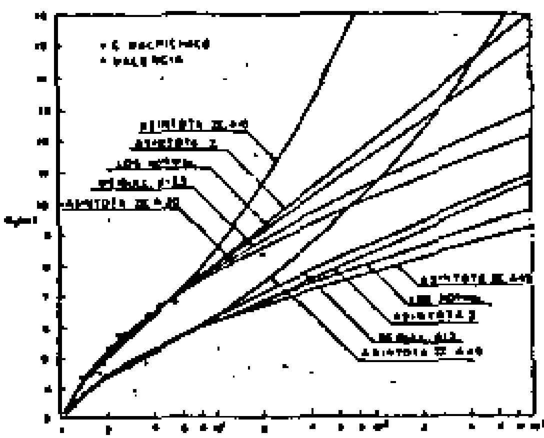
Aun si fueran aceptados como válidos los criterios habituales de bondad de ajuste que tratan uniformemente la totalidad del recorrido muestral (lo que es incorrecto, según se discute en (12 y 14)), estos criterios no son capaces de proporcionar, a partir de una sola muestra, una elección de FDO satisfactoria en la práctica cuando se pretende extrapolar aquella FDO. La razón está en que, para los tamaños muestrales que son corrientes en la realidad, hay normalmente varios tipos distintos de FDO que son capaces de proporcionar ajustes de bondades muy similares. Esas funciones toman valores muy próximos a lo largo del recorrido de los valores muestrales, pero al ser extrapoladas pueden divergir ampliamente produciendo resultados muy diferentes al nivel de los periodos de retorno requeridos usualmente para el uso práctico. En estos casos (que son la norma más bien que la excepción) no es ingenierilmente permisible el realizar la elección entre funciones con el solo criterio de una pequeña diferencia entre las bondades calculadas de los ajustes. Esas diferencias menores son perfectamente atribuibles a la inevitable variabilidad entre muestras distintas de una misma población.

Para mostrar un caso real se han tomado dos muestras extremales de altura de ola significante, obtenidas mediante métodos de previsión para Cabo Machichaco (Vizcaya) y Valencia (tomadas de P. Suárez Bores, 1976). En las figuras 2 a 6 se muestra el resultado de ajustar visualmente ambas muestras, representadas según metodología de E. Gumbel (1958), con funciones de distribución Asíntota-I, Asíntota-II, Asíntota-III, Weibull y Log-Normal. Todos los ajustes son aceptables según los contrastes estadísticos habituales, si bien existen pequeñas diferencias entre ellos. En la figura 1 se comparan las extrapolaciones correspondientes a las distintas distribuciones. Las diferencias, al nivel de los periodos de retorno usuales (por ejemplo, T = 500 años; Riesgo de superación en 50 años = = 10 por 100) son muy grandes, totalmente fuera de proporción con las diferencias entre los ajustes. Es tan importante la repercusión que supone, en proyectos de obras marítimas, la diferencia entre estas estimas, que un ingeniero que tuviese que hacer uso práctico de estos resultados no podría permitirse el hacer la elección entre las distribuciones con el solo criterio de las pequeñas diferencias en la bondad de aquellos ajustes. Para realizar en este caso una elección razonable, sería necesario acudir a otro tipo de criterio.

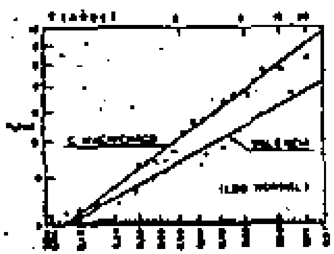
Cuando se utiliza un criterio racional para los ajustes, que tenga en cuenta la fiabilidad variable de la estima muestral a lo largo de su rango de valores (12 y 14), se desechan dos tramos en los extremos del recorrido muestral. El recorrido muestral restante (*útil o efectivo*) es más corto y resulta, por tanto, aún más susceptible de ser estrechamente ajustado por un amplio número de FDO diferentes.

Puede concluirse que, al menos en el campo

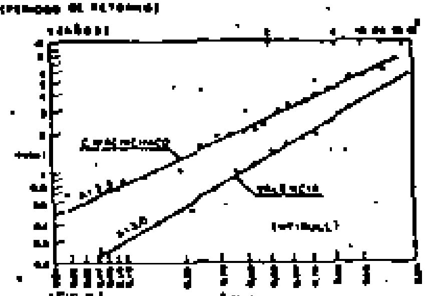
FUNCION DE DISTRIBUCION EXTREMAL EN FENOMENOS DE TIPO METEOROLOGICO



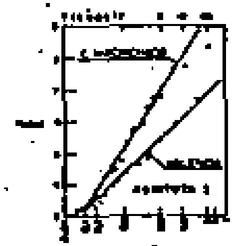
(FIG. 1)



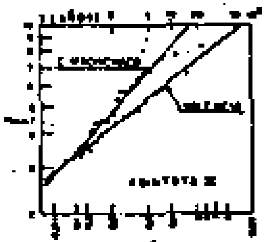
(FIG. 4)



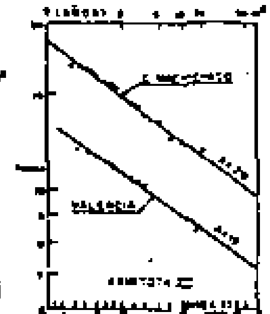
(FIG. 6)



(FIG. 5)



(FIG. 3)



(FIG. 2)

de los tamaños muestrales habituales y a efectos de extrapolar resultados a períodos de retorno también habituales (superiores al menos en un orden de magnitud al tamaño muestral, según es corriente), *la mejor bondad de ajuste no proporciona un criterio fiable de elección de FDDE cuando se analiza una sola muestra aisladamente.*

3.2. *El otro de los criterios mencionados en el apartado anterior (la selección a partir de varias muestras extremales de la variable en cuestión),* tampoco ha producido hasta el momento elecciones satisfactorias a pesar de que, como se verá a continuación, está en una posición teórica favorable.

Un caso significativo que es interesante comentar, a este respecto, es el estudio comparativo citado anteriormente (5) en que se comparó el comportamiento de seis funciones de distribución con 10 muestras extremales correspondientes a avenidas fluviales. El Grupo de Trabajo Federal escogió la distribución Log-Pearson-III por ser la que mejores bondades de ajuste dio en el conjunto de muestras. Sin embargo, algunos años más tarde (8) otros autores compararon aquella función con la Pearson-III, con ayuda de otras 27 muestras extremales pertenecientes a 13 países distintos de cuatro continentes, y concluyeron que la distribución Pearson-III proporciona mejores ajustes y debe, por tanto, ser preferida a la otra. Si ambas distribuciones tomaran valores muy próximos en las extrapolaciones, el problema planteado sería sólo aparente puesto que desde un punto de vista práctico el uso de una u otra función daría *resultados muy similares* y, desde el punto de vista teórico, no debe olvidarse que este tipo de búsqueda de *FDD* sólo pretende llegar a una aproximación empírica de la distribución real. Sin embargo, de hecho las extrapolaciones producidas por las distribuciones Pearson-III y Log-Pearson-III divergen ampliamente, originando repercusiones importantes en su uso práctico dentro de los períodos de retorno usuales. Por tanto, el problema de selección entre ellas resulta crucial.

Se podrían citar otros casos análogos a este último. No parece que la metodología en uso actualmente está en camino de proporcionar selecciones fiables. Por el contrario, a medida que se incorporan nuevas funciones y nuevas muestras a los estudios comparativos, la corona de favorita va pasando de una distribución a otra de una forma realmente caótica, desgraciadamente alejada del deseable proceso convergente que fuera centrando los resultados en un acercamiento progresivo a la realidad.

La raíz del problema está en la aplicación de los criterios de bondad de ajuste. Los criterios usuales de ajuste, y en particular los que utilizan las ventajas del papel probabilístico, emplean para el ajuste todos los puntos muestrales y en igualdad de condiciones. Este tipo de criterio está sometido a errores potencialmente importantes. Puede demostrarse fácilmente (12 y 14) que la fiabilidad de las estimas muestrales de función de distribución no es uniforme, sino que varía a lo largo del recorrido de valores muestrales. En el caso de muestreos aleatorios (de los que las muestras extremales son en general casos típicos), la fiabilidad (aproximación esperada de los valores estimados mediante el muestreo, a los valores reales de la población) de la probabilidad muestral es máxima en el centro de la distribución de la población ($F(x) = 0{,}5$) y disminuye hacia ambas colas de ella. Por lo tanto, *suponiendo que la muestra obtenida tenga un tamaño suficiente, en la serie de puntos correspondientes a las probabilidades muestrales pueden distinguirse dos regiones características:*

— Una zona "central", donde las probabilidades muestrales constituyen una buena (según criterio del usuario) aproximación esperada de las probabilidades de la población.

— Dos zonas, situadas en ambas colas de la serie de puntos muestrales, donde la aproximación esperada es pobre y, por tanto, existe una elevada probabilidad de que las probabilidades muestrales presenten amplias desviaciones aleatorias respecto a las de la población. La aleatoriedad de esas desviaciones no excluye la existencia de un sesgo en ellas, dado por la distribución binomial, empleada en (12 y 14) para mostrar la existencia de la dispersión).

Si se conoce previamente el tipo de FDD adecuada, un criterio eficaz de ajuste consiste en eliminar las dos zonas extremas de baja fiabilidad, ajustando sólo la zona central. Cuando (como es el caso que nos ocupa) el problema estriba precisamente en averiguar cuál es el tipo de FDD adecuada, a efectos de extrapolar en uno de los dos sentidos, el estudio del comportamiento de las colas de los puntos muestrales proporciona un criterio de gran utilidad práctica. Para probar la conveniencia de determinado tipo de FDD, cuando se dispone de un cierto número de muestras aleatorias, se ajusta con aquella función el tramo "central" de cada una de las muestras y se observa la tendencia de las colas muestrales situadas en el extremo correspondiente al sentido en que desea extrapolarse la FDD. *Si el conjunto de colas presenta una dispersión aleatoria (contando con el sesgo indicado antes) en torno a la FDD ajustada, no hay razón para rechazar esa FDD. Por el contrario, la FDD se rechaza si se observa sistematicidad en las desviaciones de las colas. A medida que se vaya disponiendo de mayor número y tamaño de muestras y se prueben nuevos tipos de FDD, el uso de este criterio irá centrando progresivamente los resultados hacia la distribución real de la población, para cada variable estudiada.*

Una ventaja adicional del criterio propuesto está en que permite analizar aisladamente cada cola de la FDD. La conveniencia práctica de centrar el análisis en la cola de interés para las extrapolaciones está en que, en ocasiones, el tipo de FDD que se comporta más adecuadamente en las extrapolaciones hacia un sentido, es distinto del más adecuado para extrapolar hacia el sentido opuesto. Este fue el resultado obtenido en la referencia (12), donde el criterio de elección que se está tratando fue empleado satisfactoriamente en la selección de FDD para la variable altura de ola significante (con muestras casi continuas, que incluyen algunas peculiaridades que se indicarán más adelante). Cuando se da un caso tal, los criterios "convencionales" de selección, basados en la bondad del ajuste bruto a la totalidad del recorrido muestral, son doblemente inadecuados.

En cualquier caso, el criterio de elección propuesto precisa del uso de un cierto número de muestras distintas de la misma variable. Para conseguir estas muestras puede operarse de dos formas:

1. Efectuar mediciones en un cierto punto durante un tiempo muy largo para conseguir suficientes muestras de tamaño aceptable. Esto es, en general, inviable, a nivel práctico, cuando se trata de seleccionar FDD extremal. Resultaría, en cambio, abordable para la FDD no extremal en el caso de bastantes variables de interés.

2. Tomar muestras individuales en un cierto número de puntos. Esta opción resulta más asequible que la anterior, y es la única de las dos abordable para distribuciones extremales. Sin embargo, se precisa formular la hipótesis previa de que la variable en cuestión sigue el mismo tipo de FDD en todos los lugares muestreados. Si bien la hipótesis de que cada variable obedece a la misma FDD en cualquier lugar es una generalización excesiva, resulta en cambio razonable el suponer que existen zonas de comportamiento homogéneo, dentro de las cuales la variable sigue un mismo tipo de FDD. *Esto supone establecer una discriminación entre tipos "climáticos" diferenciales de cada variable.* En tal discriminación, es lógico suponer que el comportamiento estadístico diferencial entre aquellos tipos responde a cambios sustanciales en la constitución física de la variable; de ahí la denominación alusiva al clima. Esta hipótesis fue empleada con buen resultado en (12) al realizar la elección (aludida antes) de FDD para la altura de ola significante: aparecieron dos tipos de FDD distintos según que dominase el "Sea" o el "Swell" en el lugar de observación y en los rangos de altura de ola considerados.

En resumen, se considera inviable la elección de FDD extremal a partir de una sola muestra extremal y sin otra información adicional, pero *resulta abordable la elección cuando se dispone de un cierto número de muestras extremales de la misma variable* y se utiliza el criterio de elección que ha sido propuesto, junto con la *hipótesis de diferenciación climática.*

Existen, sin embargo, variables para las que aún aquella segunda alternativa es inviable por carecerse de muestras extremales suficientes en número, extensión y/o precisión. Un caso significativo a este respecto es la altura de ola. Para abordar estos casos se propone utilizar directamente la ecuación extremal, que no precisa de muestras de valores extremos. Como se verá en los apartados siguientes, *la estima de los parámetros relevantes en la ecuación extremal precisa de tiempos de observación muy inferiores (al menos en un orden de magnitud) a los que serían necesarios para obtener resultados de fiabilidad comparable utilizando muestras extremales con los criterios expuestos hasta aquí.*

4. ECUACION EXTREMAL

La función de distribución extremal de una variable expresa la probabilidad de que un valor cualquiera de esa variable no sea superado (o no sea minorado) en un intervalo de tiempo dado, es decir, la probabilidad de que aquel valor sea el máximo (o mínimo) que se presente en el intervalo.

En la hipótesis de que una variable X es aleatoria, su función de distribución $F(x)$ en un intervalo es conocida, y a lo largo de ese intervalo la variable toma un número n de valores estadísticamente independientes, la función de distribución extremal $\Phi(x)$ en el intervalo es:

$$\Phi(x) = [F(x)]^n \qquad (1)$$

Esta sencilla ecuación es la piedra angular de la estadística de extremos, y en su acepción más elemental expresa la probabilidad de que x sea el mayor (o menor) valor obtenido en n experimentos estadísticos independientes con la variable aleatoria X.

El interés de los técnicos que han venido haciendo uso de la estadística de extremos no se ha centrado en la utilización directa de la ecuación extremal exacta, sino que se ha dirigido hacia el empleo de algunas funciones de distribución que, en ciertas condiciones, aproximan a aquella ecuación de forma asintótica. Estas distribuciones asintóticas han venido siendo estudiadas y desarrolladas desde hace unos setenta años por una serie de autores, entre los cuales R. Fisher y L. Tippett (1928) y M. Fréchet (1927) sentaron las bases de partida y dieron forma a las funciones fundamentales.

La utilidad de las formas asintóticas de $\Phi(x)$ está en que a veces, no se conoce la función de distribución $F(x)$ o/y el parámetro n, pero, en cambio, se dispone de una muestra de valores extremos de la variable. Si se conoce cuál es la forma analítica que toma asintóticamente la distribución extremal, los parámetros de ésta pueden ser estimados mediante ajuste de aquella muestra. Una vez realizada esta estima, se pueden realizar predicciones extremales fuera del rango ocupado por la muestra. Para la determinación del tipo de distribución extremal asintótica que corresponde a la variable en cuestión, es preciso conocer ciertas propiedades de la cola de la función de distribución $F(x)$ y asegurar la conveniencia de las hipótesis formuladas sobre el parámetro n.

En la referencia (20) pueden consultarse los fundamentos y demostraciones de la teoría asintótica: en (12) se da un resumen de las características de las asíntotas y del uso práctico que se viene haciendo de ellas.

El uso directo de la ecuación extremal precisa obviamente de la caracterización de los dos parámetros $F(x)$ y n. Sobre la elección de función de distribución se han indicado ya algunos conceptos básicos. En cuanto al parámetro n, una hipótesis básica implícita en las deducciones de las tres asíntotas es n = constante. Por otra parte, la buena convergencia de las asíntotas con sus correspondientes distribuciones extremales depende de que n tome un valor suficientemente alto; además, naturalmente, de que la cola de la distribución $F(x)$ converja a su vez con suficiente rapidez hacia la forma característica indicada para cada asíntota.

A continuación se va a considerar con algún detenimiento este parámetro n, que apenas ha merecido la atención de la literatura especializada y que, sin embargo, presenta características peculiares que llevan a conclusiones de primera importancia. Una de ellas es que, como se verá en el apartado siguiente, para un amplio grupo de variables geofísicas importantes en ingeniería civil, una de las hipótesis básicas que permitieron la formulación de las tres distribuciones extremales asintóticas no se cumple. Este hecho ha sido ignorado hasta el momento. La consideración del parámetro n lleva, como se expone a continuación, a establecer una *diferenciación de las variables climatológicas en dos tipos básicos con comportamientos característicos distintos.*

El elemental modelo estadístico expuesto anteriormente permite el planteamiento de la ecuación extremal sobre la base de que la variable se presenta un cierto número n de veces, por término medio en el intervalo de tiempo (generalmente un año) considerado. Se parte, por tanto, de suponer que la variable en cuestión caracteriza un fenómeno que aparece de forma discreta en el tiempo. Pueden citarse variables que de forma natural se adaptan a ese modelo: Una de ellas podría ser, por ejemplo, la cantidad total de nieve caída en una localidad en el paso de cada frente frío sobre esa localidad. El valor correspondiente del parámetro n perteneciente a la distribución extremal en el año medio; sería el número medio de frentes fríos que pasan por el lugar en un año.

Sin embargo, *muchas de las variables geofísicas no son directamente asimilables al modelo descrito.* Como ejemplos característicos pueden mencionarse la humedad, la temperatura o la velocidad instantánea del viento o de una corriente. *No tiene sentido hablar del "número de veces" que una de estas variables se presenta en un año.* La variable presenta una evolución continua a lo largo del tiempo, se define en cada instante y, por tanto, se "presenta" tantas veces como uno quiera considerar, en cualquier intervalo de tiempo. *Puede afirmarse que la gran mayoría de variables geofísicas significativas en ingeniería pertenecen a esta clase, cuya evolución es continua.* Dentro de estas últimas, un grupo particularmente importante es el constituido por magnitudes totalizadas o promediadas en un cierto intervalo de tiempo; Como ejemplos pueden citarse el recorrido del viento (o su velocidad media) en un intervalo estándar como una, seis o veinticuatro horas; el caudal medio (o el volumen total desaguado) de un río en veinticuatro horas; la precipitación total en veinticuatro horas o treinta días; la temperatura media en una hora, etc. *En publicaciones estadísticas que tratan de las distribuciones extremales asintóticas, es habitual considerar que la ecuación extremal [1] es directamente aplicable a estas variables con sólo igualar n al cociente entre el intervalo de definición de la distribución extremal y el intervalo de definición de la variable.* Como ejemplo característico puede citarse la referencia fundamental de E. Gumbel (1958), que al tratar de la distribución extremal anual del volumen desaguado por ríos en veinticuatro horas ("Statistics of Extremes", pág. 237), hace n = 358 con la observación (matizada) de que no todas las 355 observaciones anuales son estadísticamente independientes. Sin embargo, *esta forma de tratar a las variables citadas es inadecuada y conduce a resultados erróneos.* Un ejemplo en que se toman dos variables-promedio con intervalos de definición de pequeña duración, ayuda a mostrar cómo se llega a conclusiones absurdas: Consideremos el caudal medio de un río en cinco minutos. Si su función de distribución en el año medio es $F(x)$,

la distribución extremal sería $\Phi(x) = [F(x)]^{[illegible]}$ donde la cifra en el exponente es el número de intervalos de cinco minutos que se yuxtaponen en un año. Si el río es de grandes dimensiones, la función de distribución del caudal medio en diez minutos es prácticamente idéntica a la del caudal medio en cinco minutos ($F(x)$), ya que la curva de variación de estas variables en el tiempo, cuando ésta se toma a escala de minutos, es frecuentemente tendida. Por tanto, las distribuciones extremales de ambas variables deben ser también iguales. Sin embargo, la distribución extremal para el caudal del medio en diez minutos obtenida a partir de la misma hipótesis anterior, sería $\Phi(x) = [F(x)]^{[illegible]}$, que es sustancialmente diferente a la expresión obtenida para la otra variable. Los períodos de retorno correspondientes a ambos casos difieren en un factor $\simeq 2$ dada la siguiente relación aproximada, válida para $T(x) > 5$-10:

$$T(x) = \frac{1}{1 - [F(x)]^n} \simeq \frac{1}{n\,[1 - F(x)]}$$

En este caso la hipótesis de independencia estadística entre las observaciones es obviamente inadecuada, pero aun así ambas ecuaciones conservarían asintóticamente su aplicabilidad en virtud de un teorema de G. Watson, citado por E. Gumbel ("Statistics of Extremes", pág. 164), según el cual si la variable es limitada la distribución asintótica del valor mayor es la misma en el caso de observaciones interdependientes que para las independientes.

El resultado absurdo al que se ha llegado, indica que no es esta la forma correcta de aplicar la ecuación extremal a las variables totalizadas o promediadas en un intervalo. Una mirada más atenta descubre que este tipo de variables es, en realidad, en todo análogo a las variables de definición instantánea, a que se aludió anteriormente, en cuanto que todas ellas toman un número ilimitado de valores en cualquier intervalo (mayor que el intervalo de definición de la variable), que se desee. Desde este punto de vista, todas ellas son variables de evolución continua en el tiempo. *El caudal medio (o el volumen total desaguado) de un río en veinticuatro horas posee, en efecto, una evolución continua a lo largo del año y el número de valores que toma en un año no es 365, sino infinidad de ellos.* Es cierto que sólo 365 de ellos se yuxtaponen exactamente, pero ello no tiene una relevancia particular: Ni en la propia constitución de la variable, cuyo ritmo de evolución natural no tiene conexión alguna con su intervalo de definición, ni en las aplicaciones ingenieriles, puesto que quien utilice la distribución extremal está interesado en conocer el *"máximo volumen desaguado en veinticuatro horas, en un año"* y *no solamente "el máximo volumen desaguado en aquellos intervalos de veinticuatro horas que comienzan a las cero horas, en un año".* Ambos conceptos pueden diferir grandemente, dando el segundo de ellos valores significativamente inferiores al otro en la gran mayoría de los casos. El segundo de aquellos conceptos, pues, alude a una interpretación de la variable que no solamente está desconectada de la constitución física de ésta, sino que también se aleja peligrosamente de los requerimientos prácticos de la predicción extremal.

Una variable de gran interés en ingeniería marítima, que pertenece a este mismo tipo cuya evolución es continua, es la altura de ola significante (o cualquier otro promedio de la altura de las olas) que en un cierto instante se halla en una cierta área del mar. Fue precisamente tratando de esta variable cuando por primera vez (según los datos conocidos por el autor del presente trabajo) se mostró que una hipótesis n = constante produce resultados incoherentes en el análisis extremal. El autor en cuestión es J. Battjes (1970), quien empleó una demostración por reducción al absurdo parecida a la anterior para refutar un análisis extremal de H_s efectuado por N. Nordenstrom (1959).

El análisis extremal criticado partió de registros de doce minutos de duración, con los cuales se estimó la probabilidad de excedencia en el año medio, $q(H_s)$. El período de retorno que fue asignado, en años (núm. de minutos al año = 525.600), es:

$$T(H_s) = \frac{12}{525.600\, q(H_s)}$$

Battjes observó que podría haberse obtenido la misma función $q(H_s)$ con observaciones mucho más frecuentes, por ejemplo cada minuto, utilizando fotografía aérea o un procedimiento similar. En ese caso, el período de retorno a que se llega con el mismo planteamiento es:

$$T(H_s) = \frac{1}{525.600\, q(H_s)}$$

Resultado incompatible con el anterior. Battjes no intentó dar solución al problema en su artículo, cuyo tema central no era precisamente el análisis extremal.

En el apartado siguiente va a tratarse de la forma en que la ecuación extremal [1] puede ser aplicada a las variables de evolución continua en el tiempo.

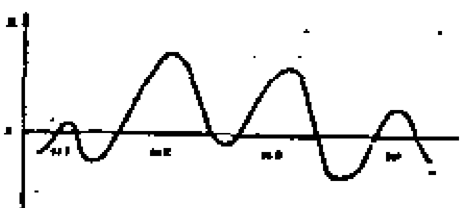
5. LA ECUACION EXTREMAL PARA VARIABLES DE EVOLUCION CONTINUA

La búsqueda de una forma de aplicar la ecuación extremal a las variables de evolución continua, debe dirigirse a encontrar una variable relacionada con ella cuya evolución en el tiempo sea discreta. Sólo con una variable de este tipo el parámetro *n* tiene sentido, y, por lo tanto, en esas condiciones la ecuación puede ser utilizada. Esta discretización de la variable es sólo útil a condición de que, una vez utilizada la ecuación extremal, se pueda volver de nuevo a la variable continua. A continuación va a utilizarse la curva que describe la variable en el tiempo para realizar ese proceso.

La figura siguiente muestra la evolución de los valores de la variable. Se ha visto cómo el considerar un cierto número de valores correspondientes a un valor fijo del espaciamiento entre ellos conduce a un resultado absurdo. Como solución alternativa, vamos a considerar las ondulaciones que describe la curva de evolución. Cada ondulación constituye un cierto fenómeno físicamente independiente del resto y con una forma propia. Sin embargo, no podemos asignar a cada ondulación entera una determinada duración. En lugar de eso, efectuamos un corte de la curva a un nivel *x* cualquiera de la variable y consideramos los tramos ondulados que quedan encima de él.

Cada tramo señala un intervalo de tiempo durante el cual la variable toma valores que exceden a *x*. Podemos denominar a estos tramos "curvas de excedencia del nivel *x*", o abreviadamente "*excedencias de x*".

Supongamos que las excedencias de *x* se presenten de forma independiente. La referencia [12] se detiene en esta afirmación para precisar algo más su alcance; por ahora la consideraremos válida, en función de la evidente individualidad física de cada curva de excedencia. Vamos ahora a establecer, *en cada nivel x, una dicotomía constituida por dos posibilidades: la aparición de una excedencia de x, o su no aparición.* Para ello necesitamos hacer abstracción del tiempo y considerar tan sólo las "veces" que aparece al año una excedencia de *x*, y las "veces" que "podría" aparecer, o número de pruebas estadísticas aleatorias.



Para reducir el tiempo durante el cual el nivel *x* es excedido a las "veces" que esto ocurre, tomamos como duración de cada "vez" o prueba estadística, la duración media $t(x)$ de las excedencias de *x*. El número de pruebas estadísticas en un año medio es:

$$n(x) = \frac{T_y}{t(x)} \qquad [2]$$

donde T_y es el tiempo total del año, expresado en las mismas unidades que $t(x)$.

La probabilidad $q(x)$ de que en una de las $n(x)$ pruebas aparezca una excedencia de *x*, es:

$$q(x) = \frac{n_x}{n(x)} \qquad [3]$$

Donde n_x es el número medio de apariciones de la excedencia de *x* en un año.

$$n_x = \frac{\sum t_{xi}}{t(x)} \qquad [4]$$

Combinando las dos expresiones anteriores:

$$q(x) = \frac{\sum t_{xi}}{T_y} \qquad [5]$$

Esta es precisamente la expresión de la probabilidad absoluta de presentación de valores de la variable superiores a *x*, es decir, el valor complementario de la función de distribución $F(x)$. Con ello hemos vuelto a la variable continua original:

$$q(x) = P(X > x) = 1 - F(x) \qquad [6]$$

Ahora podemos llegar a la función de distribución extremal de la variable. La probabilidad de que en el año medio no se presente ninguna curva de excedencia de *x*, es:

$$[1 - q(x)]^{n(x)} = [F(x)]^{n(x)} \qquad [7]$$

Esta es la probabilidad de que el valor *x* de la variable no sea superado en el año medio, es decir, la función de distribución extremal de la variable:

$$\Phi(x) = [F(x)]^{n(x)} \qquad [8]$$

La expresión [8], distribución extremal de una variable de evolución continua, es similar a la [1], distribución extremal de una variable de evolución discreta, excepto que el exponente n es una constante en el caso discreto pero es función de x en el caso continuo. Esta diferencia origina, naturalmente, que las propiedades características de las distribuciones extremales difieran para ambos casos. En particular, *las propiedades límites que dieron lugar a las tres distribuciones asintóticas fueron deducidas para n = constante, y, por tanto, la extensiva aplicación que se ha hecho de ellas para variables continuas carece totalmente de base teórica, contra lo que se ha venido suponiendo.*

El modelo descrito hasta aquí se presentó ya en [11], donde fue deducido para su uso en análisis extremal de altura de ola significante.

Para definir la distribución extremal en la ecuación [8] es suficiente disponer de la función de distribución de la variable $F(x)$, y la función de duraciones medias de las excedencias de la variable $n(x)$. En los apartados siguientes se trata de las formas que poseen ambas funciones en ciertos casos característicos, a partir del análisis de un número de datos referentes a algunas variables geofísicas significativas en ingeniería civil.

6. FUNCION n (x).

Para abordar la caracterización de la forma funcional de $n(x)$ debe primeramente investigarse cuál es el tipo de relación existente entre los valores correspondientes a la población y las estimas obtenidas a partir de muestras limitadas. Dado un determinado período de observación, donde el valor muestral de $n(x)$ para cada nivel de la variable se obtiene a partir de la duración media de las excedencias contenidas en la muestra a ese nivel, las relaciones entre estima muestral y población son:

— La fiabilidad de las estimas muestrales de la duración media es tanto mayor cuanto mayor sea el número de excedencias con-

DATOS DE CAUDAL MEDIO EN VEINTICUATRO HORAS, EMPLEADOS EN LA DETERMINACION DEL PARAMETRO n (x)

Nombre estación	Situación	Período observado	Duración de cada registro	Intervalo entre registros	Fuente
Trillo.	Río Tajo.	1 octubre 1963 1 octubre 1973	24 horas	24 horas	Centro de Estudios Hidrográficos (Madrid).
Oruseo.	Río Tajuña.	1 octubre 1957 1 octubre 1963			

DATOS DE VELOCIDAD MEDIA DEL VIENTO EN VEINTICUATRO HORAS, EMPLEADOS EN LA DETERMINACION DEL PARAMETRO n (x)

Nombre estación	Situación	Período observado	Duración de cada registro	Intervalo entre registros	Fuente
Valladolid.	Valladolid.	1 octubre 1970 1 octubre 1972	24 horas	24 horas y 12 horas	Instituto Meteorológico Nacional (Madrid).
Peinador.	Vigo.	1 febrero 1970 30 septiembre 1972			

DATOS DE PRECIPITACION EN TREINTA DIAS, EMPLEADOS EN LA DETERMINACION DEL PARAMETRO n (x)

Nombre estación	Situación	Período observado	Duración de cada registro	Intervalo entre registros	Fuente
Los Llanos.	Cuenca del Guadiana. (Prov. de Cuenca).	1 octubre 1940 1 octubre 1970 (excepto 1966-67)	1 mes	1 mes	[illegible] (Madrid).

CUADRO 1-A

lmido de la muestra. Este número (ver figuras 7 y 8, referentes al cuadro 1) es máximo en un cierto nivel de la variable y disminuye hacia niveles superiores e inferiores. Del mismo modo varía la fiabilidad de las estimas de $n(x)$.

— Aceptando que existe una cierta zona "central" de valores de la variable donde las estimas muestrales de $n(x)$ son correctas, los tramos inmediatos por encima y por debajo dan estimas de baja fiabilidad debido al pequeño número de excedencias contenidas en la muestra. Estas estimas se desvían de forma aleatoria en torno a los valores de la población.

— Supongamos que la población de la variable en cuestión es limitada en ambas direcciones. Cada muestra específica está limitada por un valor máximo de la variable y otro mínimo. En estos niveles, los valores muestrales de $n(x)$ son respectivamente ∞ y 0. Los tramos anteriores de desviación aleatoria terminan en dos tramos que se desvían sistemáticamente de los valores correspondientes a la población, tendiendo asintóticamente a ∞ y 0. En el caso de variables cuyo límite natural inferior es cero (como ocurre con H_s), esta desviación sistemática sólo ocurre en el extremo superior muestral.

Por tanto, para cualquiera de estas últimas variables, toda muestra de $n(x)$ con tamaño suficiente posee un tramo central que se ajusta bien a la función correspondiente a la población; dos tramos adyacentes con desviación aleatoria acusada (de los cuales el superior es el más largo, debido a la más lenta disminución del número de excedencias hacia los niveles altos, ver figuras 27 y 28 referentes al cuadro 1), y, por último, un tramo límite superior que se desvía asintóticamente tendiendo hacia $n(x) = \infty$ en el mayor valor muestral.

En la referencia (11), donde se tuvo que abordar la caracterización de $n(H_s)$ sin disponerse de muestra alguna de la curva de evolución de la variable, se admitió una ley exponencial para aquel parámetro con el fin de llegar a una función doble exponencial para la distribución extremal de H_s (función que pareció comportarse adecuadamente en varias muestras extremales obtenidas con la siempre dudosa exactitud de las estimas visuales individuales en temporales). Posteriormente han sido analizados varios años de registro casi-continuo de algunas variables significativas en ingeniería civil (altura de ola significante, velocidad media del viento en veinticuatro horas, caudal fluvial en veinticuatro horas, precipitación en treinta días) con el fin de estimar directamente una forma funcional adecuada para $n(x)$ en estos casos. En el cuadro 1 y figura 9 se muestran las características de las muestras utilizadas. El resultado obtenido del análisis es que la sencilla función lineal, $n(x) = Ax + B$, se comporta correctamente en todo el conjunto de muestras analizado. En las figuras 10 a 19 se muestran los puntos muestrales con las funciones lineales ajustadas a sus tramos centrales. En estas figuras no se han representado los puntos para cuya estima se contaba con un número de excedencias inferior a 10, con lo cual se han limitado considerablemente las desviaciones. En los tramos superiores de desviación aleatoria (los más largos), esa desviación es hacia la derecha de la recta en un mayor número de casos que hacia la izquierda. Esto es coherente con el hecho de que las funciones de distribución de la duración de las excedencias para cada nivel de la variable (de la que se han analizado un cierto número de casos), son apreciablemente sesgadas en el sentido de que las duraciones inferiores a la media resultan más frecuentes que las superiores a ella (a cambio, naturalmente, de tener un recorrido inferior).

El ajuste correcto de las funciones lineales para $n(x)$ requiere el uso de "intervalos de precisión", tal como se describe en la referencia (12). Sin embargo, el ajuste visual al tramo de valores centrales resulta aceptablemente aproximado para el uso de la ecuación extremal, siempre que la *longitud efectiva* de la muestra (longitud del tramo central de estima correcta) sea suficiente. Esto último sólo puede determinarse convenientemente tras análisis de muestras de cada variable objeto de interés. En la referencia (12) se han obtenido algunos resultados interesantes respecto a las variables H_s y V.

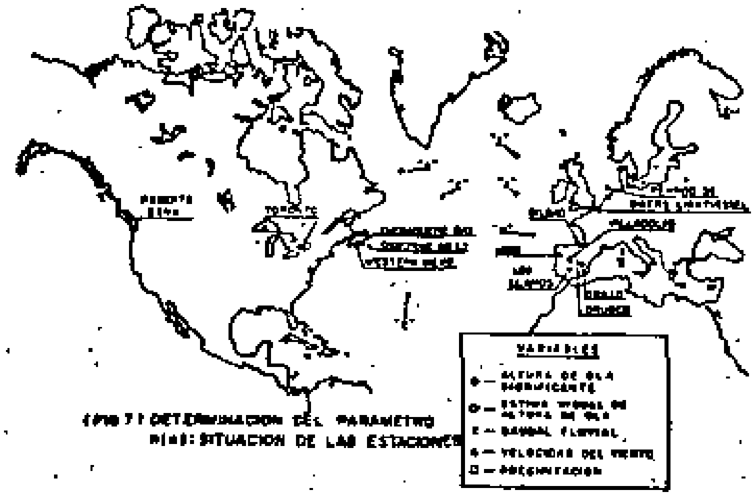
Como puede observarse, no se ha detectado dependencia de la tipología climática en la caracterización funcional de $n(x)$ en los casos estudiados. Sólo el análisis de un mayor número de muestras de cada variable permitirá evaluar hasta qué punto es esto correcto; mientras tanto, los resultados obtenidos en (12) relativos a la baja sensibilidad de $\bullet(x)$ a variaciones apreciables de $n(x)$ llevan a estimar que la función lineal puede ser utilizada con confianza en la práctica.

Entre las figuras anteriores, las referentes a observaciones visuales de altura de ola no tienen un valor elevado en la selección de la relación lineal para $n(H_s)$, ya que la precisión de aquellas observaciones es menos clara de lo deseable en la región de valores altos de la altura de ola. Se las incluye, sin embargo, como comprobación "a posteriori" de que la función lineal produce ajustes razonables y de valor práctico para estas estimas de oleaje realizadas un tanto rudimentariamente, pero que constituyen una fuente de datos de primera importancia (a menudo la única disponible).

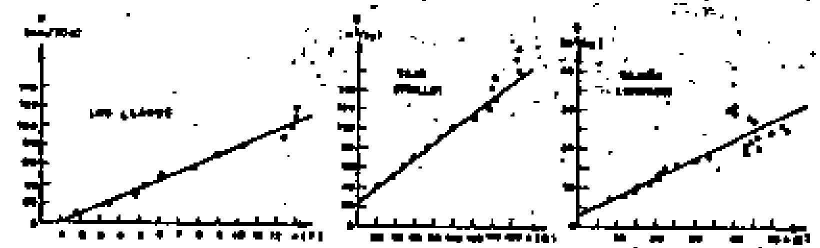
DATOS DE ALTURA DE OLA EMPLEADOS EN LA DETERMINACION DEL PARAMETRO $n(x)$

Nombre estación	Situación	Área marítima	Profundidad (m)	Período observado	T. [illegible] (h)	% T. del	Tipo de observac.	Duración de cada registro	Intervalo entre registros	Fuente
Osborne Head	Nova Scotia (Canadá)	Atl. Norte (W)	10,2	13 diciembre 1970 1 septiembre 1976	37.134	78,1	Instrument. (Boya)	20 min.	3 horas	
Western Head	Nova Scotia (Canadá)	Atl. Norte (W)	40,3-43,0	16 abril 1970 5 mayo 1973	19.484	74,2	Instrument. (Boya)	20 min.	3 horas	
Chedabucto Bay	Nova Scotia (Canadá)	Atl. Norte (W)	86,7	24 octubre 1974 4 febrero 1976	9.081	61,3	Instrument. (Boya)	20 min.	3 horas	[illegible] (Marine Information Directorate, Canadá)
Roberts Bank	British Columbia (Canadá)	Estrecho de Georgia	130,0	7 febrero 1974 3 abril 1976	12.[illegible]	78,9	Instrument. (Boya)	30 min.	3 horas	
Toronto	Toronto (Canadá)	Lago Ontario	107,8	14 abril 1972 15 junio 1973	7.163	61,2	Instrument. (Boya)	30 min.	3 horas	
Owers Lightvessel	Owers (Inglaterra)	Canal de la Mancha	13-19	1 octubre 1968 30 septiembre 1969	8.704	99,4	Instrument. (Boya)	12 min.	3 horas	L. Draper (I.O.S., Gran Bretaña)
Plat. "[illegible]"	53° 24,0' Norte- 4° 11,0' Este (Holanda)	Mar del Norte	28,0	1 marzo 1973 28 febrero 1974	7.160	81,8	Instrument. (Boya)		3 horas	E. Bouws (K.N.M.I., Holanda)
[illegible]	Bilbao	Cantábrico	40,0	2 abril 1975 2 abril 1976	17.330	98,8	Instrument. (Boya)	12 min.	2 h - 4 h	L. Tejedor (Tesis, para el Puerto de Bilbao)
Barco meteor. "B"		Atlántico Norte		1949-1972		81,0	Visual		1 h - 3 h	
Barco meteor. "C"		Atlántico Norte		1952-1972		96,0	Visual		1 h - 3 h	
Barco meteor. "E"		Atlántico Norte		1952-1971		88,0	Visual		1 h - 3 h	
Barco meteor. "I"		Atlántico Norte		1949-1971		90,0	Visual		1 h - 3 h	Environmental Data Service (U.S.A.)
Barco meteor. "J"		Atlántico Norte		1949-1971		84,0	Visual		1 h - 3 h	
Barco meteor. "K"		Atlántico Norte		1949-1966		83,0	Visual		1 h - 3 h	

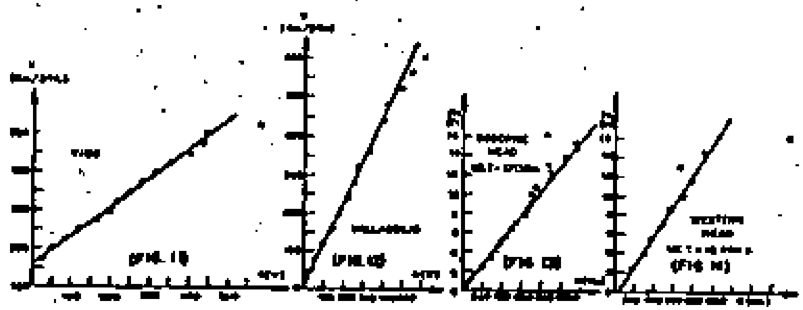
CUADRO 1-B



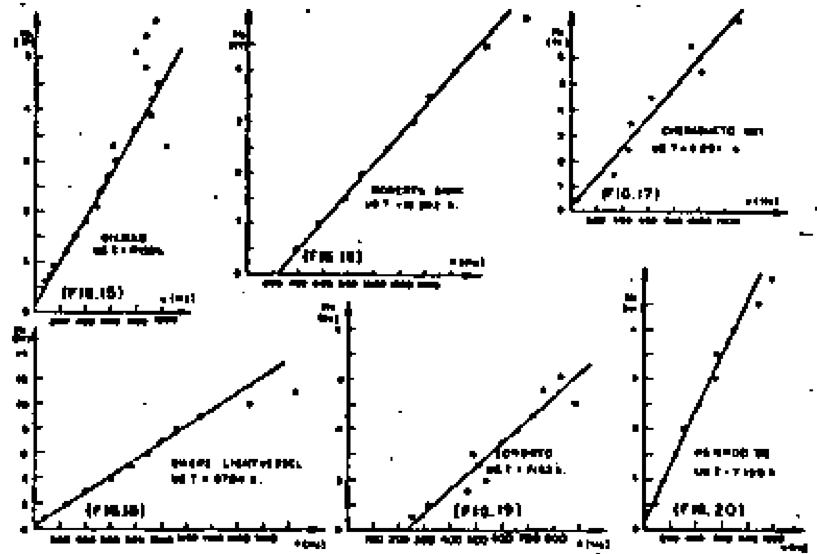
(FIG 7) DETERMINACION DEL PARAMETRO ... SITUACION DE LAS ESTACIONES

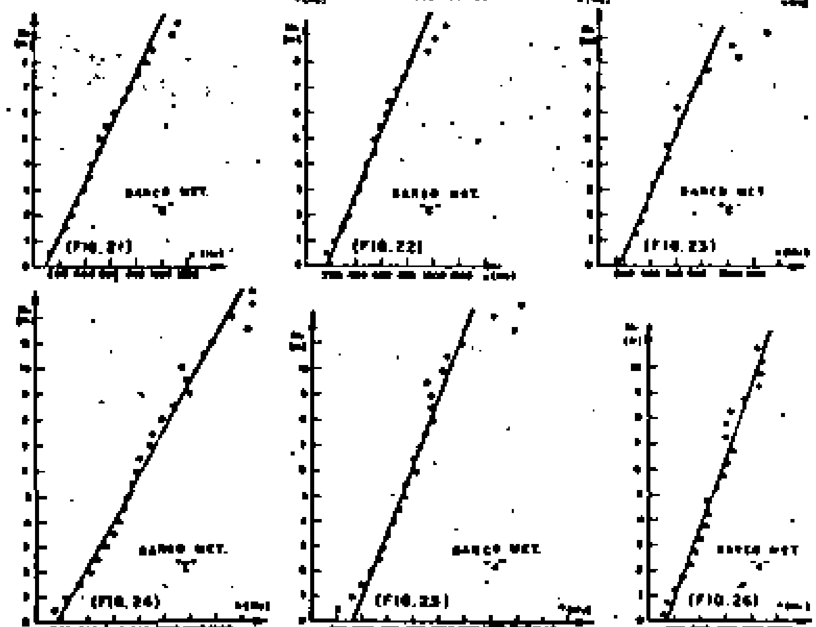


(FIG. 8) (FIG. 9) (FIG. 10)



FUNCION DE DISTRIBUCION EXTREMAL EN FENOMENOS DE TIPO METEOROLOGICO





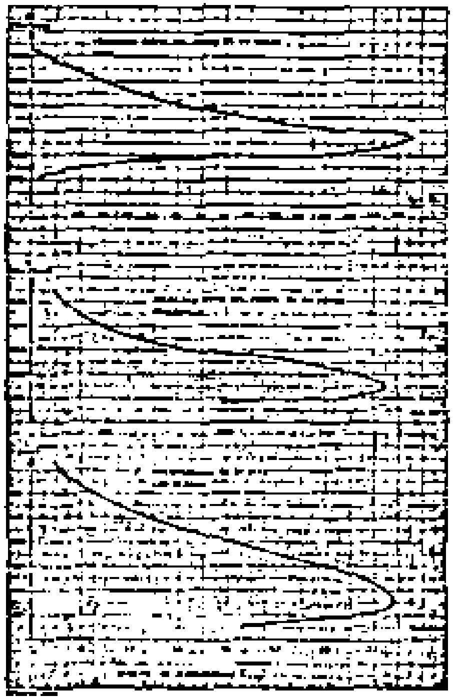


Figura 27.

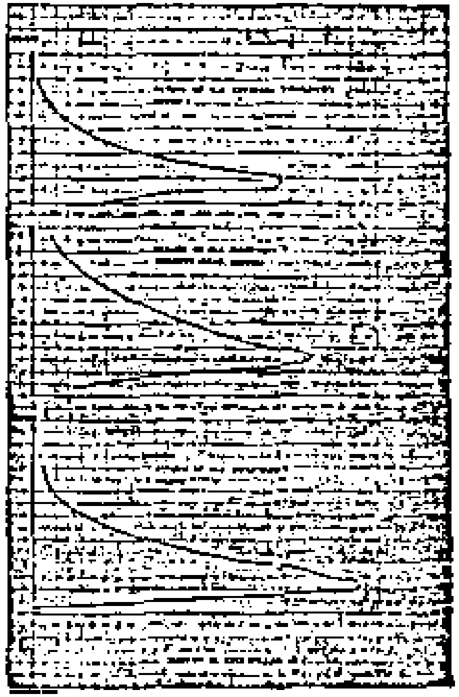


Figura 28

7. FUNCION F (x).

Anteriormente se ha descrito la relación que existe entre los valores de F (x) correspondientes a la población, y las estimas obtenidas a partir de muestras limitadas, en el caso de muestreo aleatorio. *Para muestreo continuo o casi-continuo, la situación es similar incluyendo además tramos de desviación sistemática en los extremos muestrales (12 y 14).* Según se indicó en aquel apartado anterior, la determinación de función de distribución para cada variable necesita del análisis comparado de un elevado número de muestras diferentes (en la referencia (12) se realiza esta determinación para la variable altura de ola significante, con ayuda de 20 muestras distintas pertenecientes a diversas áreas marítimas).

Sin embargo, una sola muestra de buen tamaño puede servir de base a una determinación efectiva de F (x) si se dispone al mismo tiempo de una muestra extremal cuyo *recorrido* sea largo una longitud apreciable, aunque esta sola no permite, como se indicó antes, la elección directa de distribución extremal. Para ello hay que partir de suponer que el tipo de función adecuado para n (x) es conocido. En el apartado anterior se vio que n (x) puede admitirse lineal, al menos para varias variables importantes. Como ejemplo van a tomarse dos casos: La velocidad media del viento en veinticuatro horas registrado en Valladolid (1970-1975) y la precipitación total en treinta días registrada en Los Llanos. Cuenca (1940-1969), que son dos de las estaciones que figuran en la tabla 1. En las figuras 29 a 31 se muestran ajustes de las distribuciones observadas para ambas estaciones, por funciones de distribución Weibull, Log-Normal de tres parámetros, y Doble Exponencial. Con estas tres hipótesis para F (x), y las funciones n (x) que fueron determinadas en las figuras 6 y 12, se han calculado las distribuciones extremales correspondientes, que se han representado en las figuras 32 a 34 junto con dos muestras extremales de ambas

FUNCION DE DISTRIBUCION EXTREMAL EN FENOMENOS DE TIPO METEOROLOGICO

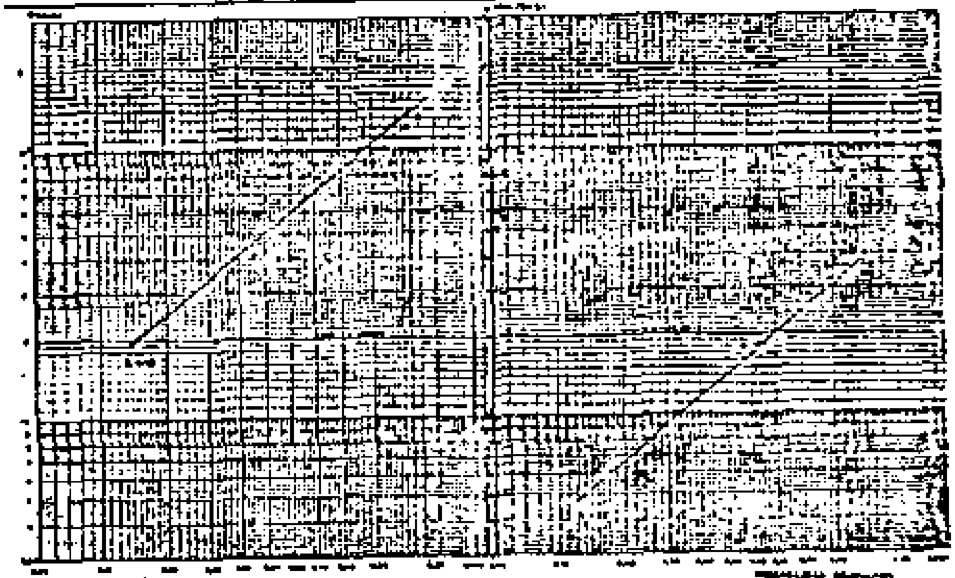


Figura 29.

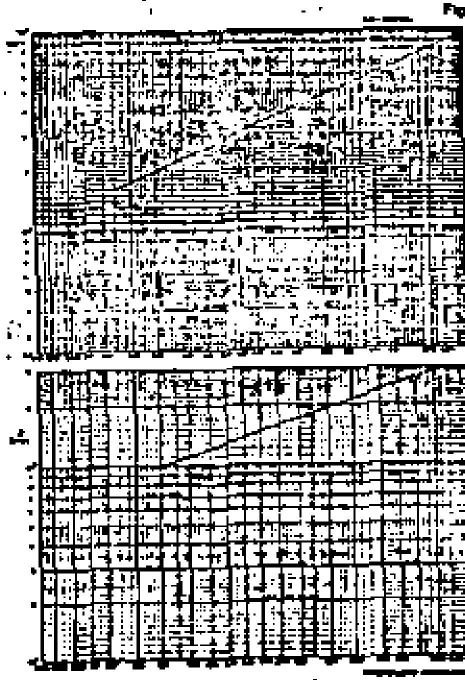


Figura 30.

estaciones. Estas últimas han sido representadas según tres criterios distintos: frecuencias observadas (12 y 14); fórmula de Weibull-Gumbel; fórmula de Gringorten; sin establecer intervalos de precisión y, por tanto, sin delimitar un recorrido útil.

De la inspección de las figuras se desprende que la función Doble Exponencial es, en ambos casos, adecuada para F (x). Al mismo tiempo, se han determinado las distribuciones extremales φ (x) correspondientes a las dos estaciones. Ninguna de estas funciones F (x) y φ (x) hubiera podido ser determinada directamente de forma individual, a partir de sus muestras respectivas, a no ser que por otro lado se conociese ya previamente cuáles eran los tipos de función adecuados para ambas distribuciones.

8. RESUMEN

El cuadro 2 esquematiza los sistemas de estima de distribución extremal que han sido discutidos en este artículo.

Han quedado por tratar algunos aspectos complementarios de esta problemática, que son importantes para utilizar el análisis extremal con el ne-

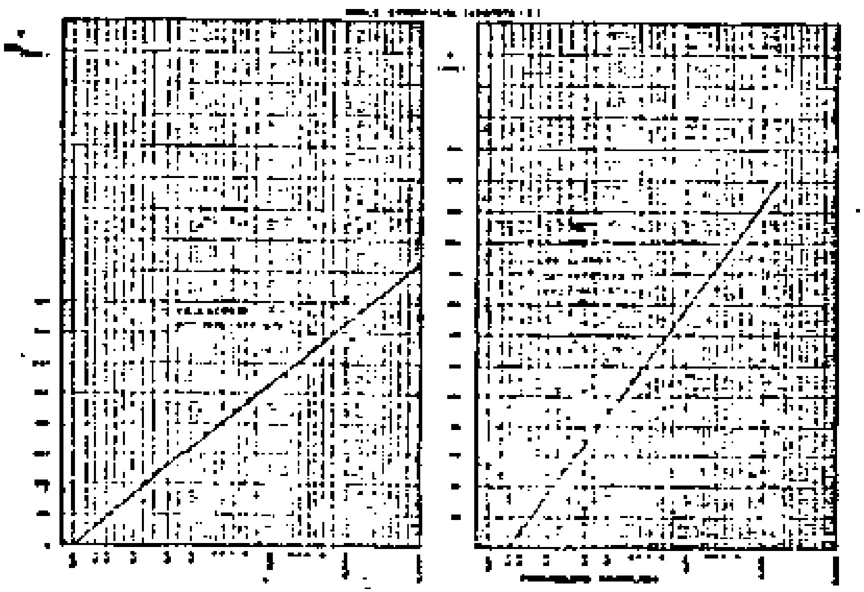


Figura 31.

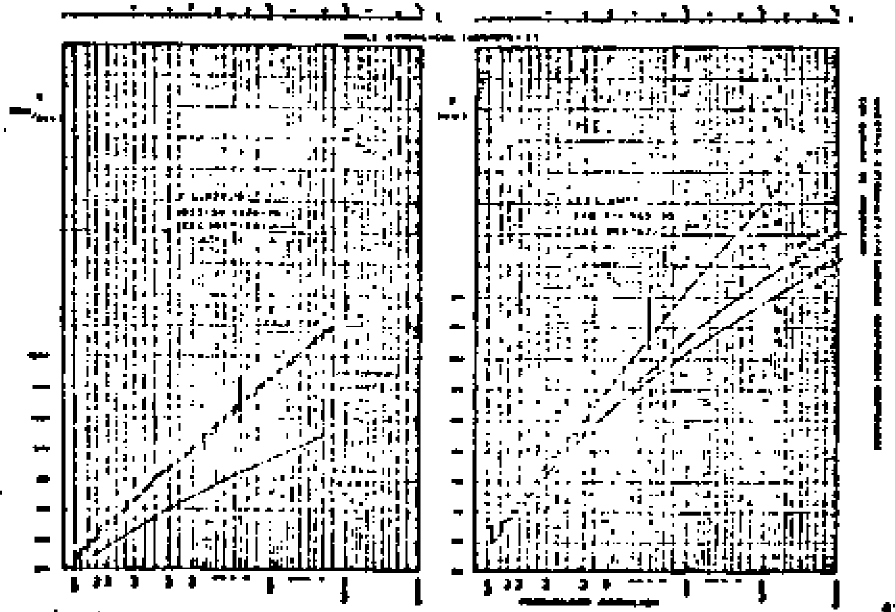


Figura 32.

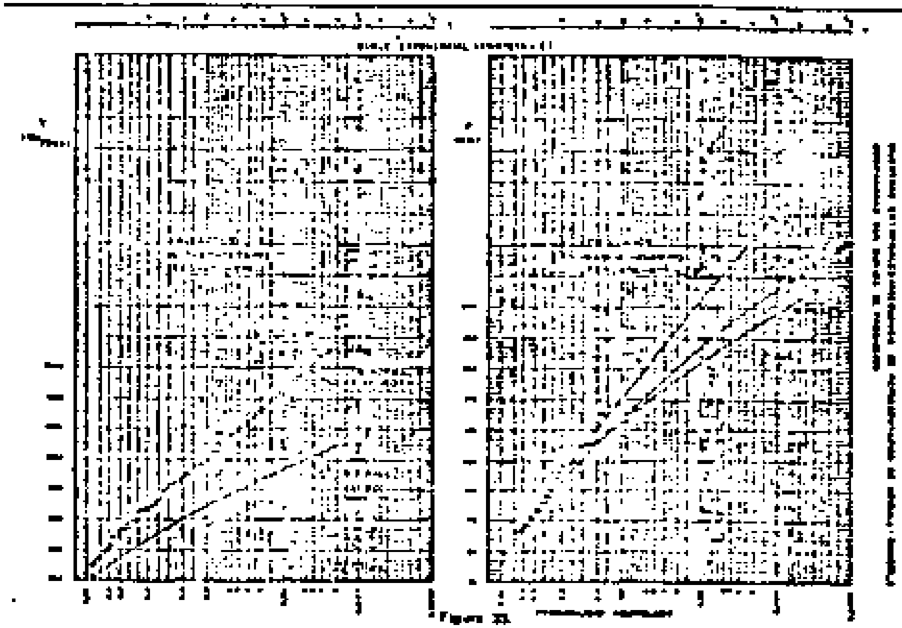


Figura 33.

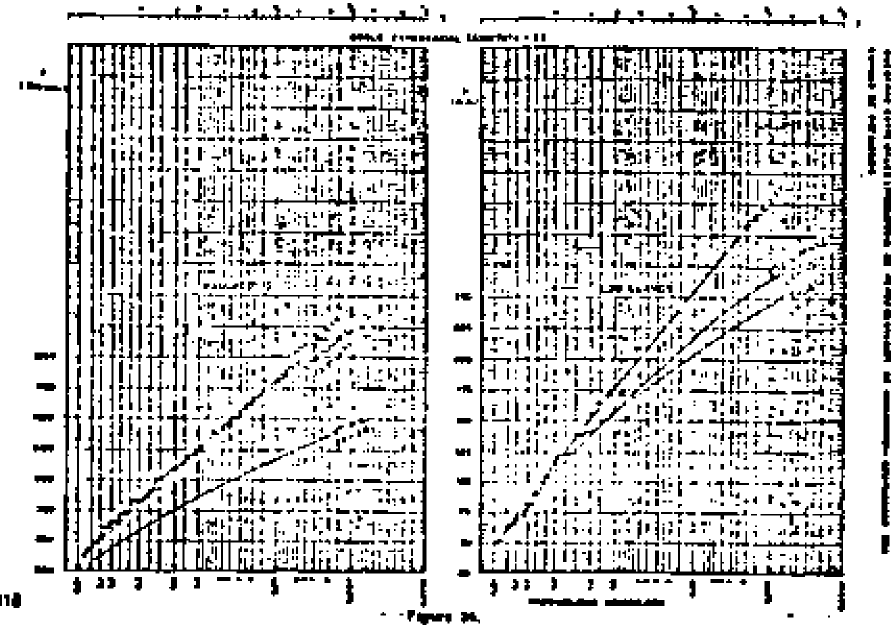
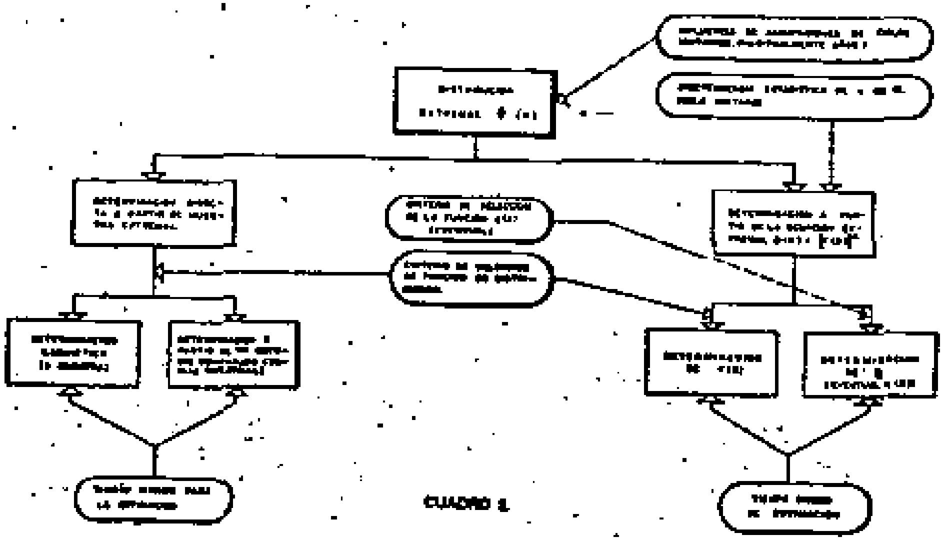


Figura 34.

cesario conocimiento de las limitaciones y posibilidades de la metodología a nivel de su empleo en la práctica: Por un lado, los tiempos mínimos de observación o tamaños muestrales mínimos para una estima correcta de las funciones utilizadas.

Por otro, las repercusiones de la mayor o menor corrección de la hipótesis de independencia estadística para n dentro del ciclo unitario escogido y para los propios ciclos unitarios entre sí. Estos temas se discuten en (12).



CUADRO 5

8. AGRADECIMIENTOS

En la obtención y tratamiento de los datos empleados para la caracterización de varias funciones se ha contado con la amable colaboración de un número de personas e instituciones enumeradas en (12). Entre ellas ha resultado particularmente importante la colaboración de Intecsa (Madrid), que por medio de Victoriano Fernández Dupuy proporcionó uso libre de su ordenador IBM-370-138 (VM) y la asistencia de sus especialistas en programación para elaborar los programas de ordenador necesarios.

El presente artículo constituye una parte de la tesis doctoral presentada por el autor a la E. T. S. de Ingenieros de Caminos, Canales y Puertos de Santander, y publicada en (12). Parte del mismo fue también parte de una comunicación presentada (13) a la XVI Conferencia Internacional de Ingeniería de Costas (I.C.C.E.; Hamburgo, 1978).

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(14)

Estima de función de distribución a partir de una muestra aleatoria (*) /102

Por ENRIQUE COPEIRO

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1. Factores de indeterminación en la estima de función de distribución a partir de una muestra de valores independientes.

En la estima de función de distribución a partir de una muestra aleatoria existen varias etapas sucesivas. Selección de la muestra, asignación de probabilidad a los puntos muestrales, ajuste de una función a aquellos puntos. Para la realización de cada una de estas etapas han sido propuestos diversos criterios y fórmulas, que producen una variedad de resultados distintos. El estado de indeterminación a que se refiere este apartado es consecuencia de que *la metodología estadística en uso no ha proporcionado hasta el momento criterios inequívocos de elección entre las diversas propuestas existentes*. Los principales factores de indeterminación son:

1.1. *Frecuencia de representación*

Si los valores de una muestra de una variable aleatoria pueden suponerse independientes (como ocurre típicamente con las muestras de valores extremos, por ejemplo) puede hacerse uso de la teoría estadística de valores ordenados. Ordenados los N valores muestrales en sentido creciente (o decreciente), se les asigna un número de orden $m = 1, 2, 3, \dots, N$. A cada valor le corresponde una probabilidad de ser excedido (o no serlo), de acuerdo con su posición dentro del grupo ordenado. La propuesta aparentemente más adecuada sería utilizar la probabilidad observada en la propia muestra, sin embargo, esta posibilidad ha sido rechazada en función de las dos razones siguientes (E. Gumbel, 1958):

1. A cada valor m-ésimo se le puede asignar dos valores distintos de la frecuencia acumulada, según se le incluya a uno u otro lado del límite de integración. Es consecuencia natural de tratar un proceso discreto. Ambos valores son, respectivamente, $F(X_m) = \frac{m}{N}$ y $F(X_m) = \frac{m-1}{N}$. Ambas series de probabilidad convergen en la zona central de la muestra, pero divergen significativamente en sus extremos en términos de período de retorno.

2. La fórmula $\frac{m}{N}$ asigna al mayor valor una probabilidad $P(X_N) = 1$. La fórmula $\frac{m-1}{N}$ asigna al menor valor una probabilidad $P(X_1) = 0$. Los papeles probabilísticos que generalmente se usan en el análisis extremal no incluyen normalmente alguno (o ninguno) de ambos valores límite de la probabilidad y, sin embargo, se desea utilizar todos los N valores muestrales para el ajuste de la función.

Entre las propuestas efectuadas para solucionar este problema figuran varias fórmulas empíricas que constituyen compromisos de carácter práctico entre las dos series de probabilidad señaladas arriba. Por el lado teórico se han presentado también una serie de fórmulas basadas en distintas propiedades de la distribución del valor m-ésimo de una muestra aleatoria de tamaño N.

La función de densidad del valor m-ésimo de una muestra aleatoria de tamaño N perteneciente a una población cuya función de densidad es $f(x)$, puede escribirse:

$$\varphi_m(X_m) = \frac{N!}{(N-m)!\,(m-1)!}\,[F(X_m)]^{m-1}\,[1 - F(X_m)]^{N-m}\,f(X_m) \dots (1.1)$$

A partir de esta ecuación pueden obtenerse diversos valores estadísticos característicos del valor m-ésimo.

En concreto, se han empleado las probabilidades correspondientes a la media, moda o mediana muestrales o bien la media, moda o mediana de la probabilidad. Estas posibilidades producen resultados que varían notablemente en los extremos de la muestra. Los extremos muestrales son precisamente regiones a las que se ha venido atribuyendo una importancia especial en los criterios de ajuste. Tanto, que en el proceso de deducción de algunas fórmulas de frecuencia de representación, que precisaban el uso de formulaciones aproximadas debido a la complejidad analítica de la formulación exacta, se ha exigido de la aproximación propuesta la más estrecha convergencia en el extremo (Gringorten, 1963). Los criterios habituales de ajuste a una muestra extremal producen resultados que son altamente sensibles a las distintas fórmulas de frecuencia de representación. Aunque en los valores muestrales centrales las probabilidades calculadas por los diversos métodos son muy similares, en los valores extremos (altos y bajos) esas probabilidades divergen ampliamente, siendo máximas las diferencias en el último valor. Esto hace que las rectas ajustadas en el papel probabilístico tengan pendientes distintas, lo cual produce, en las extrapolaciones, predicciones tanto más dispares cuanto más nos alejemos en la escala de períodos de retorno. Por ello, la comparación entre las distintas probabilidades asignadas al último valor por las distintas fórmulas da una buena impresión de las discrepancias entre métodos. A continuación, se comparan las probabilidades correspondientes al mayor valor muestral (las del menor valor darían un panorama análogo) cuando se utilizan algunas de las fórmulas más conocidas. Se consideran dos tamaños muestrales, $N = 50$ y $N = 100$, y se incluyen junto a las probabilidades sus períodos de retorno correspondientes, que quizá proporcionan un contraste más intuitivo.

Las diferencias observadas son grandes, aun cuando las dos últimas fórmulas corresponden a hipótesis distintas sobre la F.D.D. y, por tanto, la comparación entre ellas no es estrictamente permisible.

En un reciente artículo de L. Borgman y D. Resio (1977), donde se obtuvieron predicciones ex-

TABLA 1

AUTOR	EXPRESIÓN	CRITERIO	N = 50: P Probabilidad	N = 50: T Período de retorno	N = 100: P Probabilidad	N = 100: T Período de retorno
Hazen	$\frac{m-\frac{1}{2}}{N}$	Arbitrario.	0,9900	100	0,9950	200
Weibull-Gumbel	$\frac{m}{N+1}$	Media de la probabilidad del valor m-ésimo (independiente de la F.D.D.).	0,9804	51,0	0,9901	101,0
Johnson-Benard	$\frac{m-0{,}3}{N-0{,}4}$	Mediana de la probabilidad del valor m-ésimo (independiente de la F.D.D.).	0,9851	72,0	0,9930	143,4
Blom	$\frac{m-\frac{3}{8}}{N+\frac{1}{4}}$	Probabilidad del valor medio m-ésimo de una F.D.D. normal.	0,9758	41,0	0,9878	81,0
Gringorten	$\frac{m-0{,}44}{N+0{,}12}$	Probabilidad del valor medio m-ésimo de una F.D.D. doble exponencial.	0,9888	89,5	0,9994	178,8

(*) Se admiten comentarios sobre el presente artículo, que pueden remitirse a la Redacción de esta Revista hasta el 31 de mayo de 1979.

tremales significativamente dispares con la sola variación de utilizar alternativamente las fórmulas de Weibull-Gumbel y de Gringorten, el comentario de aquellos conocidos autores sobre ello fue: "No es realmente cuestión de qué fórmula es la buena o cuál es la mala. Cada procedimiento tiene algunas justificaciones estadísticas o lógicas". Esta opinión es el reconocimiento explícito de un estado de ambigüedad en la metodología estadística existente.

1.2. *Los extremos de la muestra (caso de muestras de valores extremos)*

En ocasiones los valores límites de la muestra (los mayores y menores) se apartan apreciablemente de la tendencia seguida por el resto de la muestra. Aquellos valores condicionan, a veces de forma muy marcada, la pendiente de la recta ajustada en el papel probabilístico a través de la serie de puntos muestrales.

En la metodología en uso para los ajustes se parte de la decisión de utilizar la totalidad de los puntos muestrales. Puede citarse, como ejemplo, que, entre las seis condiciones que según E. Gumbel (1958) debe cumplir una fórmula aceptable de probabilidad de representación, la primera es que debe permitir la representación de todos los valores de la muestra. De esta forma los N puntos muestrales son utilizables para el ajuste. Algunos autores, sin embargo, han admitido la posibilidad de prescindir de algunos de estos valores extremos en las estimas de distribuciones extremales, cuando su posición se aleja de forma "excesiva" de la tendencia marcada por el resto. En este caso se presume que aquellos valores no siguen el mismo tipo de función de distribución que los demás, sino otra ley estadística propia de valores que pueden darse en intervalos de recurrencia muy largos. Por tanto, se supone que estos valores no son estadísticamente homogéneos con el resto y se les excluye del análisis (V. Chow, 1964).

Este punto de vista plantea dos problemas, cualquiera de los cuales basta para invalidarlo. El primero es que la propia posibilidad de extrapolar la curva ajustada queda básicamente cuestionada por la suposición de que los acontecimientos con período de retorno largos siguen una distribución estadística distinta a los otros. El segundo es la vaguedad de la propuesta a nivel de su uso práctico, ya que no se suministra un criterio que permita decidir cuándo la desviación de un determinado punto es "suficientemente" acusada como para decidir su exclusión. Chow llama a estos puntos "datos fuera de control", en el sentido de que caen bien fuera de las bandas de control o límites de confianza que pueden trazarse a ambos lados de la curva ajustada. Podría decidirse el excluir los puntos que no caen dentro de esta banda, pero ¿qué banda de control se adapta para ello? Se obtiene una banda de control diferente para cada nivel de confianza, y éste es una cantidad a fijar arbitrariamente. Por lo demás, la amplitud de las bandas de control habituales es tan grande en los extremos de la muestra, que aun puntos que se desvían muy pronunciadamente de la tendencia del resto pueden estar incluidos en aquellas. Esto es cierto aun en bandas de control muy estrictas, como, por ejemplo, del 68,3 por 100 de confianza (una desviación estándar).

En la figura 1 se muestra un ejemplo que permite apreciar el efecto indicado. La muestra extremal utilizada consiste en 10 máximos anuales de la altura de ola significante estimados para un punto del Cantábrico Oriental mediante previsiones realizadas utilizando cartas meteorológicas. En el caso A aparece la muestra completa en papel Asintota-I. La representación de puntos muestrales, el ajuste de la recta a ellos y el trazado de la banda de control al 68,3 por 100 de confianza, han sido realizados según la metodología propuesta por E. Gumbel (1958). En el caso B se ha empleado la misma metodología para ajustar la misma muestra de donde ha sido excluido el valor mayor. Este valor cae aún dentro de la banda de control en la figura A, pero su posición marca una ruptura tan acusada de la tendencia marcada por el resto de la muestra (nítidamente alineada) que cabe una duda razonable sobre su exclusión, en el espíritu de la recomendación indicada más arriba. Como puede verse, las rectas ajustadas en ambas hipótesis divergen ampliamente (figura B), produciendo grandes diferencias relativas para los períodos de retorno usuales en la práctica.

Este ejemplo muestra la ambigüedad de la metodología respecto al tratamiento de los extremos de la muestra. Se ve cómo dos decisiones, entre las que existe una duda que podemos calificar de razonable a la luz de los procedimientos en vigor, producen resultados desproporcionadamente diferentes, con un impacto enorme en el proceso de diseño de obras o planificación. Debe admitirse que una muestra de diez años es en cualquier caso muy corta. Sin embargo, aunque la influencia de los últimos valores disminuye naturalmente al crecer el tamaño de la muestra, no llega a ser despreciable hasta llegar a muestras muy grandes, poco frecuentes en la práctica. En ingeniería marítima, por ejemplo, se han realizado con frecuencia estimas de distribuciones extremales utilizando muestras de entre veinte y treinta años (C. Petrauskas y P. Aagaard, 1970; P. Suárez Bores, 1974; E. Copeiro, 1976) y aun inferiores (H. Thom, 1970), debido a la inexistencia de datos de larga duración. En estos tamaños la influencia de los valores extremos es aún considerable.

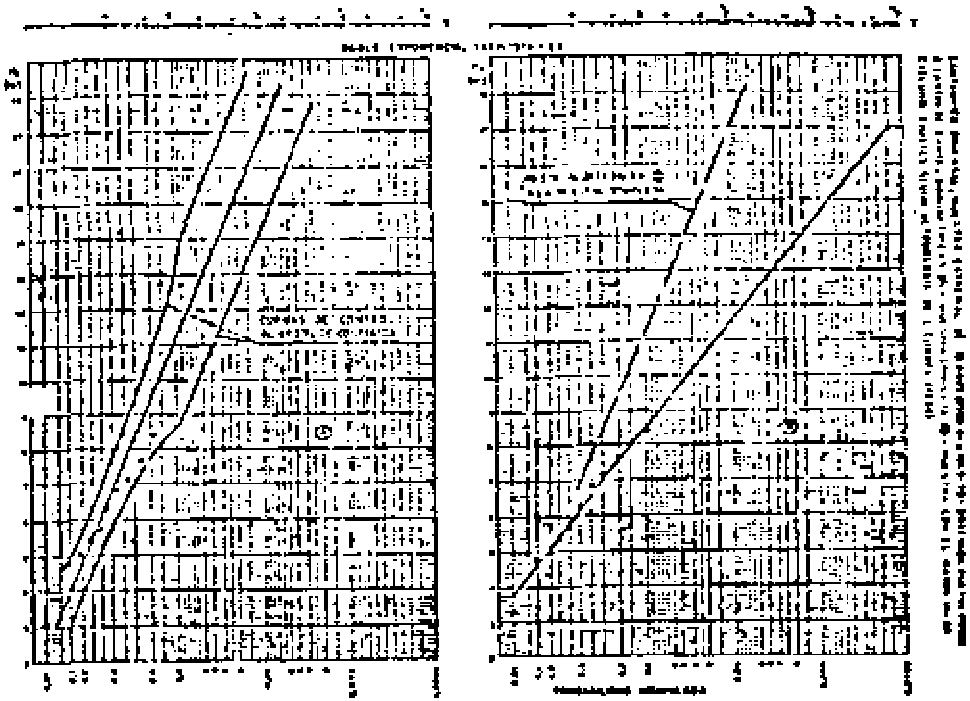


Figura 1.

Variabilidad muestral.

Distintas muestras de una misma población conducen a distintas estimas de la función de distribución. Esta afirmación es obvia.

La variabilidad de resultados obtenidos con distintas muestras depende del tamaño muestral: para mayores tamaños la dispersión es menor. De acuerdo con esto, determinados tamaños de muestra producen estimas "poco" fiables, mientras que otros son "más" fiables. La deficiencia de la metodología estadística en uso respecto a este hecho es que no toma en cuenta la diferente fiabilidad de las estimas. Una muestra de tamaño $N = 10$ es representada y aceptada análogamente a una de $N = 50$, en el sentido de que ningún término estadístico de valoración evalúa en modo alguno el hecho evidente de que la estima efectuada con $N = 50$ está situada próxima a los valores de la población con una probabilidad muy superior a la otra; o, a la inversa, que tomando la estima proporcionada por $N = 10$ corremos un riesgo mayor de estar considerablemente alejados de la población.

Como ejemplo indicativo, que da una idea de las dimensiones que puede tener el efecto de la variabilidad muestral, se han tomado 73 observaciones de precipitación total diaria máxima anual en Madrid (Retiro), correspondientes al período 1901 a 1973-74. Se han considerado años meteorológicos, con comienzo el 1 de octubre. A falta de una muestra mayor, supondremos que $N = 73$ caracteriza a la población total para ver cómo varían en torno suyo las estimas conseguidas con tamaños muestrales menores. En la figura 2 se ha representado la muestra total en un papel probabilístico Asintota-I. Como fórmula de frecuencia de representación se ha escogido la de Weibull-Gumbel, y la recta ha sido ajustada según el procedimiento de E. Gumbel (1958). Los mismos

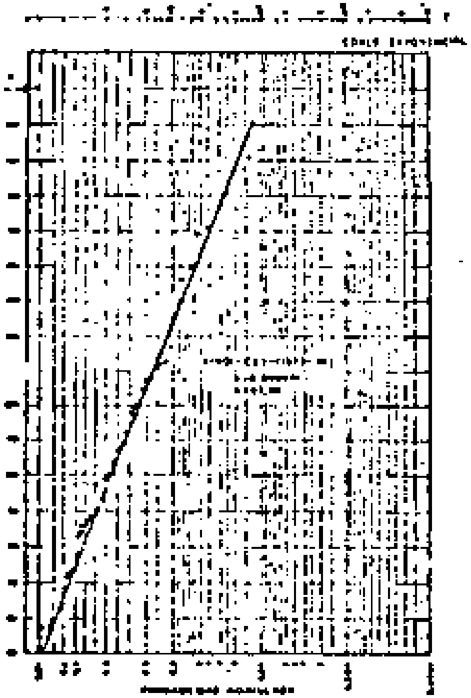


Figura 2

sistemas han sido empleados con todas las muestras analizadas, de modo que se mantiene una homogeneidad operativa necesaria para establecer las comparaciones. Puede añadirse que estos métodos son, quizá, los más populares en los medios especializados. A continuación se han escogido varios tamaños muestrales, N = 10, 20, 30, 40, 50, para ver la dispersión de resultados dentro de cada tamaño y comparar entre distintos tamaños. Para tomar un número relativamente alto de muestras dentro de cada tamaño, se ha admitido un solape de aproximadamente 3/4 de la muestra entre cada dos de ellas consecutivas, aun a costa de perder independencia estadística entre muestras. En el caso de N = 10 el solape ha sido de 1/2 de la muestra, por contarse así con un número suficiente de muestras.

Los ajustes de las muestras individuales pueden verse en las figuras A2-1 a A2-11 de la referencia 5. En las figuras 3 a 5 se muestran los grupos de rectas obtenidas para los seis tamaños muestrales considerados. Como podía esperarse, la dispersión disminuye al aumentar el tamaño muestral. En el cuadro siguiente se expresan los límites de las dispersiones obtenidas en cada grupo, respecto a los valores que señala la recta de la muestra total (N = 73), por encima y por debajo de ella. La comparación se establece en los períodos de retorno N = 100, N = 500, que son altamente significativos en la práctica (riesgo del 10 por 100 de superación en períodos de diez y cincuenta años, respectivamente).

Las dispersiones observadas en los grupos de tamaños menores son tan grandes que estas últimas resultan altamente peligrosas si se piensa en su utilización práctica. Puede hablarse de un estado de indeterminación importante.

N (tamaño muestral)	10		20		30		40		50	
T (período de retorno)	100	500	100	500	100	500	100	500	100	500
Lím. dispersión por encima	53 %	58 %	37 %	39 %	17 %	19 %	7 %	6 %	7 %	8 %
Lím. dispersión por debajo	20 %	22 %	23 %	23 %	24 %	26 %	17 %	19 %	17 %	18 %

1.4 *Ajuste de la distribución*

El método más simple es trazar visualmente una línea recta a través de los puntos muestrales, representados en papel probabilístico. Aunque probablemente es este el sistema más utilizado en la práctica, los textos normalmente los desaconsejan por estar sujeto a factores subjetivos de apreciación difíciles de evaluar. En su lugar han sido propuestos diversos métodos matemáticos de ajuste. V. Chow (1964) los clasifica en tres grupos:

1. *Momentos.*—Se calculan los momentos estadísticos a partir de los valores muestrales, y se sustituyen directamente en la función de distribución a ajustar.

2. *Mínimos cuadrados.*—Se ajusta una línea de regresión.

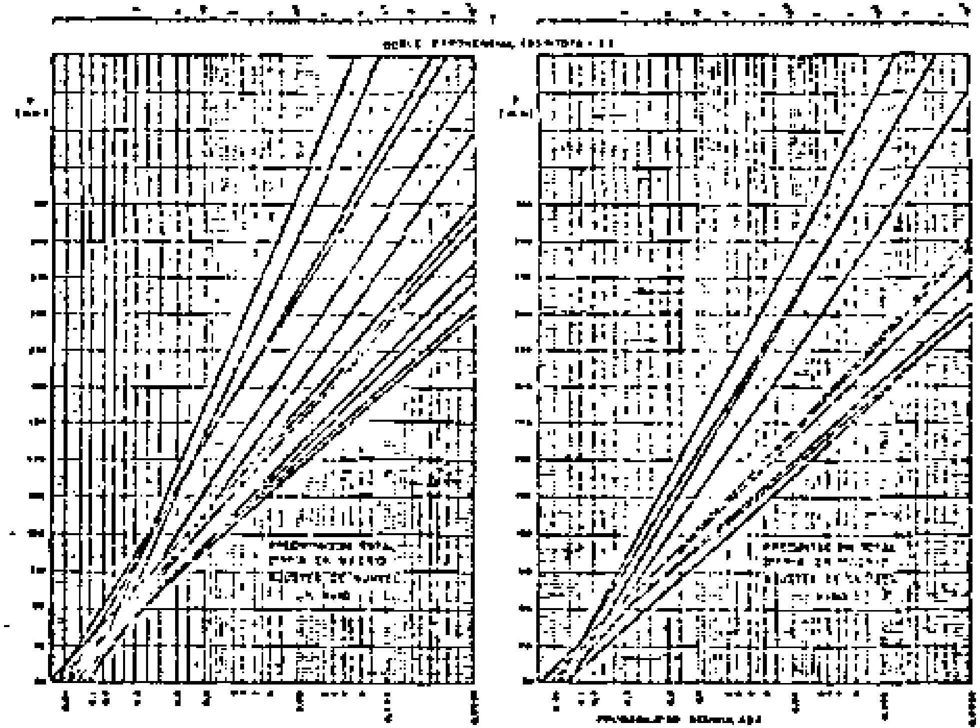


Figura 3.

3. *Máxima semejanza.*—Se determina el valor de un parámetro de forma que se obtenga una probabilidad tan alta como sea posible del resultado muestral observado.

También en esta etapa del proceso de estima pueden resultar diferencias significativas en el uso de una u otra fórmula de ajuste. Como ejemplo puede citarse el comunicado final del Grupo de Trabajo Federal sobre Métodos de Frecuencias de Avenidas, Comité de Hidrología (Water Resources Council, U.S.A.). En este comunicado (M. Benson, 1968) se analizaron 10 muestras extremales largas de avenidas fluviales, ajustándoselas seis tipos diferentes de funciones de distribución de acuerdo con varios métodos matemáticos de ajuste de uso ordinario en algunas Agencias Federales. Uno de los resultados del estudio fue la aparición de considerables diferencias entre las extrapolaciones obtenidas con la misma función de distribución (y la misma muestra), pero distintos métodos matemáticos de ajuste. Al nivel cien años de período de retorno, los rangos de dispersión (sobre el menor de los valores obtenidos en las extrapolaciones) fueron tan grandes como el 34 por 100 (distribución Log Normal, muestra de cuarenta y nueve años en Little Missouri River, Alzada); el 47 por 100 (distribución Asintota-II, muestra de cuarenta años en Mora River, Golondrinas); el 60 por 100 (distribución Log Normal, muestra de cuarenta y cuatro años en Elkhorn River, Waterloo); el 65 por 100 (distribución Asintota-II, muestra de cincuenta y un años en Llano River, Junction); el 109 por 100 (distribución Log Normal, muestra de cuarenta años en Mora River, Golondrinas); o el 119 por 100 (distribución Log Normal, muestra de cincuenta y un años en Llano River, Junction).

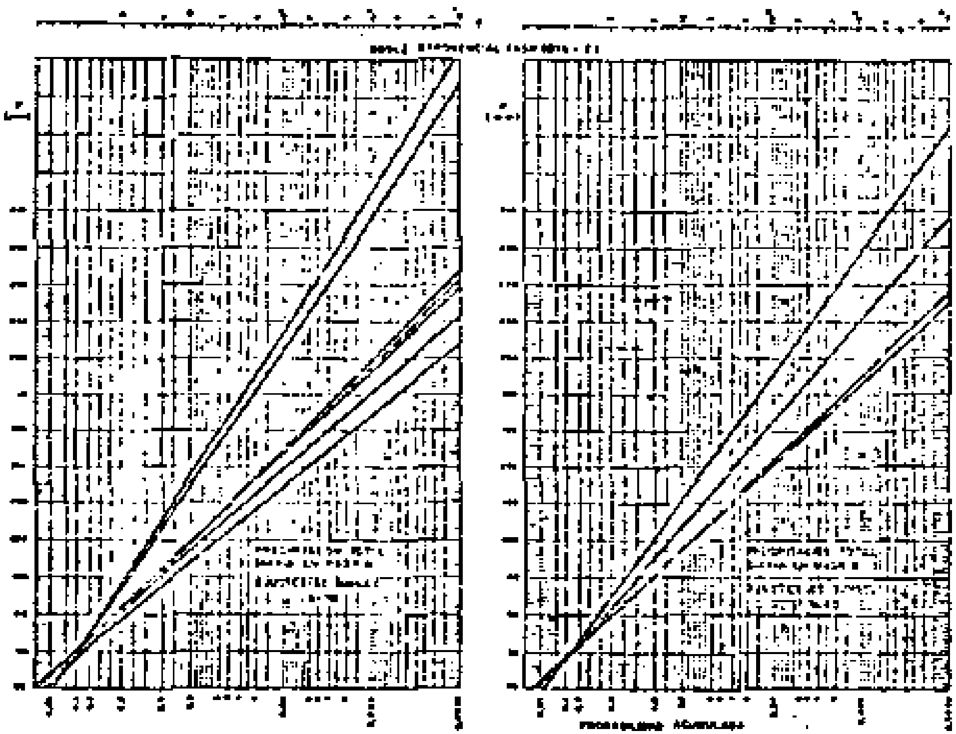


Figura 4.

Estas diferencias son tan considerables que suponen una influencia de orden muy elevado en proyectos hidráulicos. El comentario del Grupo de Trabajo, que estaba asistido en su labor por un grupo de estadísticos profesionales como consultores (J. Rosenblatt, National Bureau of Standars, G. Watson, Johns Hopkins University), fue que el estado actual de estas técnicas no permite seleccionar uno de los métodos usados como correcto o como superior al resto. Una vez más, el panorama que presenta esta metodología al ingeniero práctico es de una ambigüedad desalentadora.

2. Análisis crítico de la metodología en uso y propuesta de alternativas.

Según se ha expuesto en el apartado anterior, *la metodología existente adolece de al menos cuatro factores de indeterminación, todos ellos potencialmente conducentes a errores de orden elevado en las extrapolaciones.* El panorama global que ofrecen estos métodos es el de una amplia gama de propuestas diferentes, que pueden producir resultados tan dispares como para que la elección adecuada entre aquellos sea de una importancia a veces decisiva en las aplicaciones prácticas. Sin embargo, la teoría estadística con que se han desarrollado los métodos a que se alude no ha sido utilizada también para proporcionar al mismo tiempo criterios adecuados de discriminación que permitan adoptar alguno de ellos con confianza.

La situación es indudablemente incómoda para la ingeniería, que en multitud de campos (obras fluviales, construcciones marítimas, etc.) viene desde hace tiempo utilizando muestras extremales de las variables meteorológicas que van a afectar a sus obras para realizar con ellas estimas de sus

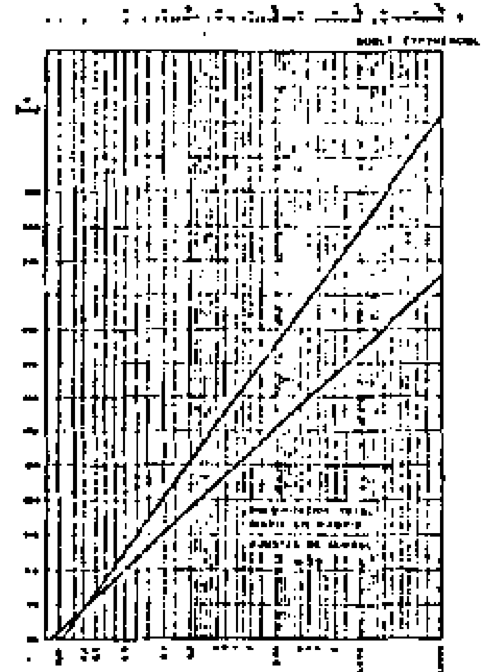


Figura 5.

distribuciones extremales correspondientes. El ingeniero práctico, sobre quien pesa la responsabilidad de adoptar decisiones de alcance real, no puede menos que sentir como poco una profunda desconfianza ante este conjunto de propuestas estadísticas cuyas condiciones de utilización son tan, al parecer, irremediablemente vagas. Sin embargo, es frecuente que los autores que tratan de estadística aplicada, al llegar a este punto, parezcan "pasar" el problema al ingeniero para su solución en un marco no estadístico. Se supone de hecho que el ingeniero es capaz de encontrar salida, haciendo uso de un ingenio peculiar, a las condiciones de indeterminación en que le dejan estos métodos. Entre los artículos que en los últimos años han aparecido sobre la aplicación de la metodología que hemos visto, se puede encontrar varias veces la observación explícita de que tales métodos sólo deben ser utilizados con el auxilio decisivo de un "juicio ingenieril" sólido. Ahora bien, ninguna de las referencias entra a analizar en qué puede consistir ese "juicio ingenieril" en estos casos.

Es posible que se considere al ingeniero capaz de obtener un número suficiente de evidencias directas de campo que permitan la selección razonable de una de las curvas extremales posibles, o quizá de establecer límites a los valores de la variable analizada. Sin embargo, en la gran mayoría de los casos reales esto es sencillamente imposible. Un ingeniero que proyecta una obra marítima en puntos alejados de la plataforma continental, por ejemplo, no dispone generalmente de la menor oportunidad para obtener datos de campo complementarios que le permitan discriminar entre las distribuciones que ha ajustado a su muestra extremal; como tampoco la tiene quien analiza pluviometría o velocidad del viento, y aun sólo muy raramente quien trata con fenómenos que pueden dejar huella clara en el terreno como las avenidas fluviales.

Puesto que, una vez que las distribuciones extremales obtenidas por los diversos métodos están sobre el papel, la información extraestadística obtenible no permite en general salir de la ambigüedad, el único campo restante donde cabe aplicar el aludido "juicio ingenieril", es la propia metodología estadística utilizada. Lo que resta de este artículo es un ejercicio en que se reconsidera la metodología expuesta en los apartados anteriores, desde el punto de vista de obtener resultados de fiabilidad controlada para las aplicaciones prácticas. La argumentación se va a articular en diez escalones, que se desarrollan a continuación:

1. Las formulaciones propuestas para la frecuencia de representación se agrupan en dos clases distintas:

A) Una asigna al valor m-ésimo observado un cierto valor "típico" (media, moda, mediana) de la probabilidad que le corresponde según la estadística de valores ordenados.

B) La otra asigna al mismo la probabilidad correspondiente a un cierto nivel "típico" (media, moda, mediana) de los valores que adopta según la función de distribución de la variable. Ambos tipos de criterio se esquematizan en la figura 6.

Un criterio del tipo A es adecuado cuando el interés de la aplicación del resultado está sobre todo en la determinación correcta del período de retorno (probabilidad) correspondiente a cada valor de la variable. Cuando el interés está en la determinación correcta del nivel de la variable que corresponde a cada período de retorno, es más apto un criterio del tipo B. Este último es el caso habitual en ingeniería. De entre las opciones disponibles en estos tipos de criterios, el elegir la moda (valor más probable) sería la "apuesta" más

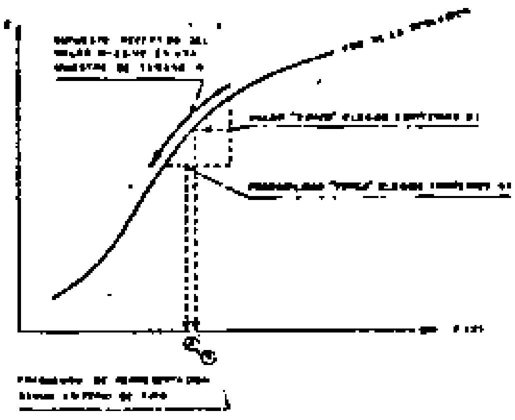


Figura 8

lógica en el caso de un ingeniero que tuviera que proyectar una sola obra en un determinado lugar. Sin embargo, el hacer uso sistemático de esta opción produce resultados que en general no están centrados en torno al valor medio y, por tanto, si se piensa en un número grande de obras proyectadas, el conjunto tenderá a estar sobredimensionado o infradimensionado. Sólo la opción por el valor medio da resultados centrados en la media de la variable y, por tanto, el conjunto de obras proyectadas con este criterio tendrá globalmente un diseño correcto. Esto ha sido argumentado por algunos autores como G. Bom (1950) y C. Petrauskas et al. (1970), para justificar la adopción de esta última opción.

Sin embargo, aunque las condiciones vistas hasta aquí son necesarias o al menos convenientes, no son en absoluto suficientes para la toma de decisiones en ingeniería. Cualquier ingeniero rechazaría un criterio de elección que tan sólo le asegure que sus obras estarán "por término medio" bien diseñadas (unas indeterminadamente sobredimensionadas, otras también indeterminadamente infradimensionadas). Pretendemos que, dentro de las posibilidades ofrecidas por nuestro conocimiento de los factores en juego, "cada una" de nuestras obras tenga un cálculo correcto aun en términos probabilísticos que incluyan un cierto riesgo conocido. *La ambigüedad de los métodos expuestos está sobre todo en que ignoran por completo en qué medida las estimas que producen están más o menos cerca del valor característico que adoptan, y, por tanto, nos dejan desconociendo el riesgo que supone su utilización.* Como se ha visto en los ejemplos de apartados anteriores, el riesgo de grandes dispersiones es, en general, elevado. Entre las seis condiciones postuladas por E. Gumbel en su clásico tratado (1958) para la frecuencia de representación, ninguna hace referencia a este problema básico de que los valores obtenidos sean suficientemente correctos en términos de aproximación de las estimas muestrales con los valores de la población. Sólo su condición número 2 establece la inclusión de la probabilidad entre sus dos límites extremos $\frac{m}{N}$, $\frac{m-1}{N}$, pero en las regiones cruciales de las colas de la distribución estos límites son tan amplios que carecen de utilidad alguna en cuanto al problema real planteado.

2. *Para salir de la indeterminación, es necesario determinar analíticamente la dispersión del valor "típico" elegido para los criterios de tipo B y (o) la dispersión de la probabilidad "típica" elegida para los criterios de tipo A (ver figura anterior).* Esto puede ser realizado para cualquier clase de función de distribución, por medio de la ecuación (11). La determinación de las dispersiones permite evaluar, en términos de probabilidades, el grado de aproximación conseguido en la estima del valor m-ésimo para cada tamaño muestral N.

3. Una vez conocida la dispersión esperada de los valores muestrales, un criterio sencillo y adecuado para incorporar este elemento adicional en el proceso de ajuste consiste en eliminar aquellos valores cuya dispersión sea superior a un cierto nivel considerado tolerable. *Para ello debe establecerse un intervalo en torno a la probabilidad típica (criterios tipo A) o valor típico (criterios tipo B) elegidos, y calcular cuál es la probabilidad de que la probabilidad o valor típico observado caiga dentro de aquel intervalo. Si la probabilidad de inclusión en el intervalo es inferior a un valor dado, el punto muestral se excluye del ajuste.*

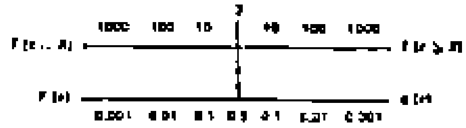
Esta forma de establecer intervalos para el control de los ajustes parte del criterio recíproco al que da origen a los intervalos de confianza que se utilizan corrientemente en el contraste de la función de distribución a partir de la muestra. En estos últimos se parte de un cierto valor de la frecuencia de presentación o nivel de confianza, a partir del cual se calcula cuál es la amplitud correspondiente de la dispersión del valor a contrastar. En cambio, para los ajustes nos interesa calcular la frecuencia de presentación de los valores observados dentro de un intervalo fijado previamente. *Denominaremos a estos últimos "intervalos de precisión", para diferenciarlos de los "intervalos de confianza".* Aquí cobra sentido el "juicio ingenieril", que debe decidir en primer lugar la amplitud del intervalo. Obviamente, esta decisión es función entre otras cosas del grado de precisión que requiere el proceso de diseño en que van a ser empleados los resultados. A este respecto puede indicarse que, en general, las aplicaciones en ingeniería de estos cálculos exigen que los intervalos de precisión sean definidos no en términos absolutos sino relativos al valor o probabilidad escogida. En el caso de que se escoja el valor de la variable, la diferencia entre un intervalo rígido y el establecido relativamente al valor llega a ser considerable si el recorrido muestral es largo. En determinados fenómenos climatológicos, el recorrido puede ser tan largo como para abarcar más de un orden de magnitud (ver, por ejemplo, los casos de máximas precipitaciones diarias en Madrid y mensual en Los Llanos, que se tratan en la referencia [5]). Si, en cambio, el parámetro a evaluar es la probabilidad, es evidente que un intervalo rígido de por ejemplo ± 0,05, que en un caso determinado puede resultar apto para el punto central de probabilidad 0,50, resulta totalmente inadecuado para un punto en la cola superior de la muestra con probabilidad 0,01. *Por tanto, los intervalos de precisión convenientes para su utilización en ingeniería son de forma ($Z \pm k Z$), siendo Z el parámetro a estimar y k la fracción del mismo que mide la dispersión considerada tolerable.*

Una vez definida la amplitud del intervalo, el "juicio ingenieril" vuelve a intervenir en la adopción del nivel mínimo de probabilidad de inclusión, que determina la aceptación o rechazo de los puntos muestrales. En esta decisión debe jugar un papel primordial la importancia atribuida al fallo de la predicción extremal realizada.

4. *En cuanto al método a usar para ajustar los puntos muestrales aceptados, es lógico pensar que el adecuado sea aquel que minimice las diferencias, en términos relativos, del parámetro escogido para establecer los intervalos de precisión.*

5. Las funciones de distribución, y entre ellas las extremales, constan de una cola superior y otra inferior cuyos extremos evalúan las frecuencias de presentación de valores raros, por ser muy grandes o muy reducidos. El valor de la probabilidad que caracteriza a la presentación de aquellos valores raros lo da, en el caso de valores grandes, la probabilidad de que ocurra uno igual o mayor [$q(x) = 1 - F(x)$], y para los valores pequeños la probabilidad de que ocurra uno igual o menor aún ($F(x)$). En el caso concreto de la función de distribución extremal, la escala de períodos de retorno sólo es significativa cuando se entiende de la misma forma: Para los valores grandes, el intervalo medio entre ocurrencias de un valor igual o mayor $\left[T(x \geq X) = \frac{1}{q(x)} \right]$, y para los valores pequeños el intervalo medio entre ocurrencias de un valor igual o menor $\left[T(x \leq X) = \frac{1}{F(x)} \right]$.

Existe, por tanto, una simetría en cuanto a la utilización de ambas ramas de las distribuciones, que puede esquematizarse como sigue:



Ambas ramas tienen su interés particular en ingeniería, si bien la superior es la que se utiliza con mayor frecuencia en el caso de funciones de distribución extremales de valores máximos.

La correcta evaluación de la función de distribución (extremal o cualquier otra) requiere que en el caso de que se decida establecer intervalos de probabilidad para los ajustes, el valor de la probabilidad que debe utilizarse sea $q(x)$ en la rama superior, y $F(x)$ en la inferior. Esto es groseramente equivalente a establecer intervalos para los períodos de retorno correspondientes a ambas ramas. En la rama superior:

Intervalo de $q(x)$:

$$(1-k)\,q(x) \leq q(x) \leq (1+k)\,q(x)$$

En términos de $T(x)$, este intervalo es:

$$\frac{1}{1-k}\,T(x) \geq T(x) \geq \frac{1}{1+k}\,T(x)$$

Para intervalos estrechos existen las siguientes equivalencias aproximadas:

$$\frac{1}{1+k} \xrightarrow{(k \ll 1)} 1-k$$

$$\frac{1}{1-k} \xrightarrow{(k \ll 1)} 1+k$$

Con lo cual el intervalo de períodos de retorno queda:

$$(1-k)\,T(x) \leq T(x) \leq (1+k)\,T(x)$$

Es decir, el mismo que el de la probabilidad (en términos relativos), con aproximación fijada por la magnitud de k. Lo mismo puede establecerse para la rama inferior, utilizando $F(x)$.

Puede verse que, si para establecer los intervalos se utilizara en toda la distribución la probabilidad $F(x)$ o $q(x)$), el extremo superior (o inferior) resultaría inadecuadamente tratado. Supongamos que se utiliza $F(x)$ estableciéndose un intervalo $F(x) \pm 0,1\,F(x)$. En el centro de la distribución $F(x) = 0,5$ el intervalo sería $0,45 \le F(x) \le 0,55$, que en términos de periodos de retorno (de la rama superior), es $1,81 \le T(x) \le 2,22$. Es un intervalo aceptable. En un punto de la rama inferior con $T = 50$ ($F(x) = 0,02$), el intervalo correspondiente es $0,018 \le F(x) \le 0,022$, que en periodos de retorno es $45,45 \le T(x) \le 55,55$. En cambio, en la rama superior, para un punto de $T = 50$ ($F(x) = 0,98$) el intervalo es $0,882 \le F(x) \le 1$ y en periodos de retorno $8,47 \le T(x) \le \infty$, un resultado de amplitud tan grande que carece de significado.

6. Una de las primeras observaciones que se desprende del establecimiento de los intervalos de precisión para los ajustes, es que la muestra *no proporciona una estima de fiabilidad uniforme para la función de distribución a lo largo de todo su recorrido*. Cualitativamente esto puede comprenderse de forma sencilla y directa estableciendo una analogía con los ejemplos familiares de una moneda y un dado. Supongamos que deseamos obtener estimas muestrales de la probabilidad de "cara" en la moneda y "uno" en el dado. Es evidente que si se desea obtener estimas de fiabilidad equivalente en ambos casos, es necesario tirar el dado un número superior de veces a la moneda. Si el número de tiradas es el mismo para dado y moneda, la fiabilidad de la estima producida por esta última es superior a la del otro. La comparación puede establecerse en términos cuantitativos cuando se establece un determinado intervalo de precisión en términos relativos ($p \pm k\,p$). La obtención de "cara" en la moneda es asimilable al punto central ($F(x) = q(x) = 0,5$) de la distribución. La obtención de "uno" en el dado es asimilable a un punto de la rama superior (o inferior) de la función de distribución cuya probabilidad de ser superado (o de no ser superado) sea $q(x) = \frac{1}{6}$.

En el caso de un muestreo discreto y aleatorio (por ejemplo, una muestra extremal), el número de "tiradas" es constante para todos los valores muestrales e igual al tamaño N de la muestra obtenida. Admitiendo que en general los N valores muestrales pueden ser considerados estadísticamente independientes, podemos emplear la distribución binomial para cuantificar la fiabilidad de las estimas en términos de nivel de confianza. Según la distribución binomial, la probabilidad de obtener m presentaciones de un fenómeno en N pruebas, cuando la probabilidad de aparición en cada una de las pruebas es $p(x)$, viene dada por:

$$f(m)_x = \binom{N}{m} p(x)^m (1 - p(x))^{N-m}$$

Esta es, obviamente, la probabilidad de que la probabilidad observada en el muestreo sea $\frac{m}{N}$.

Con esta fórmula puede calcularse cuál es la probabilidad de que esa probabilidad observada se mantenga dentro de un intervalo suficientemente estrecho (a determinar arbitrariamente) en torno a la probabilidad real $p(x)$. Este es el "intervalo de precisión" que, según se indicó anteriormente, es conveniente definir en términos relativos. Para trabajar con la distribución completa vamos a prescindir por el momento de la estadística de valores ordenados y considerar simplemente la frecuencia con que es superado (o no) cualquier valor de la variable. $p(x)$ es, para la rama superior de la distribución, igual a $q(x)$ y para la rama inferior igual a $F(x)$. Establezcamos un intervalo de precisión de $p(x) \pm 0,2\,p(x)$. Supongamos un tamaño de muestra $N = 40$. Tomemos como términos de comparación el punto central de la distribución ($q(x) = F(x) = 0,5$) y uno de la rama superior $q(x) = 0,25$. Los intervalos de precisión correspondientes son $0,4 \le \frac{m}{N} \le 0,6$ y $0,2 \le \frac{m}{N} \le 0,3$. En términos del número m de presentaciones, los intervalos son $16 \le m \le 24$ y $8 \le m \le 12$. Las probabilidades de que la probabilidad observada caiga dentro del intervalo, según obtenidas a partir de la binomial, son respectivamente 0,846 y 0,679. Por tanto, la fiabilidad de la estima es considerablemente superior en el centro de la distribución. Los extremos de ambas colas de la distribución tienen fiabilidades muy reducidas. Por ejemplo, a un valor con $q(x) = 0,05$ (periodo de retorno = 20, o sea excedencias por término medio en las muestras de $N = 40$) corresponde un intervalo de precisión de $1,6 \le m \le 2,4$ que, como m sólo adopta valores enteros, se reduce a $m = 2$. La probabilidad de inclusión es sólo 0,278.

7. El uso de intervalos de precisión para los ajustes sólo asegura estrictamente el control de dispersión y probabilidad asociada dentro de la zona ajustada o recorrido muestral útil. En extrapolaciones cortas puede suponerse que estas condiciones se mantienen casi iguales, pero si se pretende alargar las extrapolaciones la dispersión de resultados depende también en forma importante de la longitud del recorrido muestral útil. Para iguales amplitudes del intervalo de precisión, las dispersiones resultantes en las extrapolaciones son mayores para longitudes menores del recorrido muestral útil. La magnitud de este efecto podría estimarse, como una primera aproximación, gráficamente en un papel probabilístico a partir del rectángulo que forman los intervalos de precisión a lo largo del recorrido muestral útil. Pero una cuantificación ajustada de las dispersiones debe

partir de la realización de análisis sistemáticos de grandes cantidades de muestras de diversos tamaños, para cada variable de interés (produciendo gráficos de tipo similar a las figuras 3 a 5, con criterios adecuados de ajuste) manteniendo independencia entre muestras). Esto no plantea dificultades especiales.

8. La utilización de los criterios expuestos en este apartado proporciona una solución racional a las indeterminaciones propias de la metodología convencional, que fueron descritas en los apartados anteriores. En particular:

A) *La reducida fiabilidad de las estimas producidas por los valores situados hacia ambos extremos de cada muestra extremal hace que, incluso si se acepta un valor relativamente bajo para el nivel de confianza que delimita la aceptación* de las estimas, aquellos extremos resulten rechazados. Por tanto, desaparece el problema planteado respecto a la inclusión en la muestra de sus últimos valores cuando se apartan marcadamente de la tendencia definida por el resto.

B) En el epígrafe 1 de este apartado se trata de la conveniencia de adoptar una u otra formulación para la frecuencia de representación, según matizaciones del propósito de la predicción extremal. Sin embargo, a la vista de que los valores muestrales correspondientes a los extremos superior e inferior de la muestra quedan eliminados del ajuste, nos encontramos con que los diferentes criterios descritos anteriormente producen fórmulas de frecuencia de representación cuyas diferencias son muy reducidas, despreciables en muchas aplicaciones prácticas. En la tabla 2 se comparan las mismas fórmulas que se compararon en el apartado anterior, ahora en la hipótesis $N = 50$ y para dos puntos representativos: el valor central de la muestra ($m = 25$) y el quinto mayor valor muestral ($m = 5$).

Como puede apreciarse, las diferencias son pequeñas aun para $m = 5$.

C) *El efecto de dispersión producido por los diversos grupos de valores obtenidos con diversas muestras queda controlado mediante el uso de los*

TABLA 2

AUTOR	EXPRESIÓN	P (probabilidad)	T (periodo de retorno)	P (probabilidad)	T (periodo de retorno)
Hazen	$\frac{m - \frac{1}{2}}{N}$	0,490	2,04	0,0900	11,11
Weibull-Gumbel	$\frac{m}{N + 1}$	0,490	2,04	0,0980	10,20
Johnson-Benard	$\frac{m - 0,3}{N + 0,4}$	0,490	2,04	0,0933	10,72
Blom (D. normal)	$\frac{m - \frac{3}{8}}{N + \frac{1}{4}}$	0,490	2,04	0,0920	10,86
Gringorten (D. doble exp.)	$\frac{m - 0,44}{N + 0,12}$	0,490	2,04	0,0910	10,99

intervalos de precisión y sus probabilidades correspondientes. Los niveles de confianza que se asignen como límites a estas últimas determinan que, para ciertos tamaños muestrales, la totalidad o casi totalidad de las muestras es rechazable. *Por tanto, determinados intervalos de precisión y niveles de confianza exigen ciertos tamaños muestrales mínimos.*

Podemos denominar a la parte utilizable de la muestra en cada caso, recorrido muestral útil o efectivo. La aceptación de tamaños muestrales cuyo recorrido útil sea corto, depende de la habilidad requerida por el ingeniero y de la seguridad que se tenga sobre el tipo de función de distribución extremal correspondiente a la variable (y lugar) de que se trata. En efecto, recorridos útiles cortos pueden ser ajustados con gran facilidad por una variedad de funciones de distribución de tipos distintos sin diferencias significativas en la bondad de los ajustes.

9. *El uso de papeles probabilísticos para evaluar la bondad de los ajustes introduce factores adicionales en el análisis extremal.* Los procedimientos que establecen el ajuste a los puntos muestrales minimizando distancias absolutas sobre el papel probabilístico (uno de los cuales es el trazado de la recta según apreciación visual, quizá el procedimiento más extendido) no optimizan los ajustes en función de los parámetros que constituyen el objetivo del análisis extremal. Estos últimos son el valor de la variable o la probabilidad, mientras que las distancias medidas sobre el papel probabilístico vienen determinadas según las dos coordenadas cartesianas entre las que se establece la relación lineal que permite trazar la función de distribución según una línea recta. Estas son, según los casos (ver en Ref. 5, Apéndice sobre papeles probabilísticos), la variable reducida (en lugar de la probabilidad), y el valor de la variable, o su logaritmo, o el logaritmo de la variable trasladada (en lugar del valor de la variable).

En un papel exponencial, por ejemplo, la distancia entre $F(x) = 0$ y $F(x) = 0{,}1$ es casi la misma de la que existe entre $F(x) = 0{,}1$ y $F(x) = 0{,}2$. El uso estricto de los intervalos de precisión para la probabilidad en la cola inferior, tal como se indicaba en el epígrafe 3 de este apartado, pierde valor real en este caso, puesto que la amplitud de tales intervalos es, en términos de distancia sobre el papel, tan pequeña en la cola inferior, respecto a su amplitud en la cola superior, que en la práctica resulta poco sensible a importantes variaciones en la constante *k* que determina los intervalos.

Es verdad que otros papeles probabilísticos, como el Normal y Log-normal, extienden las escalas de probabilidad hacia ambas colas (que son además simétricas) mejorando la aplicabilidad de los intervalos de probabilidad. Son más corrientes, sin embargo, las escalas de probabilidad marcadamente asimétricas, como en el Doble Exponencial y el Weibull, donde las colas están desequilibradas en cuanto a sus pesos en el ajuste si se utilizan los intervalos indicados anteriormente.

Por tanto, cuando se desee utilizar papeles probabilísticos y ajustar a ojo (o minimizando distancias sobre el papel matemáticamente) la recta, es permisible, al establecer los intervalos de precisión, ampliar estos en alguna zona a la vista del espaciamiento que toman los dos ejes coordenados entre los que existe la relación lineal. La forma correcta de realizar esto está en establecer los intervalos de precisión en términos de la variable reducida, que es la que marca las distancias sobre el papel a lo largo de la escala de probabilidad.

Estas indicaciones flexibilizan la obtención de estimas de la distribución extremal cuando se desea usar un criterio de ajuste que minimice distancias sobre el papel probabilístico, pero no consiguen consistencia matemática en términos de la totalidad del procedimiento de predicción extremal. Esto último solo se obtiene utilizando el criterio indicado en el epígrafe 3 y cuando el ajuste empleado minimiza distancias en términos relativos al parámetro (o a los dos parámetros) relevante en cuanto a la predicción extremal, y que normalmente habrá servido de base para la construcción de los intervalos de precisión.

En definitiva, los ajustes en papel probabilístico minimizando distancias absolutas pueden ser en cierto grado racionalizados, pero los resultados de las estimas llevan una cierta indeterminación matemática debido a la heterogeneidad con que los valores de los parámetros relevantes están distanciados sobre el papel. Esta es una deficiencia inherente al uso del papel probabilístico, y representa quizá el precio que se paga por la comodidad indiscutible que supone su empleo.

10. A la vista de lo indicado en el epígrafe anterior sobre la consistencia matemática propia del uso de papeles probabilísticos, puede pensarse en utilizar otros métodos aproximados más cómodos (por ejemplo, que sean independientes de la función de distribución) para realizar ajustes de muestras extremales sobre aquellos papeles. Un procedimiento que se va a exponer a continuación abandona la utilización de la teoría estadística de valores ordenados para emplear en su lugar la distribución binomial, emparejada con cierta hipótesis aproximada sobre la probabilidad correspondiente a la población que es considerada aceptable en teoría de muestreo. La decisión sobre la conveniencia de este método, como también del uso del papel probabilístico en general, depende del grado de precisión que el proyectista requiera en cada caso, bien por el tipo de obra a construir, bien por la precisión q[illegible] posee la muestra extremal disponible o (y) otros elementos que intervienen posteriormente en el proceso de cálculo.

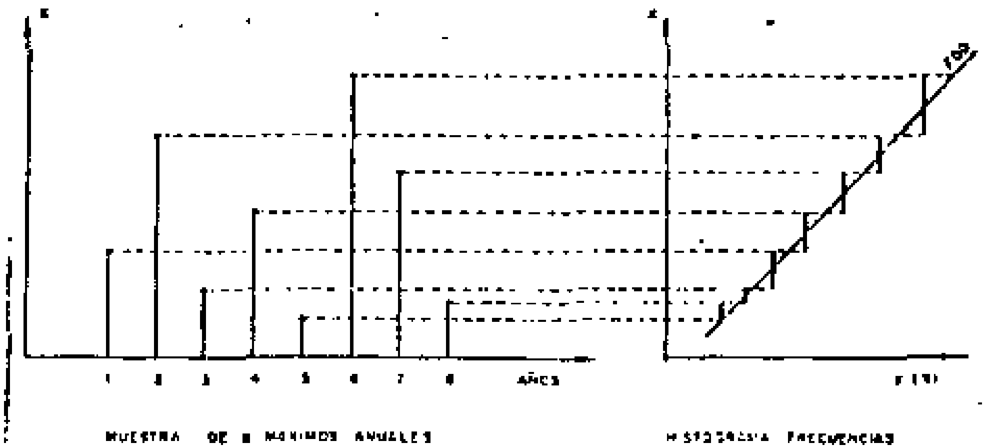


Figura 7.

Una muestra cualquiera de *N* valores independientes de los cuales x_1 es el menor y x_N el mayor, proporciona una estima de probabilidad de excedencia para cualquier valor de la variable comprendido entre x_1 y x_N. Esa distribución muestral puede ser representada mediante un histograma de frecuencias acumuladas. En la figura 7 se representa el histograma sobre un papel probabilístico que supuestamente corresponde a la distribución de la población.

En el supuesto de que las frecuencias acumuladas observadas estuviesen adecuadamente centradas en torno a las probabilidades de la población, la línea recta trazada a través del histograma sería una buena estima de la función de distribución buscada.

Los tramos horizontales del histograma son puntos de discontinuidad en la escala de probabilidad, donde esta salta de un valor a otro cuando la variable pasa por cada uno de los ocho máximos anuales. La teoría estadística de valores ordenados se ocupa de esos tramos horizontales, cuyos valores extremos de la probabilidad son los dos límites $\frac{m}{N}$, $\frac{m-1}{N}$ que establecía la condición número 2 de Gumbel. Las distintas formulaciones para la "frecuencia de representación" son distintos valores característicos de la probabilidad en esos puntos de discontinuidad, que son precisamente los utilizados para el ajuste [illegible] la distribución en esa metodología. En vez [illegible] podemos tomar el conjunto de segmentos [illegible] tales que constituyen la totalidad de información de probabilidad observada en la muestra. Para facilitar el ajuste puede utilizarse en lugar de los segmentos completos, la serie de puntos que se obtienen cuando se corta por una sucesión de valores de la variable con espaciamiento uniforme. En cuanto al criterio de aceptación o rechazo de los valores observados de la probabilidad, pueden establecerse, si se toma la probabilidad como variable relevante, *intervalos de precisión* para la probabilidad con ayuda de la distribución binomial de la misma forma que se realizó en el e[illegible] mostrado al final del epígrafe 5. Para ello [illegible] necesaba conocer de antemano cuál era la [illegible]bilidad real (de la población) para cada nivel de la variable. Esta no es conocida, pero en su lugar puede utilizarse la propia probabilidad observada en la muestra. Aproximaciones de este tipo son utilizadas en teoría estadística de muestreo con el fin de evaluar la fiabilidad de un tamaño de muestra determinado, para lo cual se necesita conocer previamente, al menos de forma aproximada, cuál es la magnitud de la probabilidad cuyo valor real va a estimarse más precisamente mediante muestreo.

Suponiendo que va a utilizarse papel probabilístico para el ajuste, y éste va a realizarse visualmente o minimizando matemáticamente distancias sobre el papel, los intervalos de precisión pueden ser establecidos en términos de la variable reducida, tal como se inició anteriormente.

Dentro del orden de exactitud en que nos estamos moviendo con este procedimiento aproximado, cuando (como es más frecuente) interese definir la precisión en términos del valor de la variable más que en términos de probabilidad o período de retorno, puede mantenerse el empleo de los *intervalos de precisión en términos de probabilidad* (para lo cual no se necesita en este procedimiento conocer la forma analítica de la función de distribución, al contrario que ocurre con los intervalos para la variable), pero determinando su amplitud en función de la incidencia que tienen estos en los valores que toma la variable. Esto puede ser estimado, aproximadamente, observando en el papel probabilístico la pendiente que toma una línea recta trazada a ojo a través del histograma muestral completo. Es decir, tomando otra vez la estima muestral completa por la distribución real de la población. Esta actitud de tomar muestreos de peso aún indeterminado como punto de partida para estimar fiabilidades es, como se indicó anteriormente, propia de teoría estadística de muestreo.

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