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COMMERCIAL IN CONFIDENCE

WAVE ENERGY STUDY

BREAKWATER DEVICE

2GW POWER STATION

REFERENCE DESIGN 1980

for

Department of Energy

Report No. PR22: WAVE 00

NATIONAL ENGINEERING LABORATORY

Department of Industry

Department of Industry
NATIONAL ENGINEERING LABORATORY
East Kilbride Glasgow Scotland
Telex 777888 Telephone 0355 2 (East Kilbride) 20222

103/80
PR22: WAVE 00
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WAVE ENERGY STUDY

NEL OSCILLATING WATER COLUMN
(WAVE PISTON)

BREAKWATER DEVICE
2GW POWER STATION

REFERENCE DESIGN
1980

ROXBURGH & PARTNERS

Mirren House
6 Maxwell Street
PAISLEY PA3 2AB

NATIONAL ENGINEERING
LABORATORY

East Kilbride
GLASGOW G75 OQU

CONTENTS

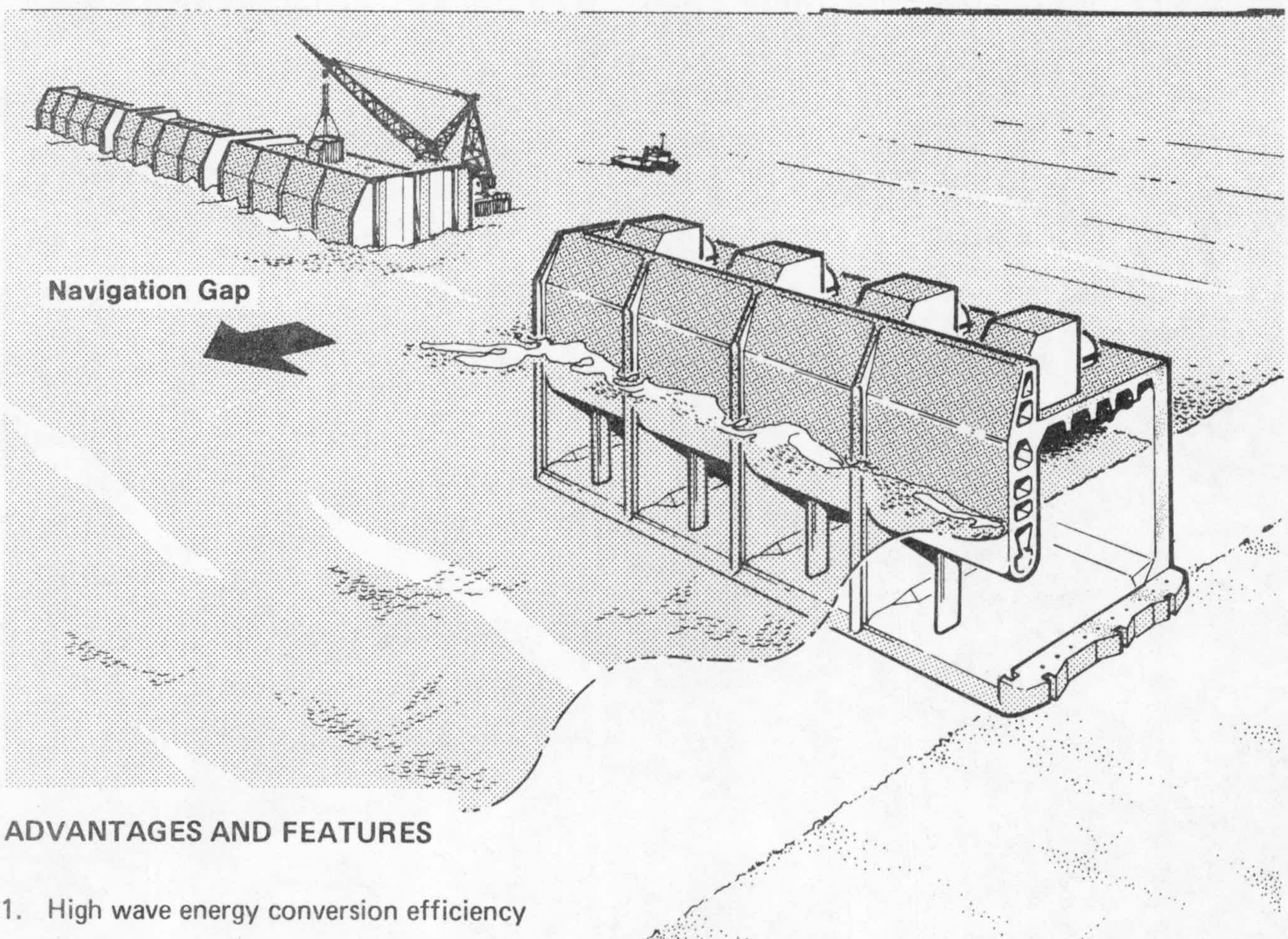
THE NEL WAVE PISTON executive summary

WHAT IS IT?

The NEL Wave Piston is a device that is being designed to convert energy from the waves into electrical power suitable for supply to the National Grid, although smaller scale applications can be designed for specific needs.

THE NEL BREAKWATER TYPE WAVE PISTON

(March 1980 reference design)

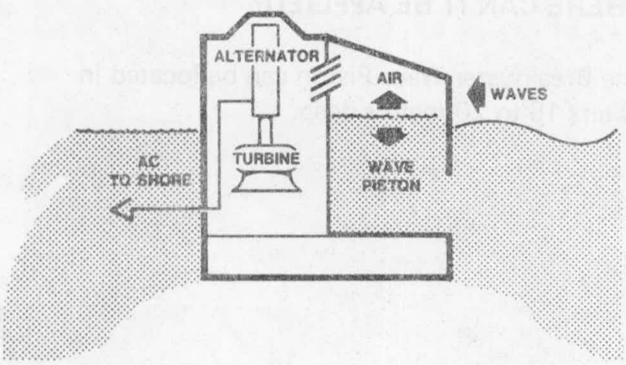


ADVANTAGES AND FEATURES

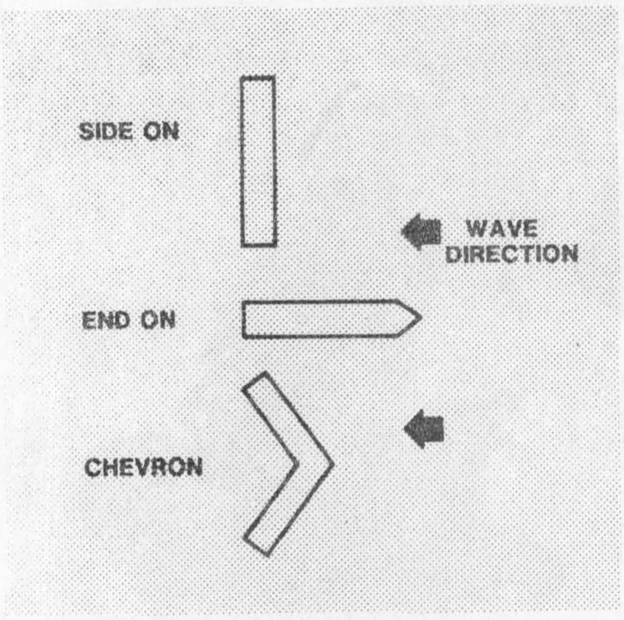
1. High wave energy conversion efficiency
2. Economic ratio of $\frac{\text{Overall Volume}}{\text{Piston Volume}}$
3. Low maintenance requirements
4. Plant modules can be removed for onshore maintenance
5. Can be constructed and installed using existing techniques
6. Practical and economic lengths can be positioned in continuous lines
7. Safety and security of a fixed structure
8. Offers some protection to inshore water
9. Eliminates the need for development of new mooring technology
10. Offers rapid development as a stepping stone within an overall strategy

HOW DOES IT WORK?

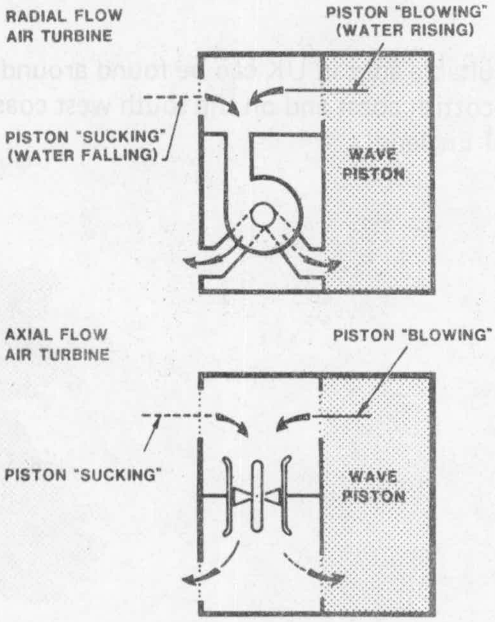
A bank of 'air over water' wave pistons is located within a concrete structure. The wave pistons move in response to the waves producing a reciprocating air flow which is rectified by louvre type valves into unidirectional flow. This air flow passes through an air turbine which drives an electric alternator. The electrical power produced is transmitted to the shore via submarine cables.



PLAN CONFIGURATION



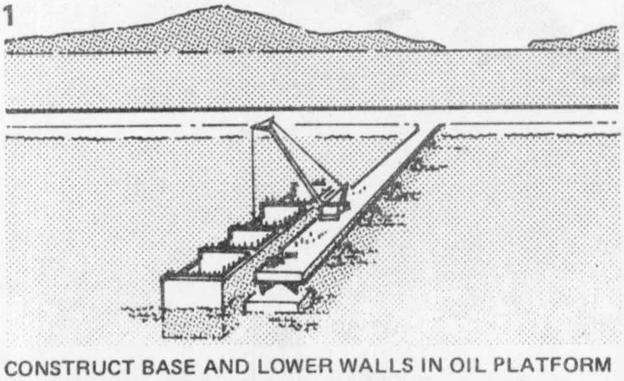
TURBINE TYPES (Plan Views)



CONSTRUCTION AND INSTALLATION

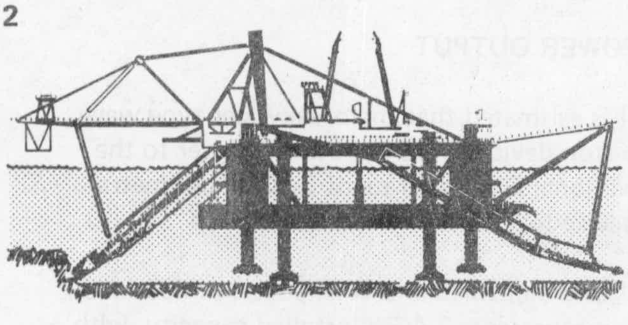
All work can be carried out using existing techniques

1



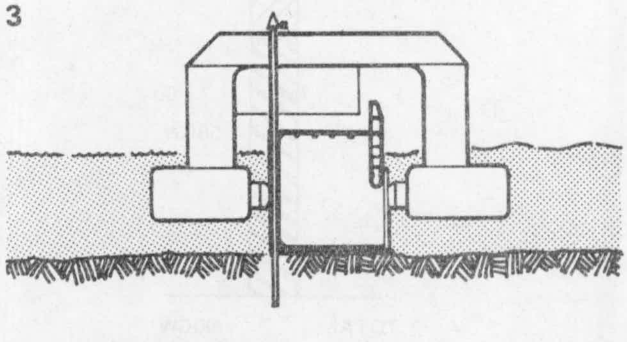
CONSTRUCT BASE AND LOWER WALLS IN OIL PLATFORM DRY DOCK (FIVE AVAILABLE IN FIRTH OF CLYDE) COMPLETE CONSTRUCTION AFLOAT.

2



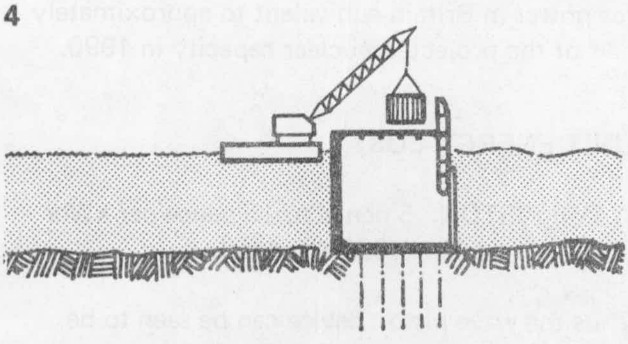
PREPARE SEA BED AT SITE.

3



TOW OUT AND BALLAST STRUCTURE ONTO PREPARED SEA BED, INSTALL ROCK ANCHORS.

4

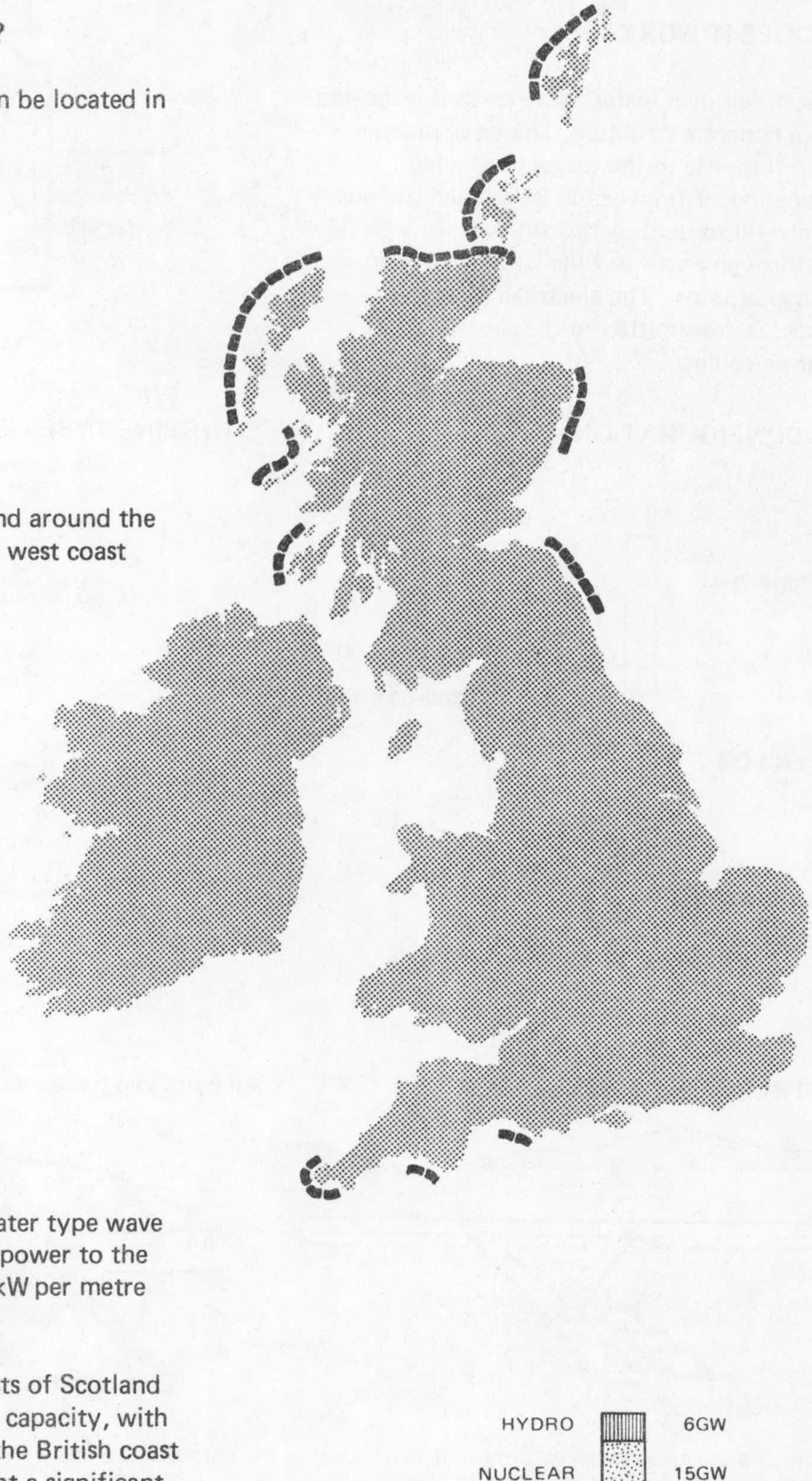


INSTALL PLANT MODULES FROM LANDWARD (SHELTERED) SIDE.

WHERE CAN IT BE APPLIED?

The Breakwater Wave Piston can be located in waters 15 to 20 meters deep.

Suitable sites in UK can be found around the Scottish coast and on the south west coast of England.



POWER OUTPUT

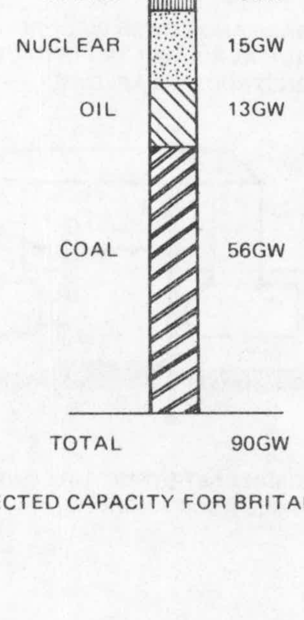
It is estimated that the breakwater type wave piston device could contribute power to the National Grid at a rate of 5-10kW per metre device length.

Along the North and West coasts of Scotland this represents 3-4GW installed capacity, with a total for all the sites around the British coast of 6-7GW. This would represent a significant contribution to the projected future demands for power in Britain equivalent to approximately half of the projected nuclear capacity in 1990.

UNIT ENERGY COST

WAVE PISTON. 5 pence to 15 pence per kWhr (1980 estimate).

Thus the wave piston device can be seen to be comparable with current conventional power generation costs for remote areas.

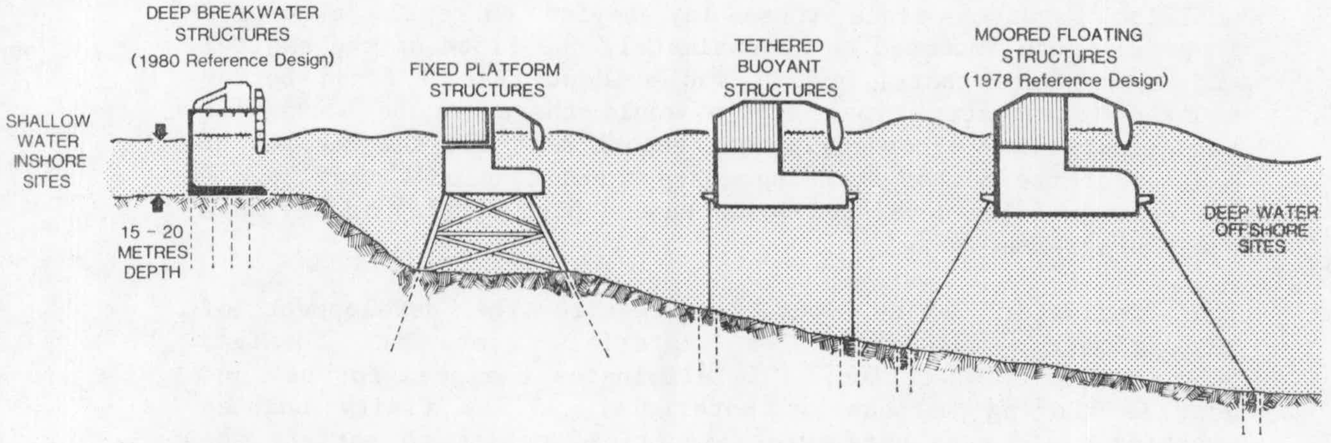


PROJECTED CAPACITY FOR BRITAIN 1990

THE NEXT STEPS

Further development work will be carried out on other designs in the wave piston family as well as work to improve the cost effectiveness of wave piston units.

THE FAMILY OF WAVE PISTON STRUCTURES



THE DEVICE TEAM

National Engineering Laboratory East Kilbride Glasgow, G75 0QU Scotland Telephone 035-52-20222 Telex 777888 Contact : George Elliot	—	Project Management Fundamental Research Tank Testing/Marine Trials Plant Design Power Output
Roxburgh and Partners Consulting Engineers Mirren House 6 Maxwell Street Paisley, PA3 2AB Scotland Telephone 041-889 0044 Telex 779684 Contact : Graham Roxburgh	—	Project Management Consultancy Structural Design Construction and Installation Methods Costing

SUB CONTRACTORS

London Offshore Consultants Ltd. London	—	Marine operations sub study
Colcrete Ltd. Rochester	—	Specialist advice on rock anchors and foundation aspects.
Scottish Marine Biological Association Oban	—	Specialist advice on marine biology and sea bed topography in Hebrides.

Issue 2 7.80

2. INTRODUCTION

In 1978 a report was submitted by National Engineering Laboratory to the Department of Energy on a floating version of the Oscillating Water Column Wave Energy Device (Reference 1). It was apparent that such a concept, while having simple and robust structure and plant, would have problems (in common with all other floating wave energy devices) with moorings and electrical umbilical systems since these lay beyond current technology. These elements amounted to approximately one fifth of the capital and maintenance costs, and it was evident that a fixed bottom mounted (breakwater type) device would therefore be worthy of investigation. Accordingly this report sets out the results of a study of the bottom standing concept and concludes that such a device when fully developed would have a part to play in any wave power programme.

The Breakwater Device does not require the development of revolutionary technology or materials for its design, construction or operation. It eliminates the need for new and untried mooring methods or materials. Its fixity enables existing rigid type underwater electrical cables to satisfy the power transmission requirements, therefore obviating one of the main problem areas. The structure is now designed to have a low material content. There is also some potential for increased power output using active valve control. Plant and structure maintenance is greatly simplified by fixing the structure, thereby providing both a stable base for onboard work and adjacent sheltered water for crane and maintenance vessels.

The design concept reported herein demonstrates both the practicability and viability of this approach and highlights areas where additional information and research would benefit the scheme.

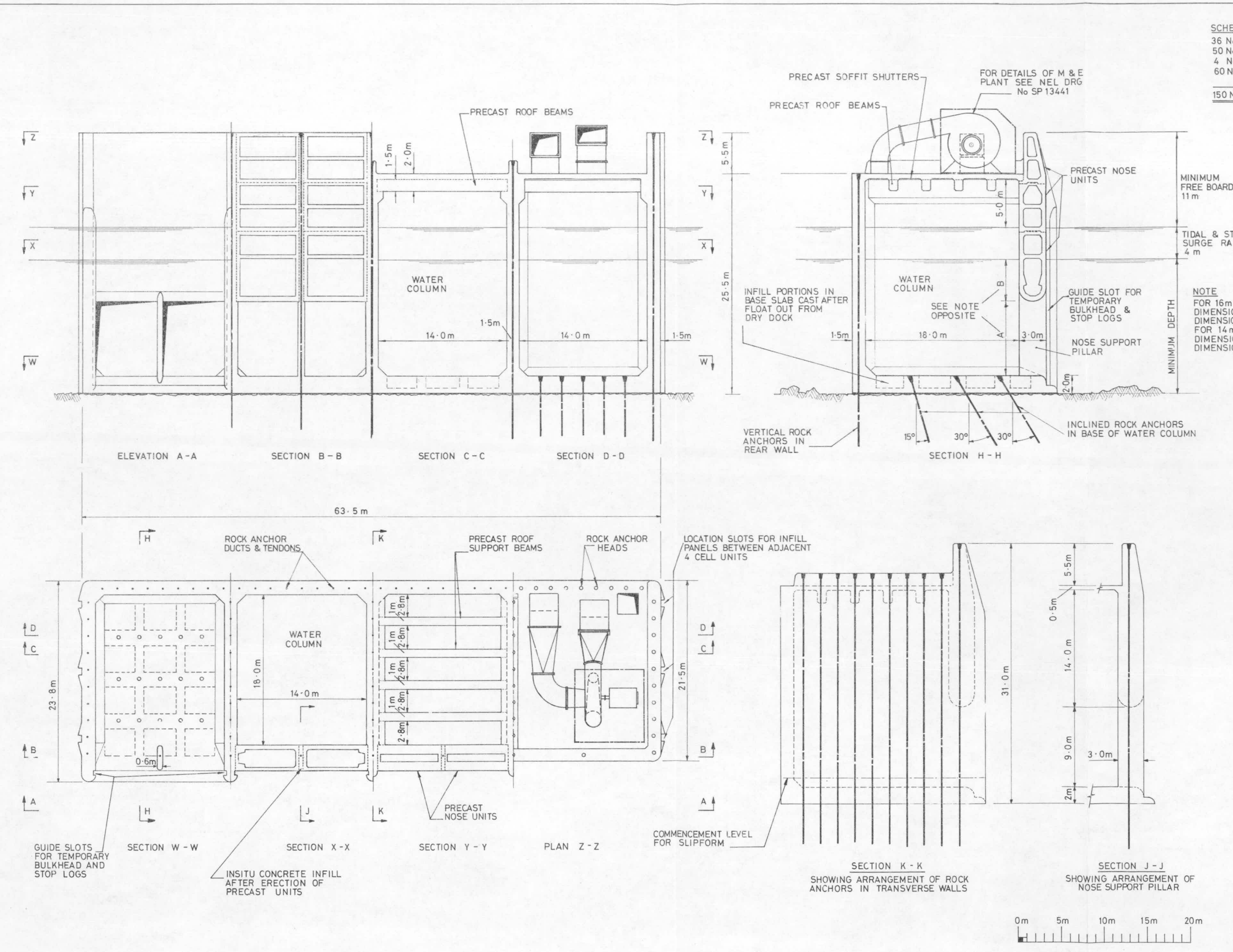
3. GENERAL DESCRIPTION

The Breakwater Device produces electrical energy from the waves using a primary air on water piston which converts the elliptical motion of the wave particles into a vertical oscillation of a water column. The resulting oscillating air flow from above the column is then converted into electrical power using a secondary system of ducts, rectifying valves, an air turbine and an alternator. A concrete structure provides an envelope for the primary system and a support platform for the secondary system plant. An artist's impression of the Breakwater Device is shown on page 6.

The primary piston structure is constructed in modules consisting of four water columns (see Roxburgh & Partners Drawing No. 3RD/101). Each column is 14m wide by 18m long with a depth of between 13 and 17m depending on the water level. The inlet to the column is 9m high. The air space above the water column is between 6 and 10m high. The overall size of each module is 24m wide by 63.5m long by 26m high. The dimensions given are for a design for water depths between 16 and 20m, although the device can be installed in shallower depths with some minor changes of dimension.

The secondary system (see layout on National Engineering Laboratory Drawing No. SP13441) works by taking the oscillating air flow from the column and converting it into unidirectional flow using four rectifying valves. These valves are in the form of cascade bends in order to minimise losses. The unidirectional pulsing air flow is then ducted into a radial flow type air turbine which has a runner diameter of approximately 2m. The turbine is then directly coupled to an alternator.

The resulting electrical power output from each alternator is fed into a single medium voltage direct current line via an isolating transformer and rectifier. Power from a number of columns is then combined, inverted and transmitted to the National Grid.



MK.	REVISION	By	Date

NATIONAL ENGINEERING LABORATORY

OSCILLATING WATER COLUMN (WAVE PISTON)

MARCH 1980
REFERENCE DESIGN
BREAKWATER TYPE

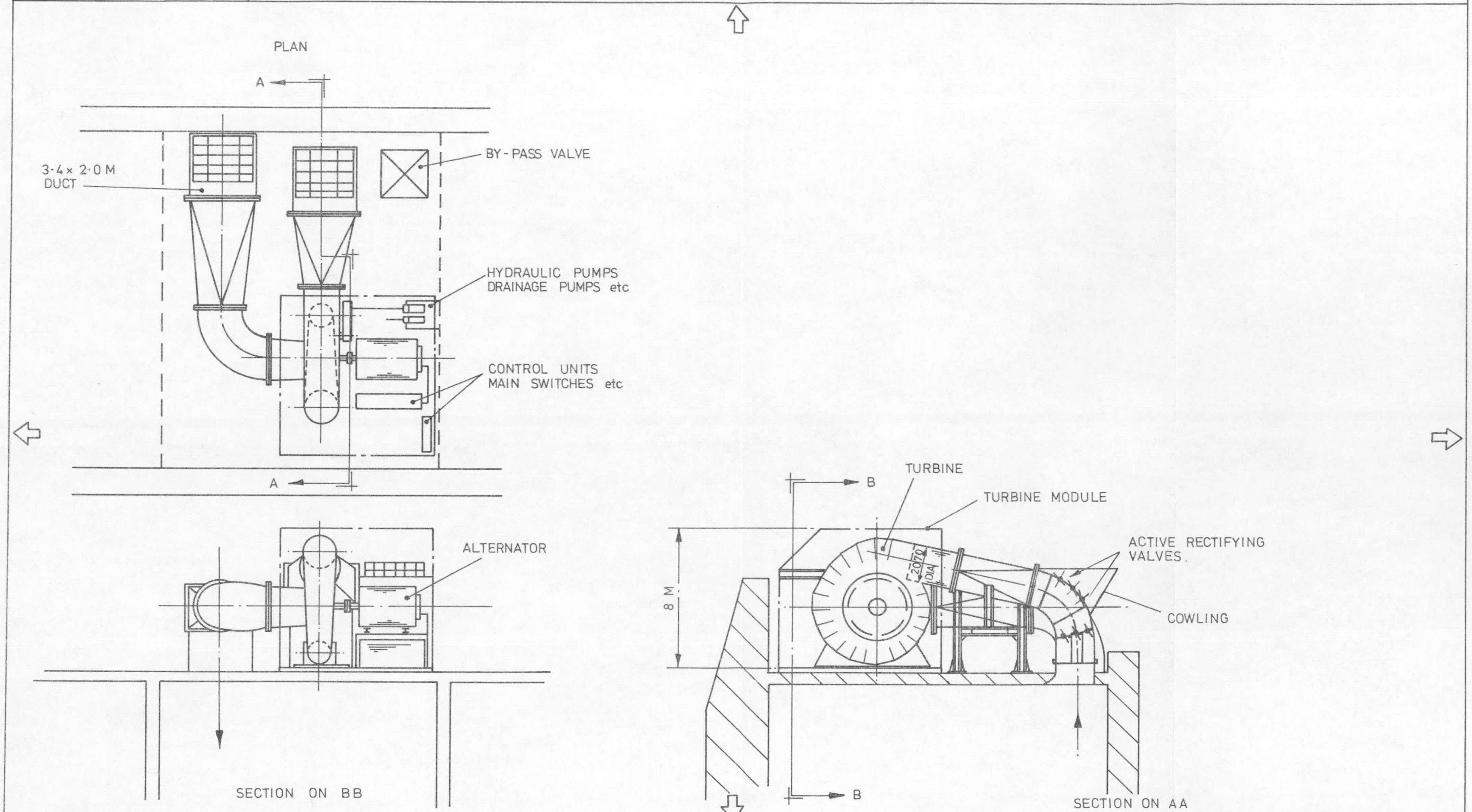
GENERAL ARRANGEMENT OF STRUCTURE

ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST.
PAISLEY SCOTLAND
Tel. 041 889 0044
Telex 779684 ROXBUR G
ABERDEEN LONDON

Job No	Drawing No	Revision
1678/71	3RD/101	

Scale 1:200	Date May 1980	
Drawn by	Passed by	Issued for
JM		MARCH 80 REPORT

DRAWING No SP 13441	DO NOT SCALE	DIMENSIONS IN mm UNLESS OTHERWISE STATED. UNTOLERANCED MACHINED DIMENSIONS TO ±	THIRD ANGLE PROJECTION.	SYMBOLS TO B.S. 308.



DATE	MODIFICATION	ZONE	SIGN	PART No			
				ASSY No			
				No OFF			
				MATERIAL			

DATE	PROJECT No		JOB No
APRIL '80			
DRN.	TRD.	CHK D.	APP.

TITLE
MECHANICAL & ELECTRICAL
PLANT LAYOUT - 1980
REF DESIGN

FRAME 267mm 390mm

NATIONAL ENGINEERING LABORATORY
EAST KILBRIDE, GLASGOW

DRAWING No
SP 13441

4. THE POWER GENERATION EQUIPMENT

4.1 Rectifying Valves

A schematic outline of the valves used for rectifying the air flow to the turbine is given in NEL Drawing No. SP13513. The valves are effectively cascade bends which assist in efficiently turning the air flows into and out of the turbine and air column. The closure flaps are always located on the downstream side of the cascades.

Actuation of the valves is carried out by either hydraulic or electrical means in response to control signals from the central operating system. When closed the flaps are sealed by a compliant element which is kept under pressure partly by the reverse air pressure and partly by the actuating system.

The internal dimensions of the valves are 3.3m wide by 2.2m deep giving a flow area of $7.26m^2$. Each bend consists of eight blades and nine 300mm deep flaps. The choice of number of flaps is dictated by considerations of inertia and actuating loads, with the greatest actuating load occurring when the flaps are held closed against the maximum column air pressure.

4.2 Air Turbine

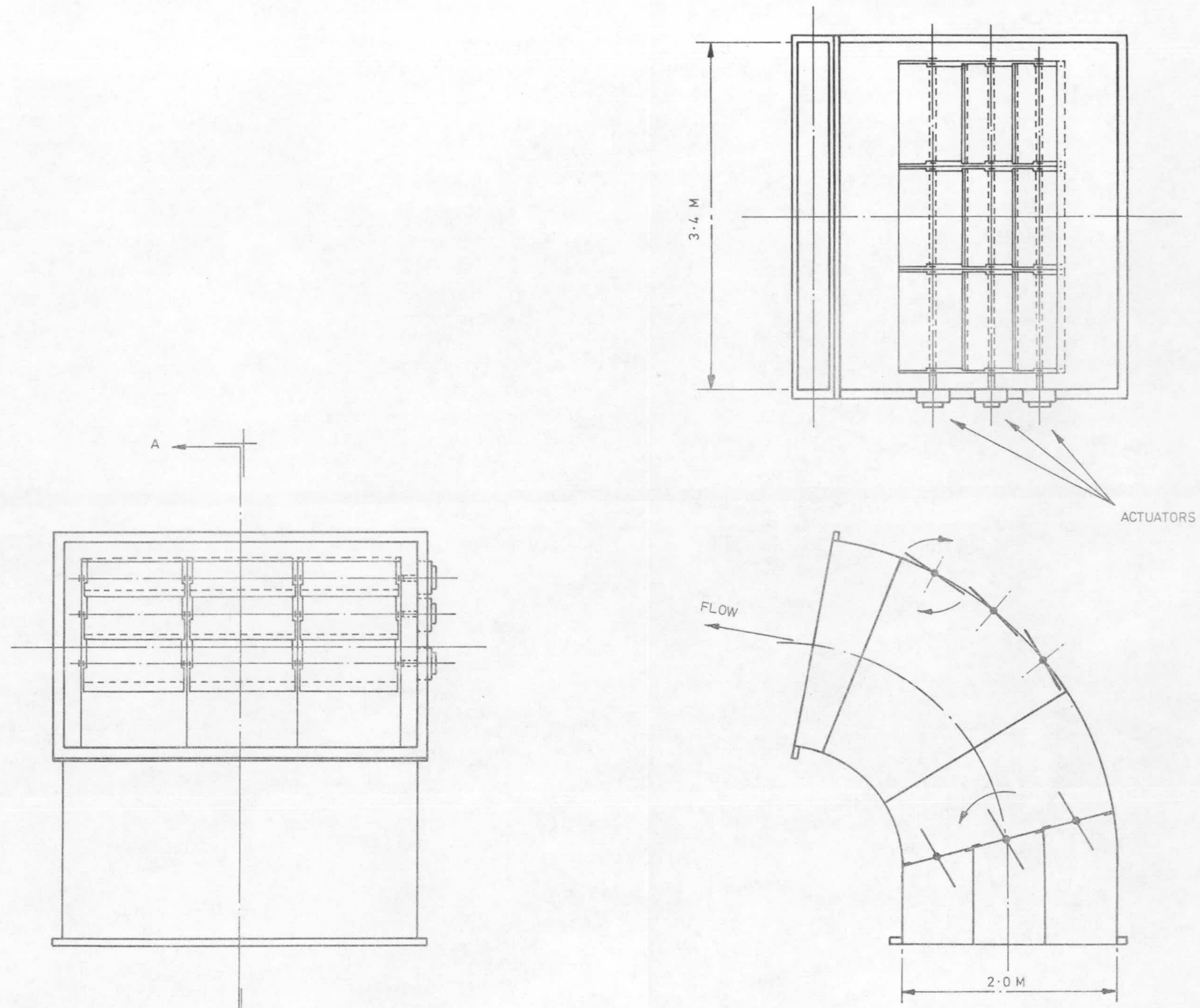
The air turbine is a radial flow machine with variable inlet guide vanes. This type was chosen because of its high efficiency, and because its head-flow characteristic provides optimum damping for the column. A photograph showing the major components of a laboratory scale model of this turbine is shown in Figure 4.1. The variable inlet guide vanes, in conjunction with the variable speed alternator, enhances the cycle efficiency by automatic adjustment of machine geometry and speed to suit the instantaneous flow rate. The vanes are controlled by the central operating system in response to pressure sensors located in the pneumatic circuit.

It is envisaged that the ducting, guide vanes and runner for the turbine will be constructed in glass reinforced plastic. Stress levels in the runner will be low due to the modest design speed and head. Gyroscopic loads will not present difficulty.

The following data applies to the turbine:

Design head	467m of air
Design flow	$67m^3/sec$
Design power (output)	336 kW
Design speed	695rpm
Runner diameter	2.07m
Estimated runner weight	1300kg
Estimated runner radius of gyration	0.895m

DRAWING No. SP 13513	DO NOT SCALE.	DIMENSIONS IN	UNLESS OTHERWISE STATED. UNTOLERANCED MACHINED DIMENSIONS TO ±	THIRD ANGLE PROJECTION.	SYMBOLS TO B.S.308.



SECTION THRO' AA

DATE	MODIFICATION	ZONE	SIGN	PART No.	DATE	PROJECT No	JOB No	TITLE	NATIONAL ENGINEERING LABORATORY, EAST KILBRIDE, GLASGOW.
				ASSY No	JUNE '80	WAVE 02		SCHEMATIC DESIGN OF AN ACTIVE RECTIFYING VALVE	DRAWING No SP 13513
				No OFF	DRN.	TRD.	CHK'D. APP.		
				MATERIAL				FRAME 380 mm x 554 mm	

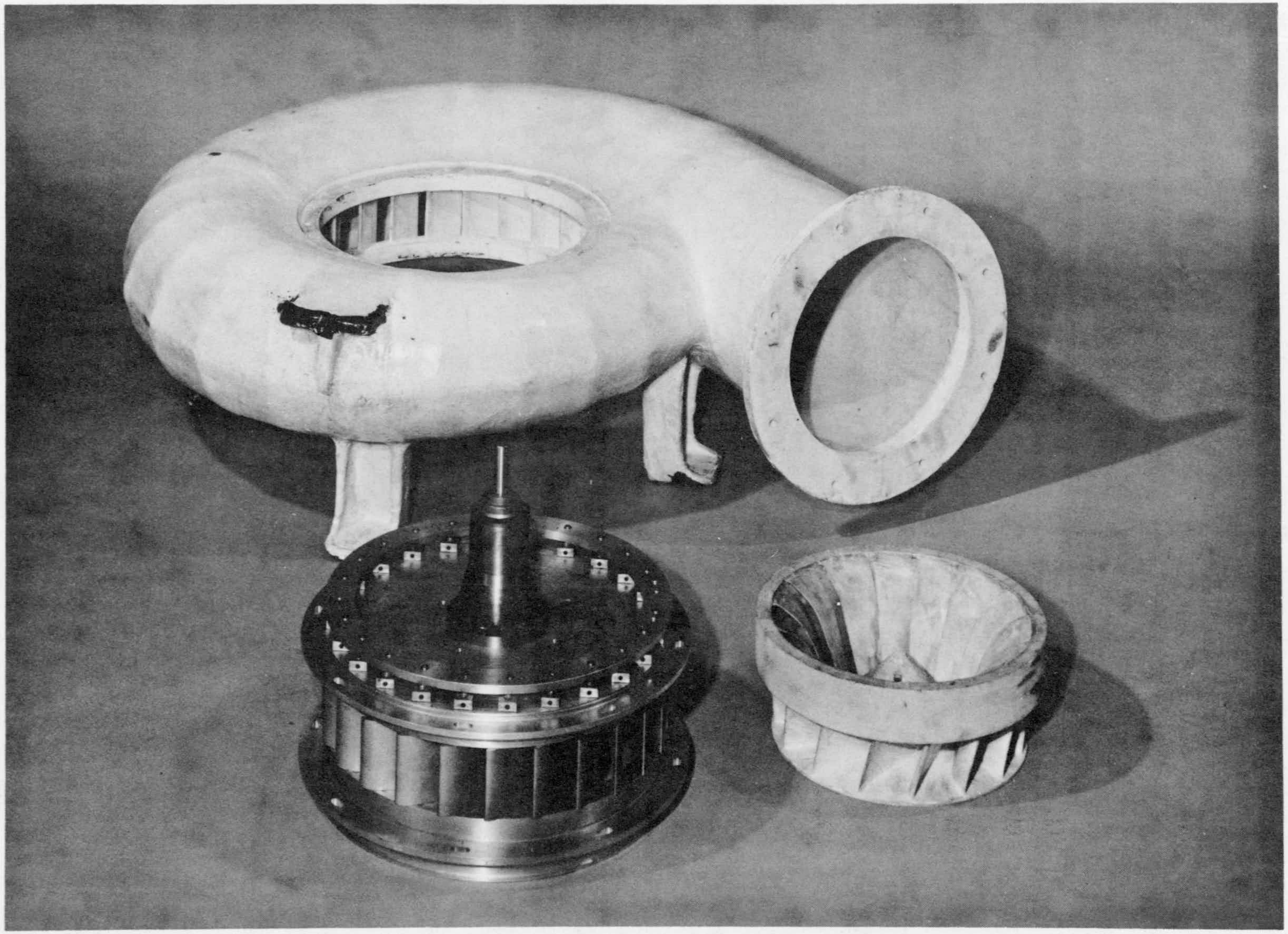


FIG. 4.1 THE MAIN COMPONENTS OF THE LABORATORY SCALE RADIAL TURBINE

4.3. Alternator

The alternator is a three phase, eight pole, brushless, synchronous AC type suitable for incorporation in the medium voltage DC transmission system proposed by Queens University Belfast (Reference 2). Energisation of the exciter field windings is provided from the generator output after rectification. Excitation for start up is provided by permanent magnet inserts in two or more of the exciter poles.

The average annual output of the alternator is calculated to be 125 to 142kW. Therefore, a maximum continuous rating of 1.2MW has been provided, although instantaneous power values may reach up to 2.4 MW. Maximum speed is approximately 1500 rpm.

The voltage/speed relationship of the alternator is governed by the exciter voltage in response to the central operating system.

4.4 Transformer and Rectifier

Associated with each alternator is an isolating transformer and rectifier bridge.

The transformer specification is dictated by the requirements of the transmission scheme (Reference 2). Therefore, it has a line voltage ratio of 1.1 to 1.5kV, with an insulation level from the primary windings and frame to the secondary windings of at least 33kV peak value.

Rectification is carried out by six transistor diode bridges rated at 1.2MW with a maximum input voltage of approximately 1.5kV (rms). No smoothing of the output is provided because the choppy voltages of several units are expected to be so continuously out of phase that the combined voltage within a group of units will be practically smooth.

4.5 Other Equipment

The following list covers the main items of ancillary equipment required for each individual water column.

M By-pass valves (2 off per column)
M Generator cooling system
M Lubrication system
E Central control system and cubicle
E Electrical control module
M Hydraulic actuation power packs
E Main switchgear
E Condition monitoring system
E Auxiliary supply transformers
E Auxiliary supply batteries

In addition, navigation and warning lights and buoys, foghorns, radar reflectors, maintenance equipment are provided for 4 cell units or lengths of device as appropriate.

The by-pass valves are incorporated in the roof of the air column and are actuated by the central operating system as necessary to maintain suitable column damping during maintenance operations when the turbine and power generation equipment is isolated from the wave piston.

All essential control and monitoring equipment is duplicated. The auxiliary batteries power standby equipment in the event of main power failure. Routine lubrication is carried out by an automatic force feed system. Drainage pumps are powered electrically and are operated as required by onboard level sensors. The navigation and warning lights operate continuously.

5. SELECTION OF REFERENCE SITE FOR POWER STATION

There are various locations around the coast of the United Kingdom where it may be possible to instal power stations using the Breakwater Device. These are shown on Figure 5.1.

The most significant location, due to the large incident wave energy and length of available coast line, is the West coast of Scotland. Within this general location, the Outer Hebridean islands have received most attention in the wave energy programme. This has included installation of wave rider buoys to determine wave climate and sonar surveys to determine sea bed topography.

Therefore, due to the relatively great amount of data available, it was decided to prepare the Reference Design for this specific location.

The water depth in which the Breakwater Device is intended to be installed is 16 to 20m. On the west side of the Outer Hebrides the 20m depth contour occurs at the following distances offshore:

Barra	1 km
South Uist	4½km
Benbecula	5 km
North Uist	1½km
Harris and Lewis	1 km

A first consideration in the more detailed site selection is the nature of the seabed and adjoining coast line. On the west sides of Barra, North and South Uist, Benbecula and part of Harris the seabed is rocky with a gentle offshore gradient, thereby providing reasonably suitable sites for the proposed Breakwater design. In addition, the coastlines have considerable stretches of sandy beaches which are preferred for the landing of the submarine power cables. A map showing some suitable locations is given in Fig. 5.2.

The preferred site in this region for installation of a prototype device is approximately 5km west of the small island of Orosay (57^o 8'N, 7^o 31'W) on the west side of South Uist. This area has already been subjected to preliminary surveys by Institute of Oceanographical Sciences and Scottish Marine Biological Association (see Section 6).

FIG 5:1 POSSIBLE BREAKWATER DEVICE POWER STATION SITES AROUND THE UNITED KINGDOM

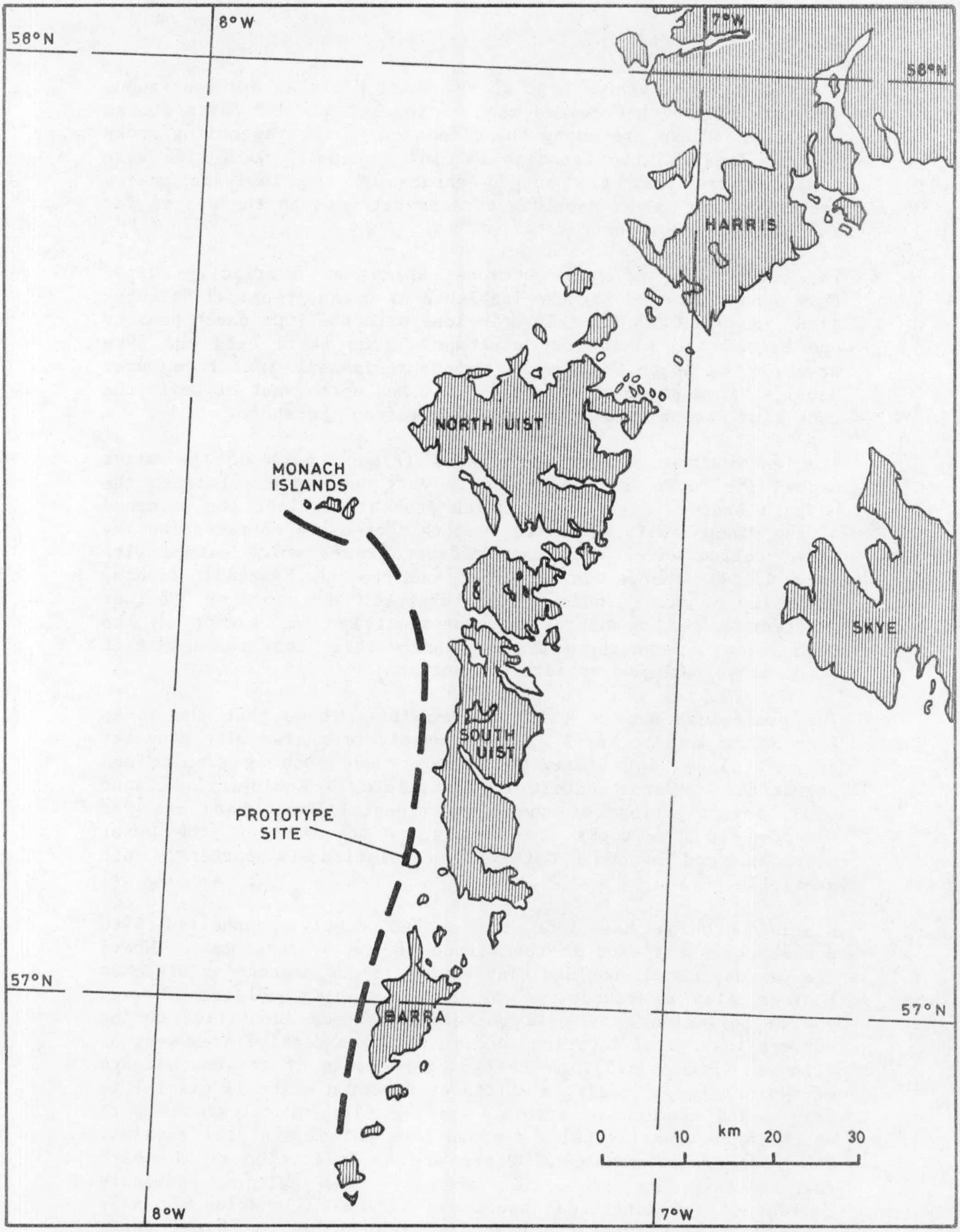


FIG 5:2 LOCATIONS OF PROPOSED SITE AT SOUTH UIST

6. SITE CONDITIONS AT SOUTH UIST

6.1 Geological

The rocks of the seabed west of the Outer Hebrides are continuous with the gneiss that covers much of the islands and parts of the mainland. These are among the oldest on earth, resembling rocks in Greenland and the Canadian Shield. Mapping with side scan sonar proves that the rugged nature of the Lewisian gneiss complex of the Outer Hebrides extends out beneath the sea as far as 75km west from the coast.

The outer limit of rock outcrops, which can be readily mapped from sonographs made by the Institute of Oceanographical Sciences (IOS) (Figure 6.1), roughly coincides with the 100m depth contour and extends at least 50km southwards from Barra Head and 40km west of the South Uist coast and approximately 50km from Outer Hebrides to west of St. Kilda. Off the north west of Lewis the rock limit is only between 2km and 5km from the shore.

Studies of the bathymetric charts (Figure 6.2) of the water around the Outer Hebrides indicate very deep water close to the eastern shores (Figure 6.6) which probably reflect the presence of the Minch Fault, while the western shores are characterised by very shallow water with many offshore reefs which extend with very little change in gradient out to the Rockall Trench. Admiralty charts indicate an absence of sand or boulder substrate, down to depths of approximately 75m, except in the ford areas between the islands. Below this limit the amount of sand, gravel and mud appears to increase.

The geological map of the Outer Hebrides shows that the rocks from Barra Head to North Uist are mainly of gneiss with a number of small local intrusions of igneous rocks such as granite and pegmatite. Unfortunately the Institute of Geological Science (IGS) investigations of the UK Continental Shelf have not yet extended to the rocks immediately to the west of the Outer Hebrides, and hence a detailed description is currently not available.

Glacial erosion has left the seabed deeply channelled with pockets of shell sand at the bottom of the depressions. Where ice was deposited, boulder clay debris is now present as winnowed boulder clay characterised by round stable boulders showing gneiss foliations. Small pockets have been identified during surveys in several locations and in a more extensive area west of Kildonan (Figure 6.5). The sand components of the beaches are of two origins, firstly, a siliceous fraction which is glacial in origin and chemically stable, and secondly, a calcareous part which is continually being renewed from animal skeletal remains. The fine cockle sands off Barra are an indication of the high calcium carbonate production area. The calcium carbonate content of the intertidal sands can vary with location but many beaches of South Uist can have a content of up to 70 per cent. Prominent features in the shelf sediments are barnacles, molluscs and echinoderms.

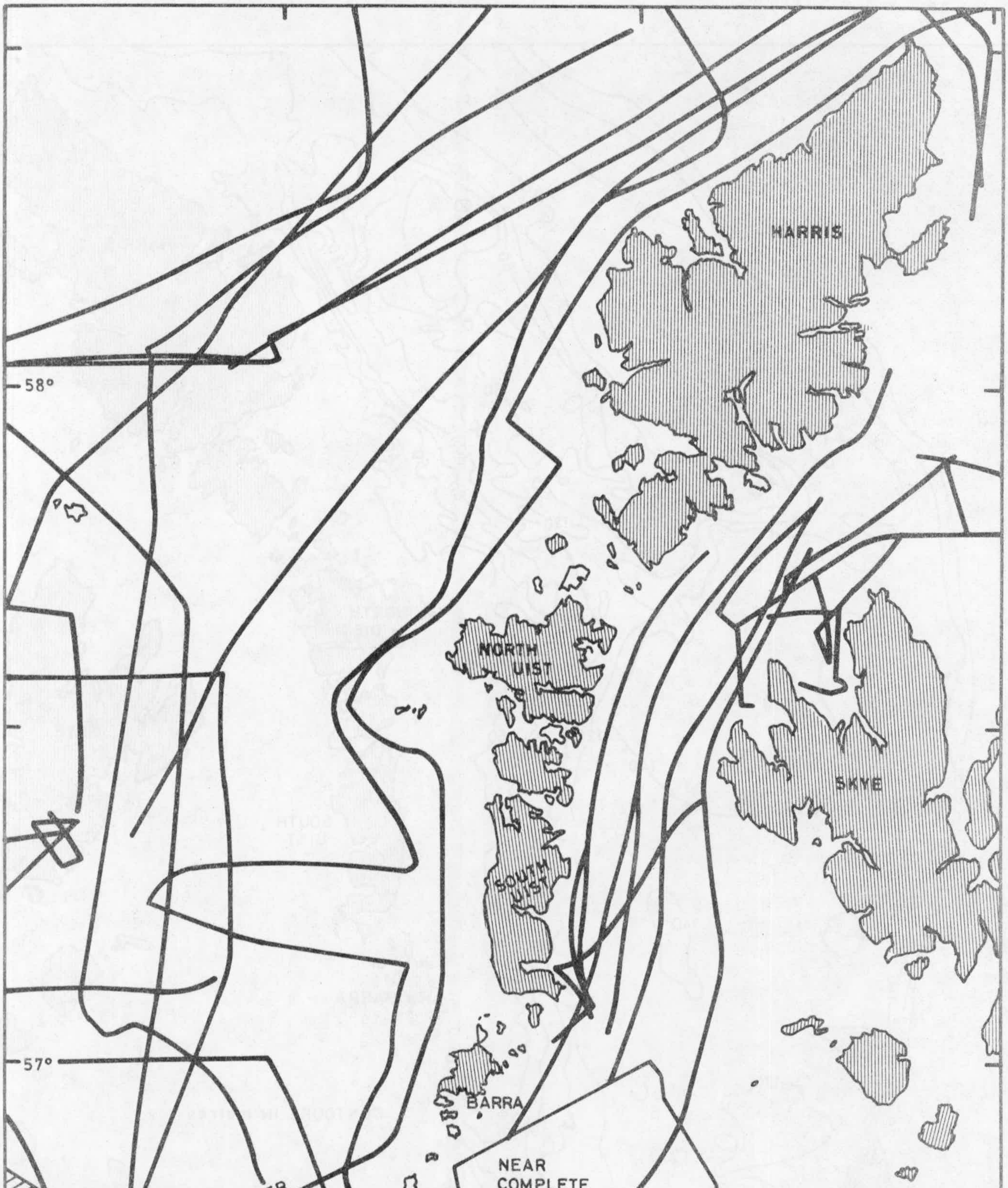


FIG 6:1 I.O.S. SIDE SCAN SONAR COVERAGE

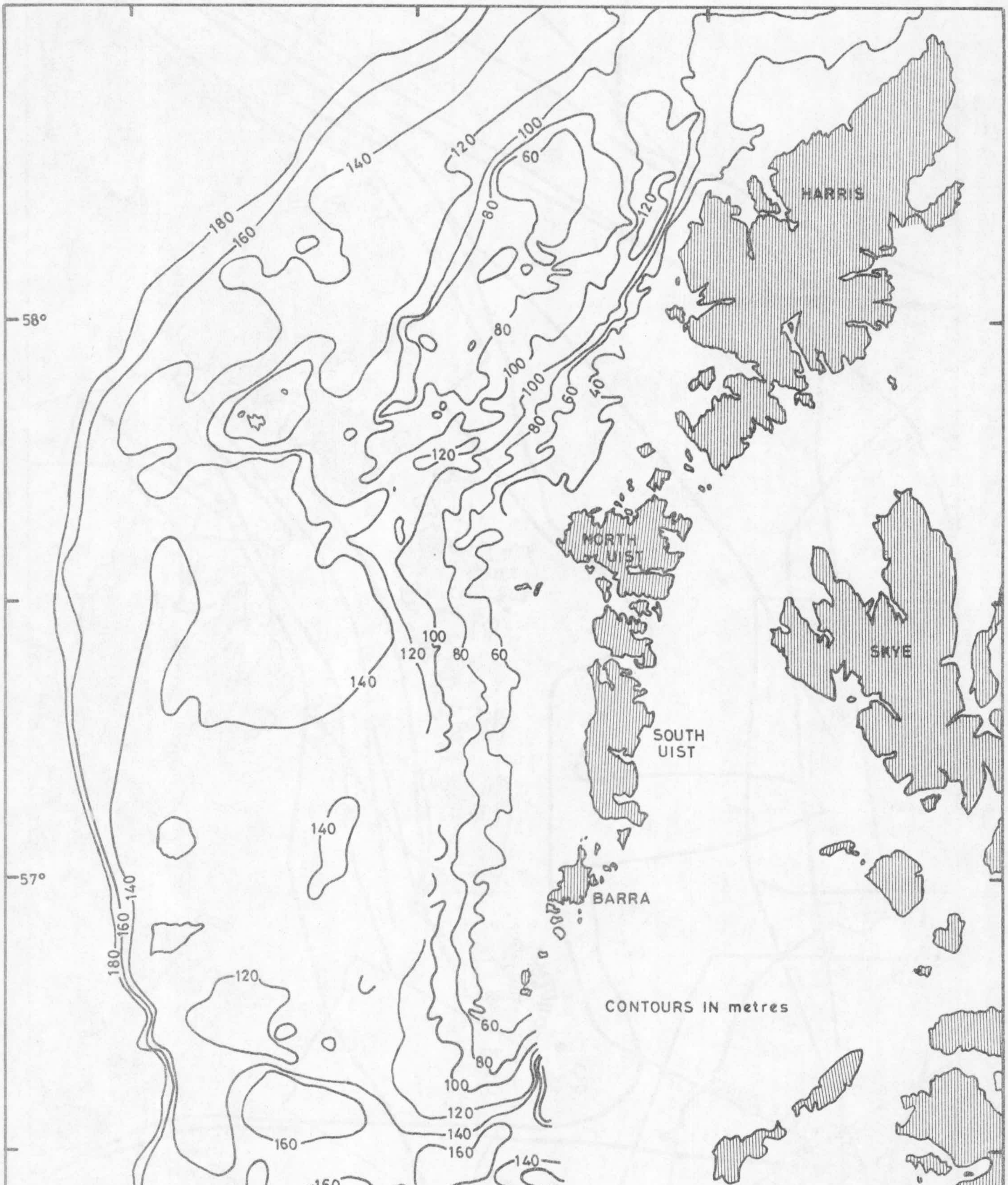


FIG 6:2 OUTER HEBRIDES BATHYMETRY

The solid geology of the seabed to the east of the Outer Hebrides, i.e. in the region of the South Minch, is somewhat different from the west coast. Here, the seabed is of mesozoic sediments and intrusions and some upper Palaeozoic sediments. The junction of these rocks with the relatively undisturbed sediments of the remainder of the basin is associated with the presence of numerous sills, whose disturbed nature appears to arise from the Minch Fault. An IGS borehole record 13km west south west of Rubha Hunish, Skye (Reference 5) shows there is approximately 34m depth of dark olive/grey boulder clay with limestone and dolerite boulders.

6.2 Topographical

Some preliminary topgraphical surveys have been carried out at the Outer Hebrides by the Institute of Oceanographical Sciences (IOS), the Scottish Marine Biological Association (SMBA) and the Institute of Geogical Sciences (IGS) (Figure 6.3). The IOS and SMBA surveys are more appropriate to possible Breakwater site locations on the west coast of South Uist, with the IGS surveys providing information for the laying of the submarine power transmission cables from South Uist to Skye.

The IOS survey (Reference 3) is in the region of their offshore wave rider buoy (see section 7.2) in depths of 15m to 40m (Figure 6.4). The underwater profile determined has similar characteristics to the dry land South Uist high ground glacial formations, with considerable lengths of fairly even ground interspaced with relatively sharp pinnacles or gullies. (NB The vertical scale of Figure 6.4 is exaggerated by a factor of five compared to the horizontal scale).

The survey carried out by SMBA (Reference 4) is in shallower water in the same general location as the first two legs of the IOS survey (Figure 6.3). Once again the sonar profile (Figure 6.5) highlights the deep glacial exploitation of the bed rock with major depressions every 60 to 100m. However, the expansion of a section of the sonar profile to equal horizontal and vertical scales shows that the gradients are relatively gentle.

An analysis of the various sonographs taken in the area shows that the more prominent rock reliefs, i.e. greater than 5m high, are generally orientated in a NE-SW and NW-SE criss-cross pattern. The resulting square, rectangular or diamond shaped lumps have slopes that are steeper at their lower edge than at their upper edge, giving an impression when viewed from above of a tray of rectangular loaves with very rough top crusts.

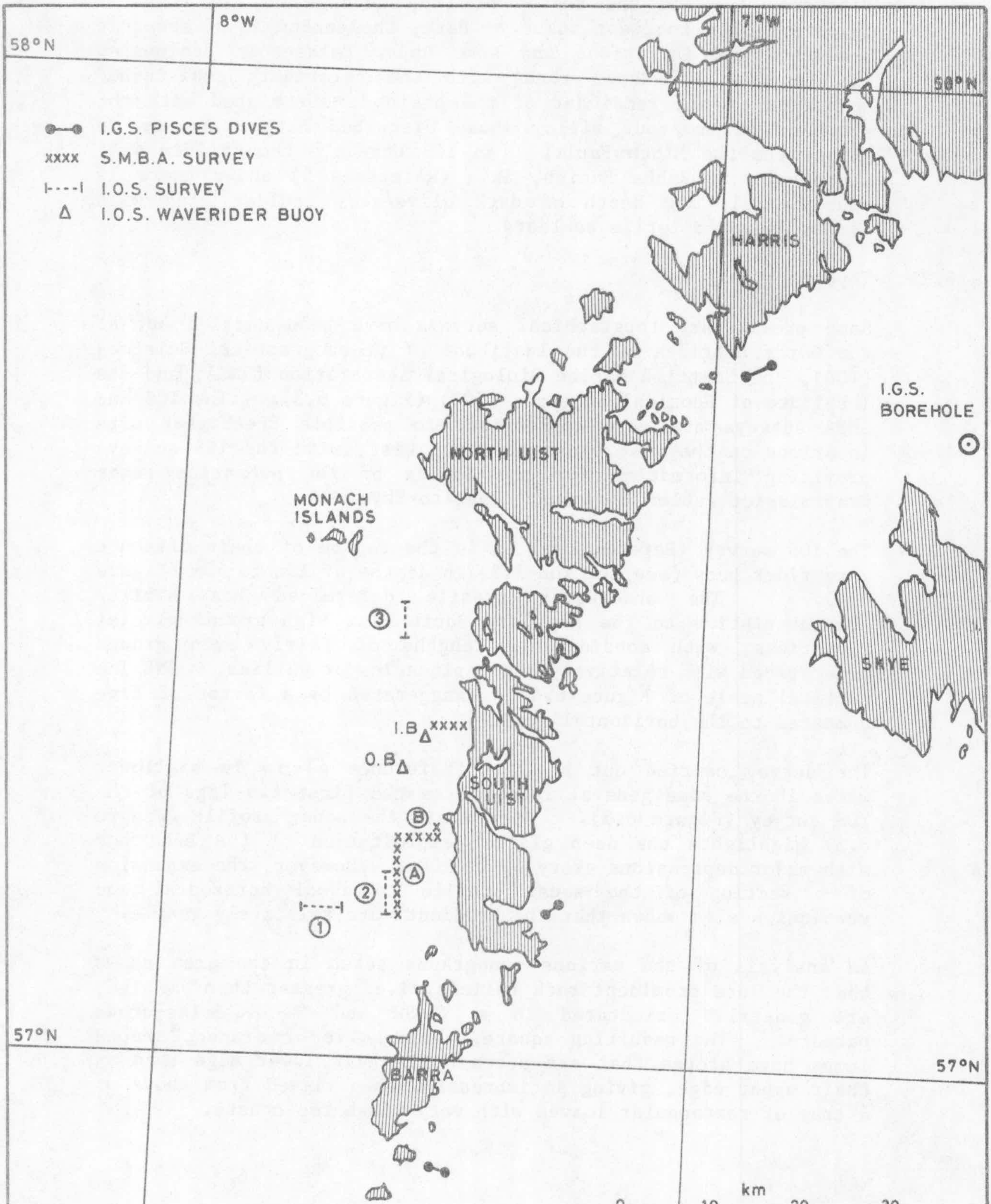


FIG 6:3 LOCATIONS OF SITE SURVEYS

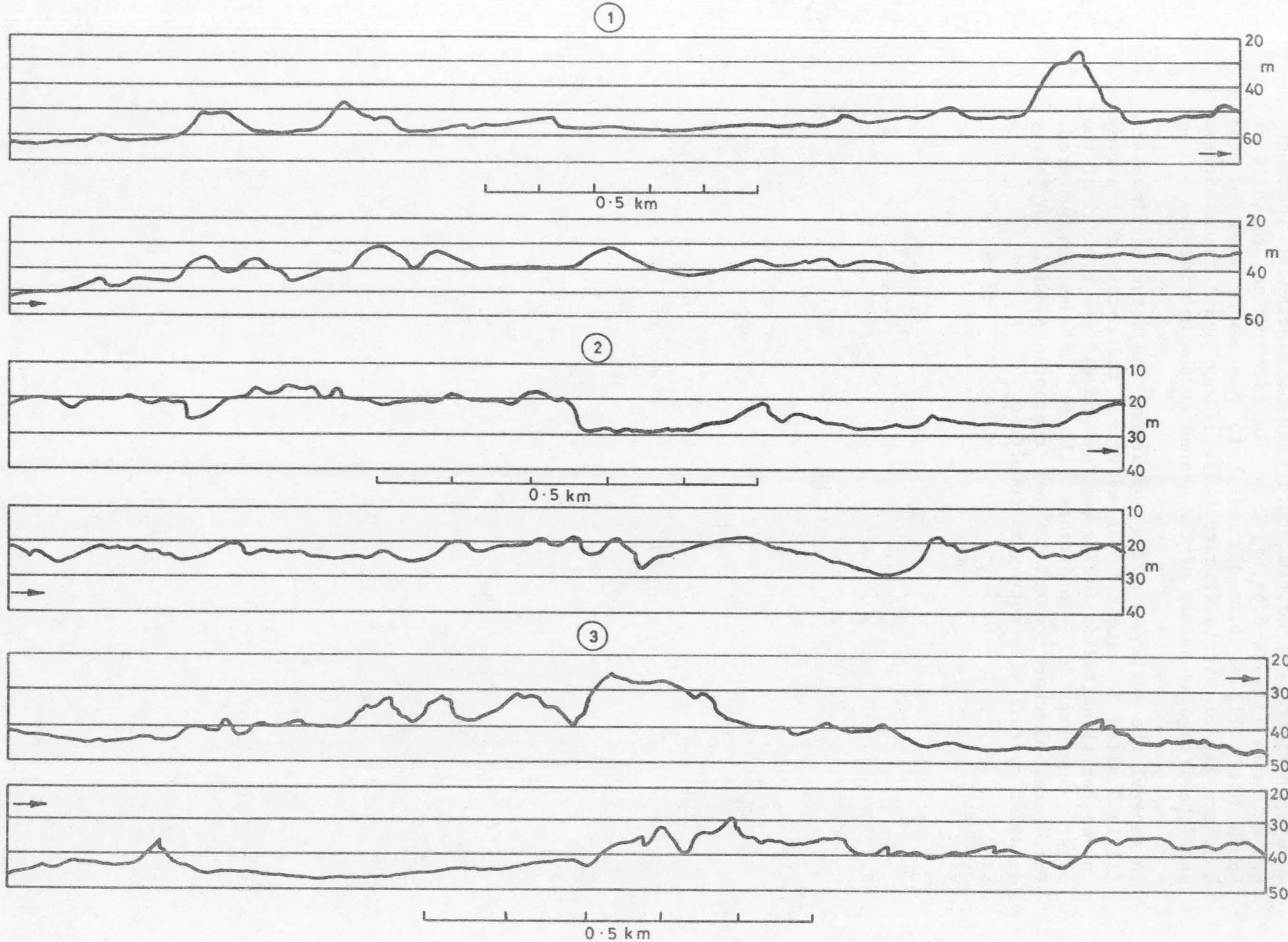
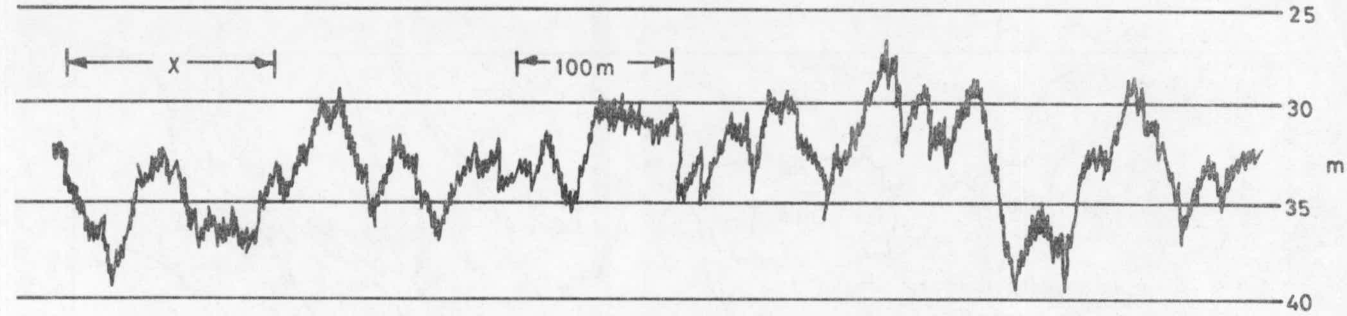


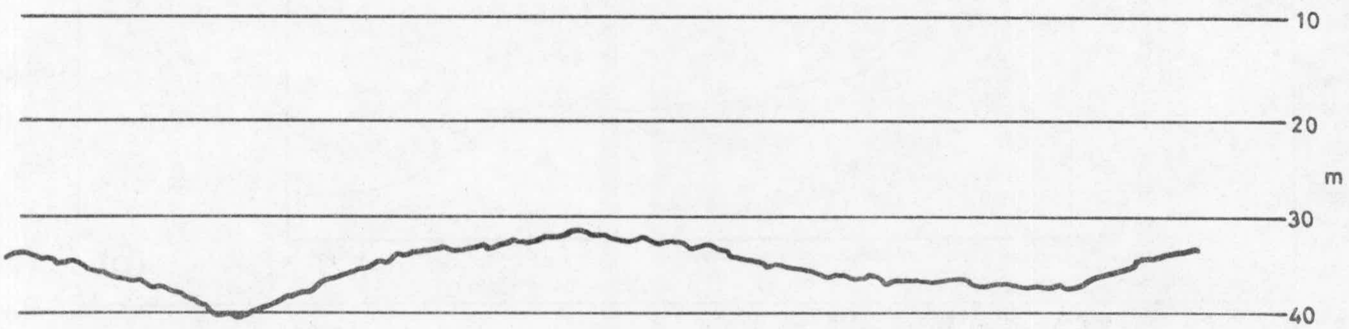
FIG 6:4 I.O.S SURVEY (TOPOGRAPHY)

The IGS surveys were made in connection with dives by their manned submersible vessel 'Pisces' at three locations on the east side of Barra and South Uist (Reference 6). The three locations were approximately 2 to 6km south east of Sandray at the south end of Barra (Figure 6.6a,b), 2 to 5km east of Loch Boisdale at South Uist (Figure 6.6c,d) and 3 to 8km south east of the southern tip of Harris (Figure 6.7). Once again these surveys show the irregular nature of the seabed at the Hebrides.

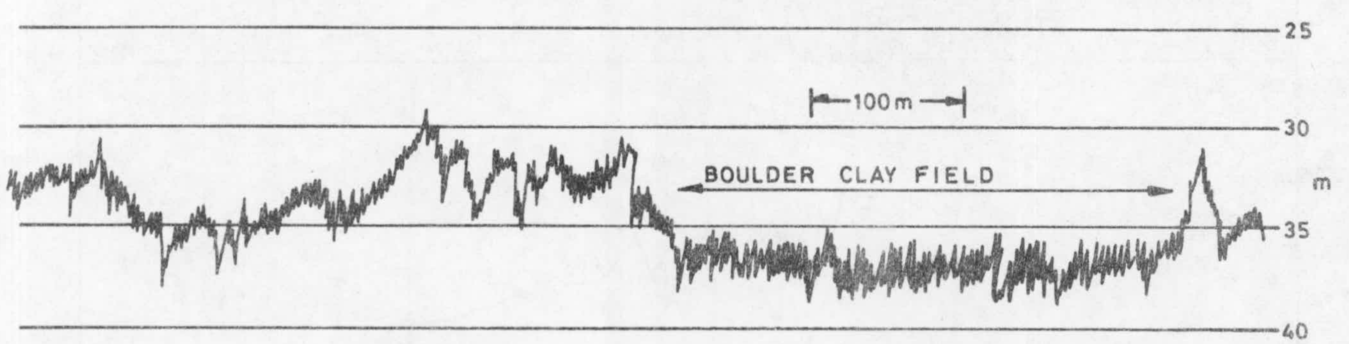
In general the surveys carried out around the Outer Hebrides are fairly limited in extent. In addition, the portions of survey chosen for illustration in Figures 6.4 and 6.5 are typical of the worst topographical conditions likely to be encountered. Therefore, further comment is not appropriate until a fairly detailed overall site survey is completed.



PART OF LEG (A)



SECTION X EXPANDED



PART OF LEG (B)

FIG 6:5 S.M.B.A. SURVEY (TOPOGRAPHY)

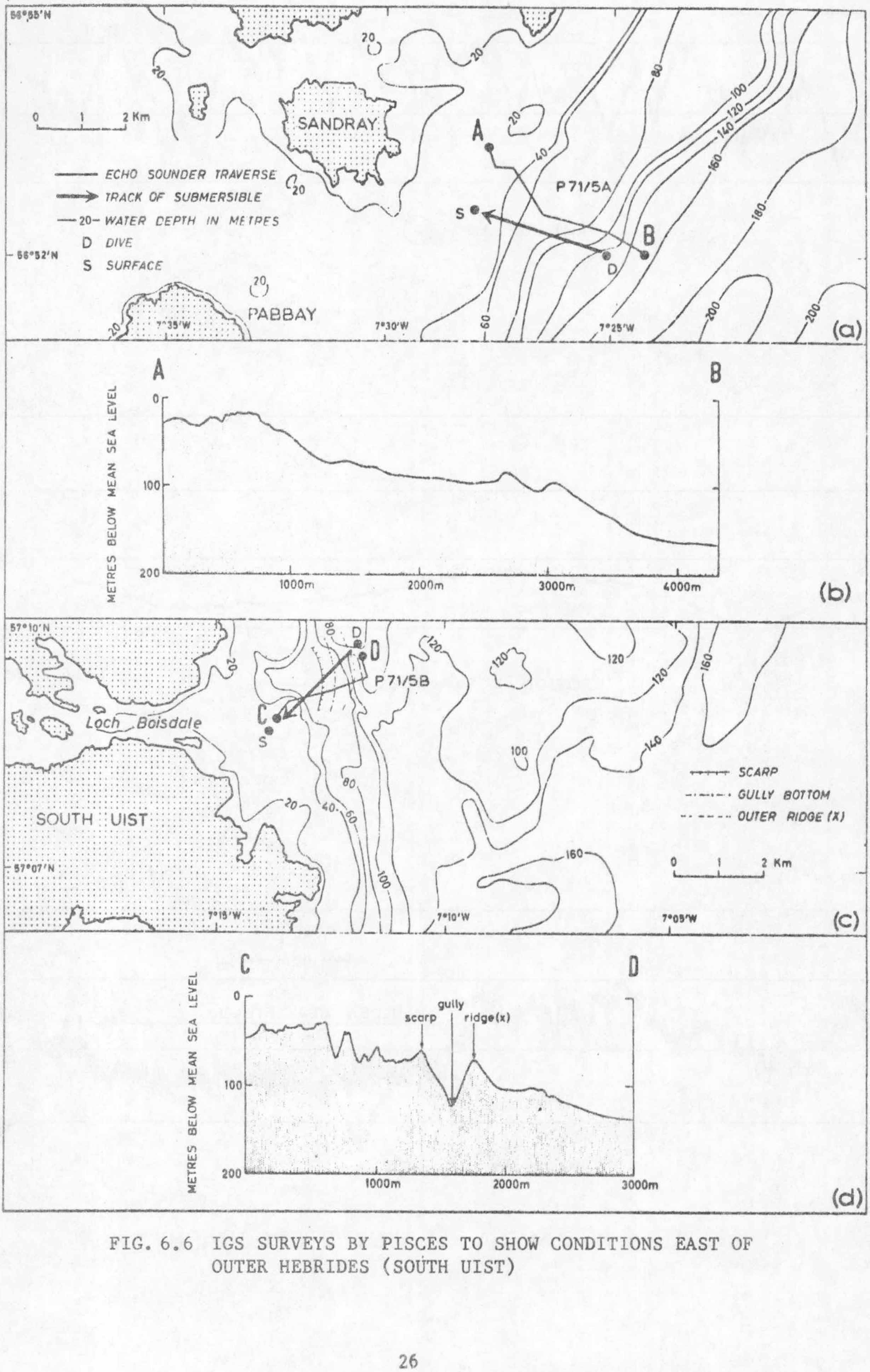


FIG. 6.6 IGS SURVEYS BY PISCES TO SHOW CONDITIONS EAST OF OUTER HEBRIDES (SOUTH UIST)

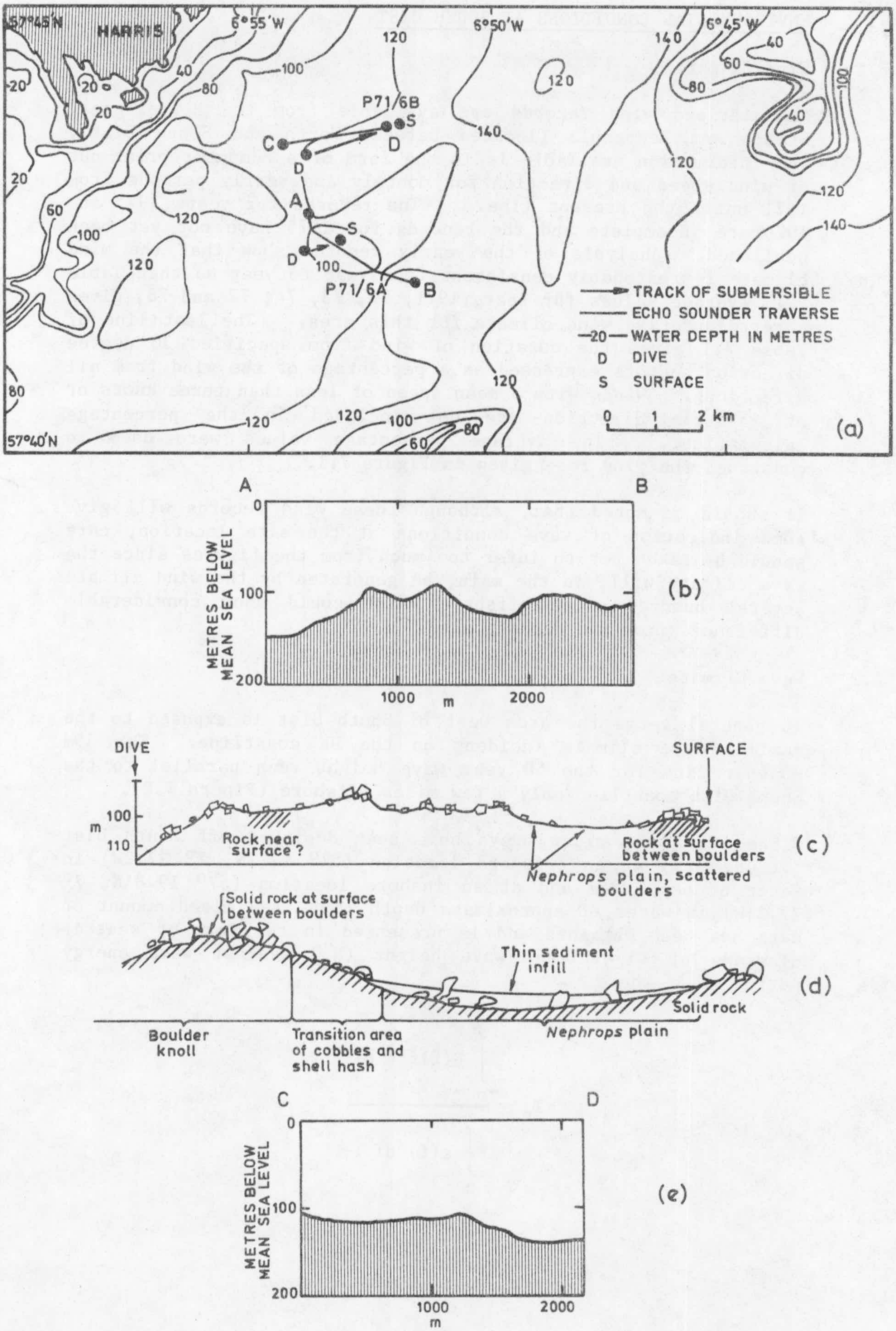


FIG 6:7 I.G.S. SURVEYS BY PISCES SHOWING SEABED CONDITIONS S.E. OF HARRIS

7 ENVIRONMENTAL CONDITIONS AT SOUTH UIST

7.1 Wind Climate

Computerised wind records are available from the Metrological Station at Benbecula (located between North and South Uist). The information available is in the form of a computer print out of wind speed and direction for monthly and yearly periods from 1971 until the present time. The records for years 1975 and 1976 are incomplete and the records for 1979 have not yet been published. Analysis of the yearly records show that the wind climate is reasonably consistent from year to year so that Table 7.1, Average Values for Years 1971, 72, 73, 74, 77 and 78, gives a representative wind climate for this area. The last line of Table 7.1 gives the duration of wind from specified 30 degree direction sectors expressed as a percentage of the wind from all directions. Winds with a mean speed of less than three knots or of variable direction are not included in the percentage calculation. The average percentage values were used to construct the wind rose given in Figure 7.1.

It should be noted that, although these wind records will give some indication of wave conditions at the site location, care should be taken not to infer too much from the figures since the wave climate will, in the main, be generated by the wind climate several hundred miles offshore which could show considerable differences from the onshore site.

7.2 Wave Climate

In general terms the area west of South Uist is exposed to the maximum wave climate incident on the UK coastline. The 35m contour line for the 50 year wave height runs parallel to the South Uist coastline only a few miles offshore (Figure 7.2).

Since 1976 wave rider buoys have been deployed off South Uist (Figure 7.3) at an offshore location (57° 12.2N, 7° 37.2W) in water of depth 44m and at an inshore location (57° 19.8'N, 7° 27.2'W) in water of approximate depth 15m. A limited amount of data has been obtained and is presented in the form of scatter diagrams of significant wave height (H_s) against wave energy period (T_e), where

$$T_e = \frac{\int_0^\infty \varepsilon(f) f^{-1} \, df}{\int_0^\infty \varepsilon(f) \, df}$$

Mean wind speed	TOTAL NUMBER OF HOURS FROM DEGREES TRUE													
	350°	020°	050°	080°	110°	140°	170°	200°	230°	260°	290°	320°	Var.	Total
0-3	16	24	28	29	24	28	32	23	14	9	14	8	164	475
4-10	209	200	207	217	202	322	300	316	267	190	210	127	2	2771
11-21	277	178	282	80	141	825	568	632	582	471	328	282	0	4326
22-33	52	22	42	9	30	179	173	121	120	172	126	78	0	1122
Over 34	3	0	2	0	1	7	12	5	8	12	4	5	0	70
Wind over 3 knots %	6.3	4.6	6.2	3.6	4.3	15.5	12.2	12.5	11.3	9.8	7.9	5.7		

Note: The percentage values given at the foot of the table do not include winds of variable direction or less than 3 knots.

Table 7.1

Wind Speed & Direction at Benbecula

Average values for years 1971, 72, 73, 74, 77 & 78

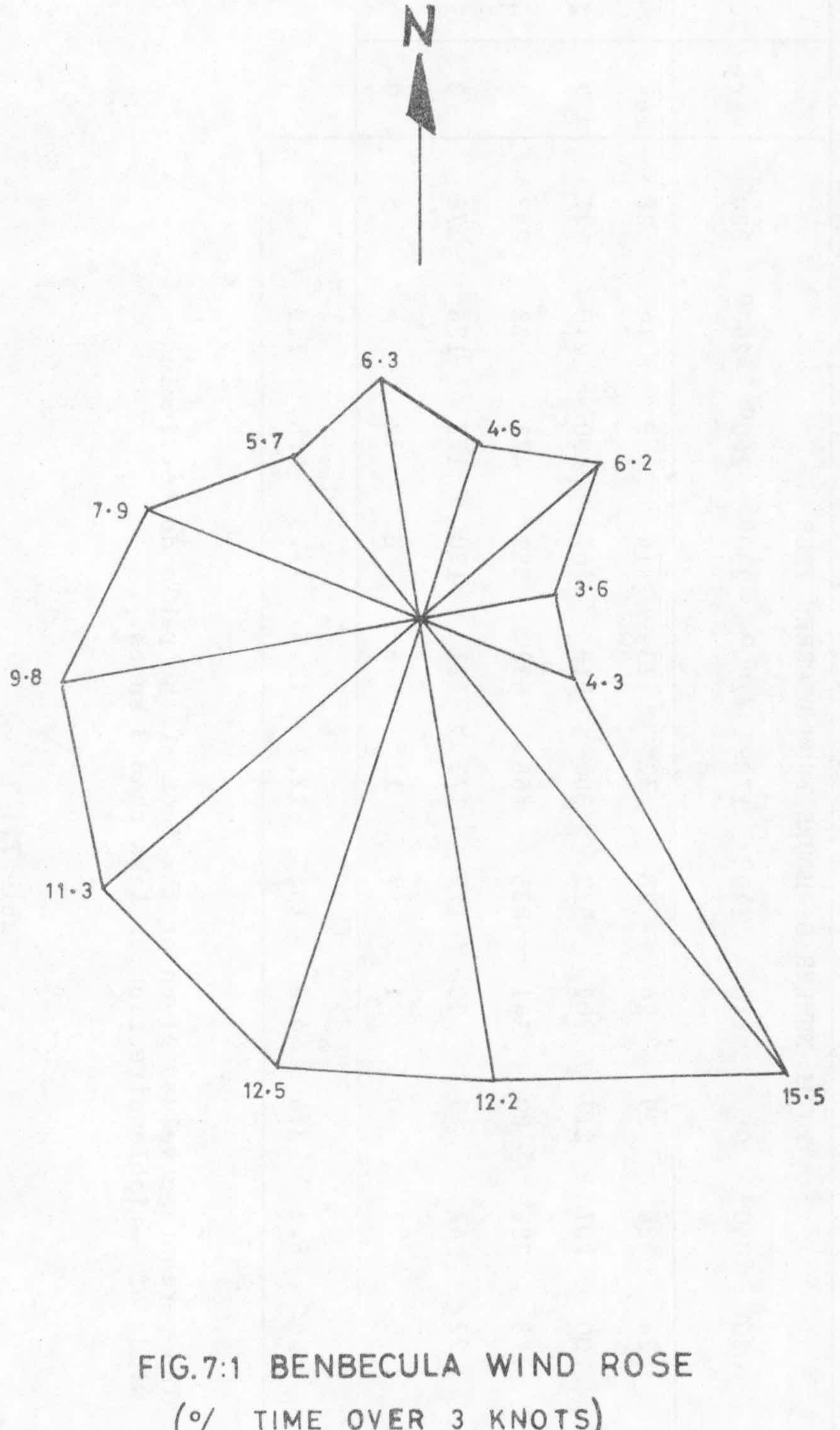


FIG. 7:1 BENBECULA WIND ROSE
(% TIME OVER 3 KNOTS)

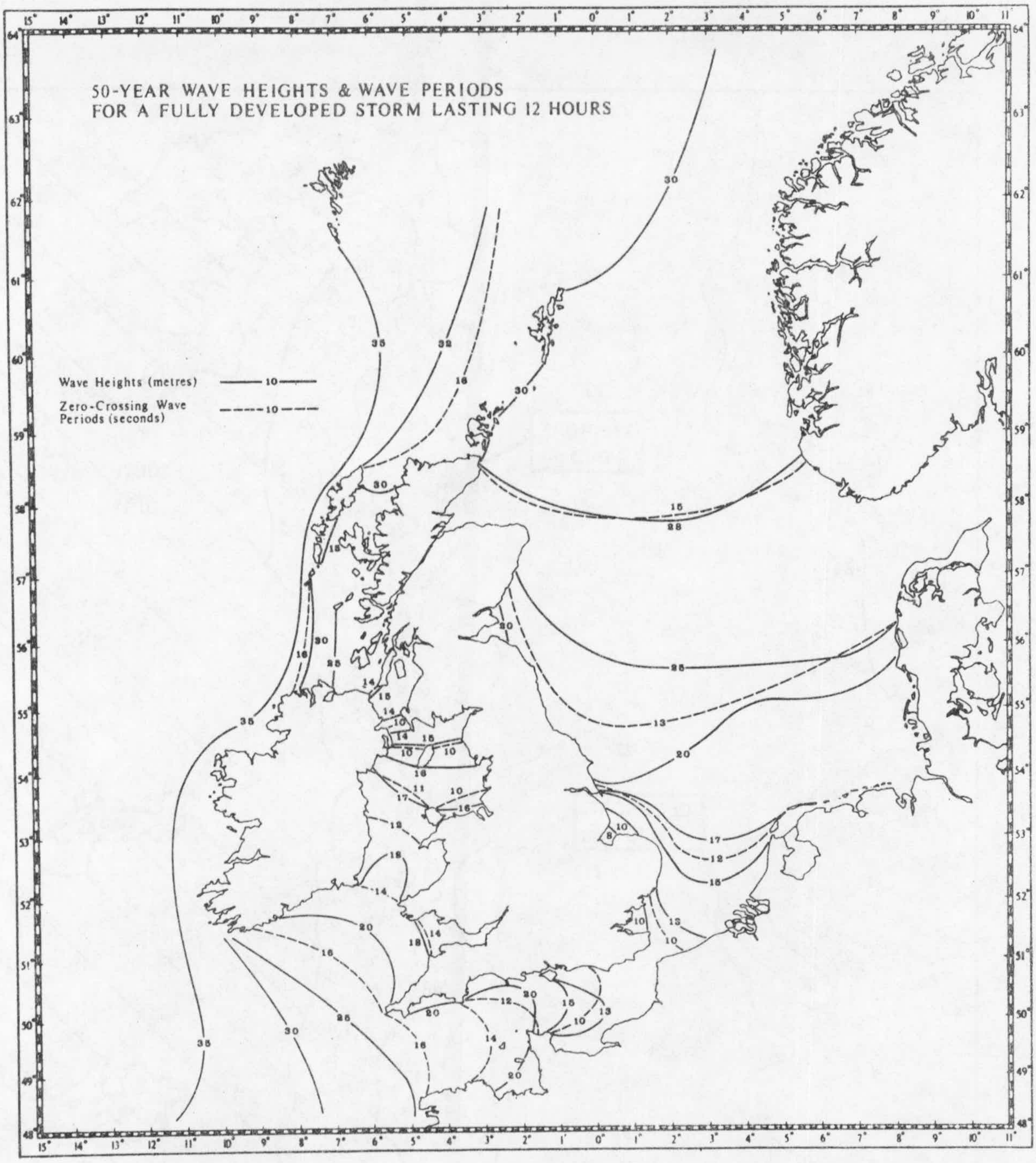


FIG. 7.2 50 YEAR WAVE HEIGHTS AND WAVE PERIODS
FOR A FULLY DEVELOPED STORM LASTING 12 HOURS

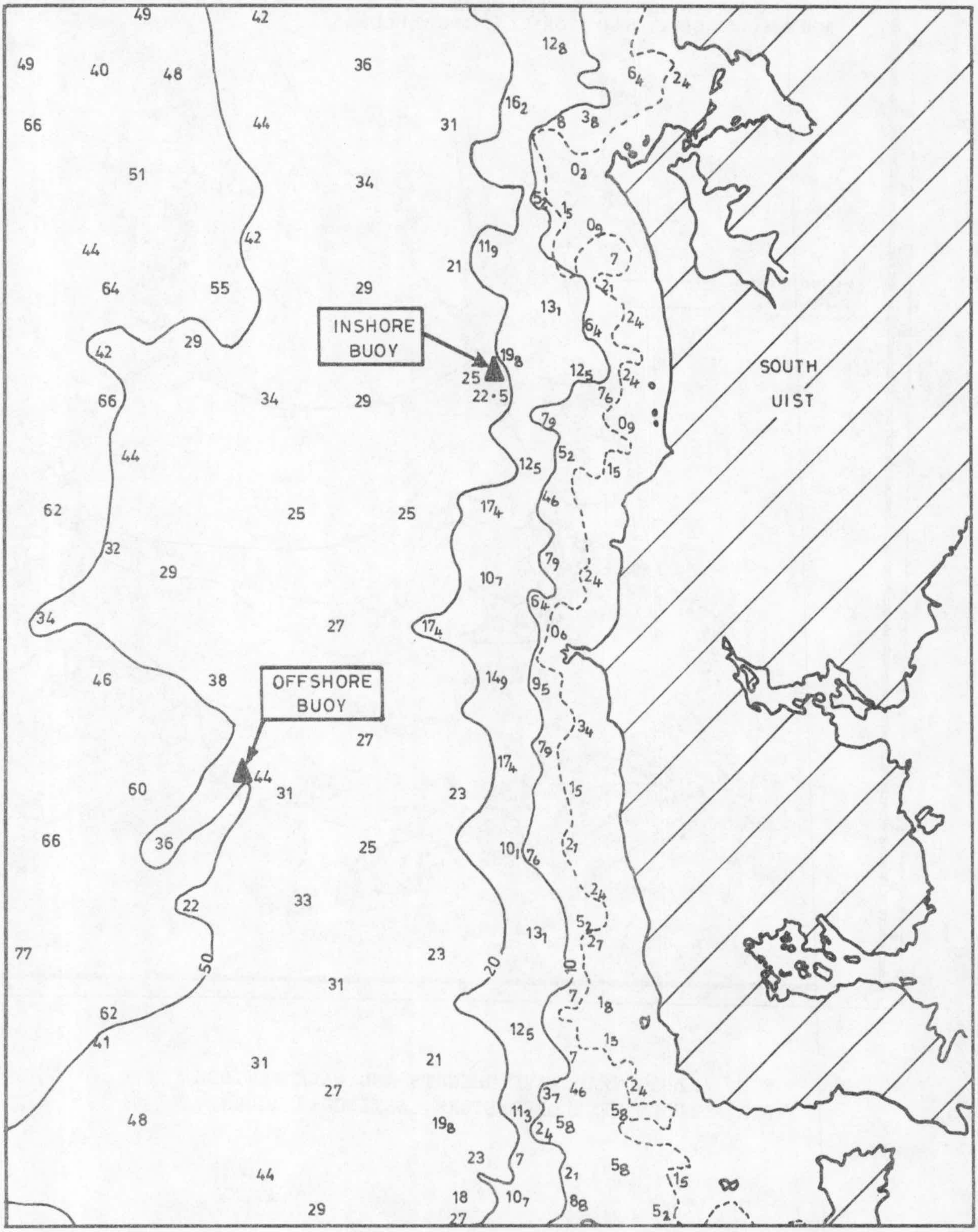


FIG 7:3 LOCATION OF WAVE RIDER BUOYS (DEPTH IN METRES)

The scatter diagram for the offshore site for the period March 1976 to February 1977 is given in Figure 7.4 and for the period August 1978 to June 1979 in Figure 7.5. The corresponding scatter diagram for the inshore buoy for the latter period only is given in Figure 7.6.

At the relatively deep water offshore site the power in the waves can be assumed to be approximately proportional to T_e. However, at the inshore site where the water is relatively shallow, T_e does not give a reliable indication of power.

The power outputs measured at the inshore and offshore locations are compared in Table 7.2 over a range of offshore values of H_s and T_e.

T_e (secs)	H_s (m)			
	0 - 2	2 - 4	4 - 6	6 - 8
4-6	2.6	2.5		
6-8	2.5	2.6	2.0	
8-10	3.0	3.0	3.4	
10-12	2.3	3.1	3.4	6.1
12-14	2.5	3.3		5.2
14-16	2.6	2.6		
16-18	1.7	1.7		

Note: T_e and H_s values at offshore location.

Table 7.2

Average power ratio offshore buoy location/inshore buoy location

Average spectra were calculated for both wave rider buoys for the period August 1978 to June 1979 and are compared graphically in Figure 7.7. The resulting average power values obtained are:

At offshore buoy location 37.2kW/m
At inshore buoy location 12.7kW/m

This gives an overall power ratio offshore location to inshore location of 2.9, i.e. the power available at the inshore location is approximately one third of that available offshore.

This overall ratio is reflected in the detailed comparison graph for H_s values between 0 and 2m, and T_e values between 8 and 10 secs (Figure 7.8).

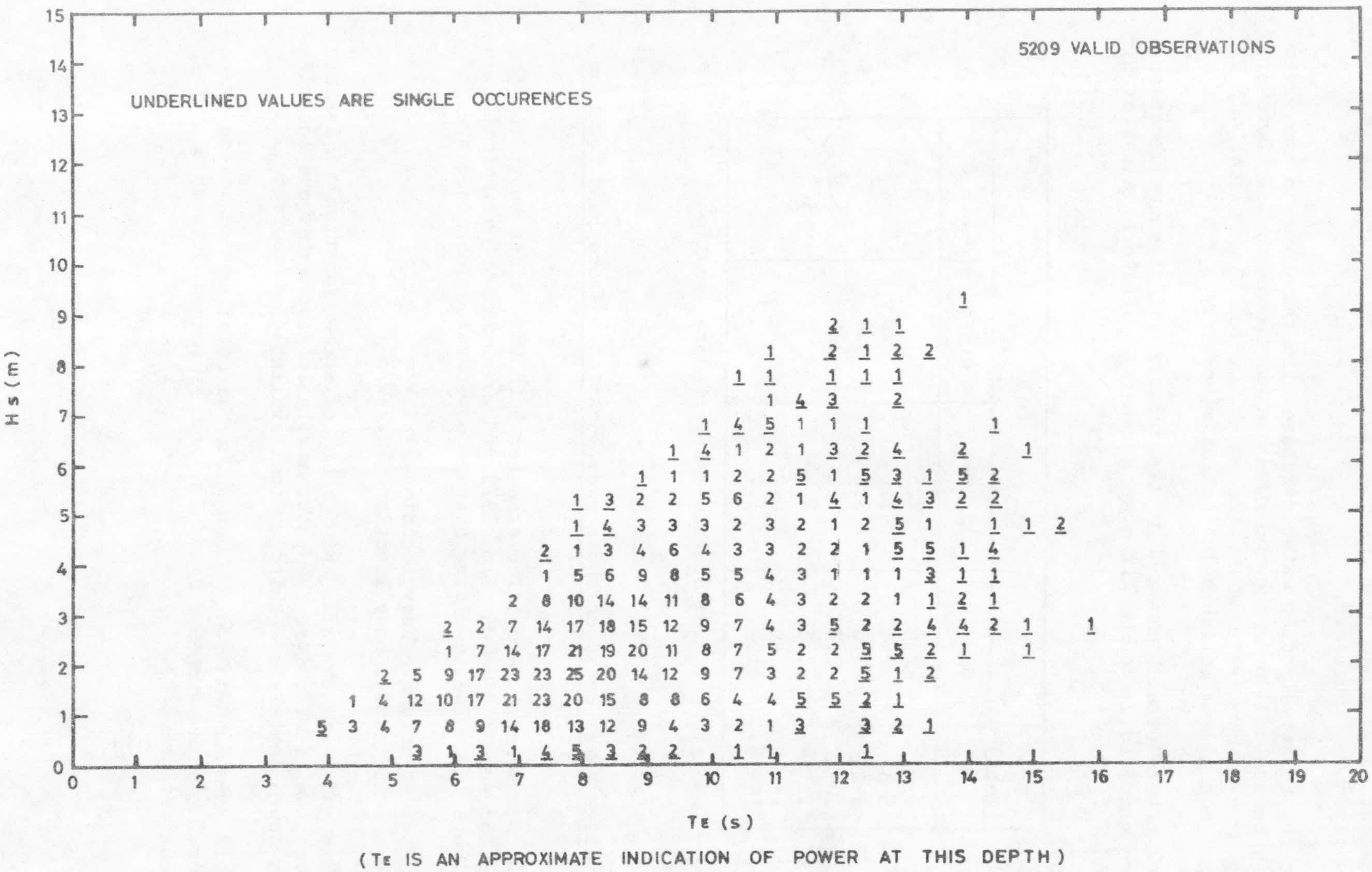


FIG. 7.4 SCATTER DIAGRAM OF Hs AND Te (ppt)
OFFSHORE SOUTH UIST MARCH 1976-FEBRUARY 1978

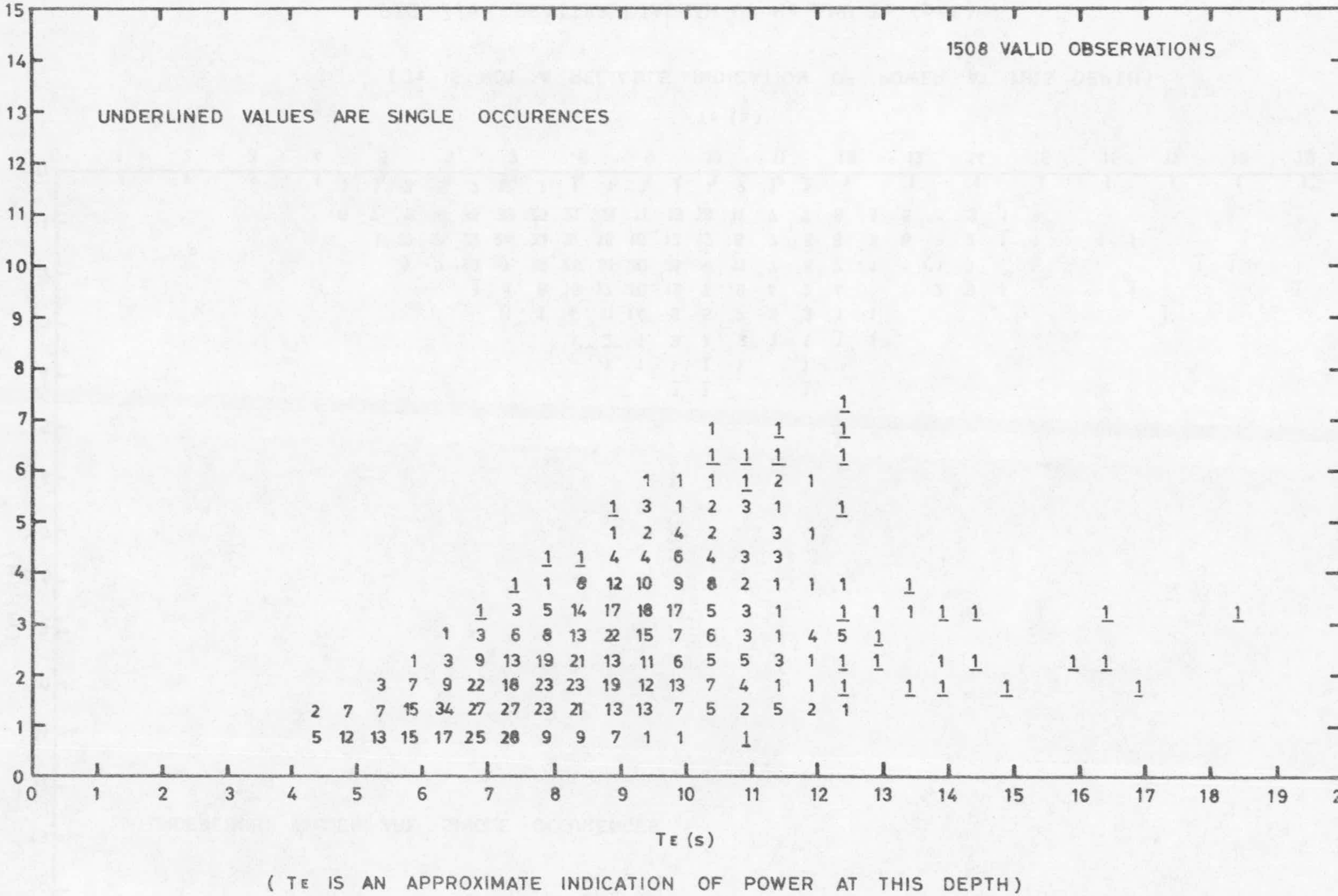
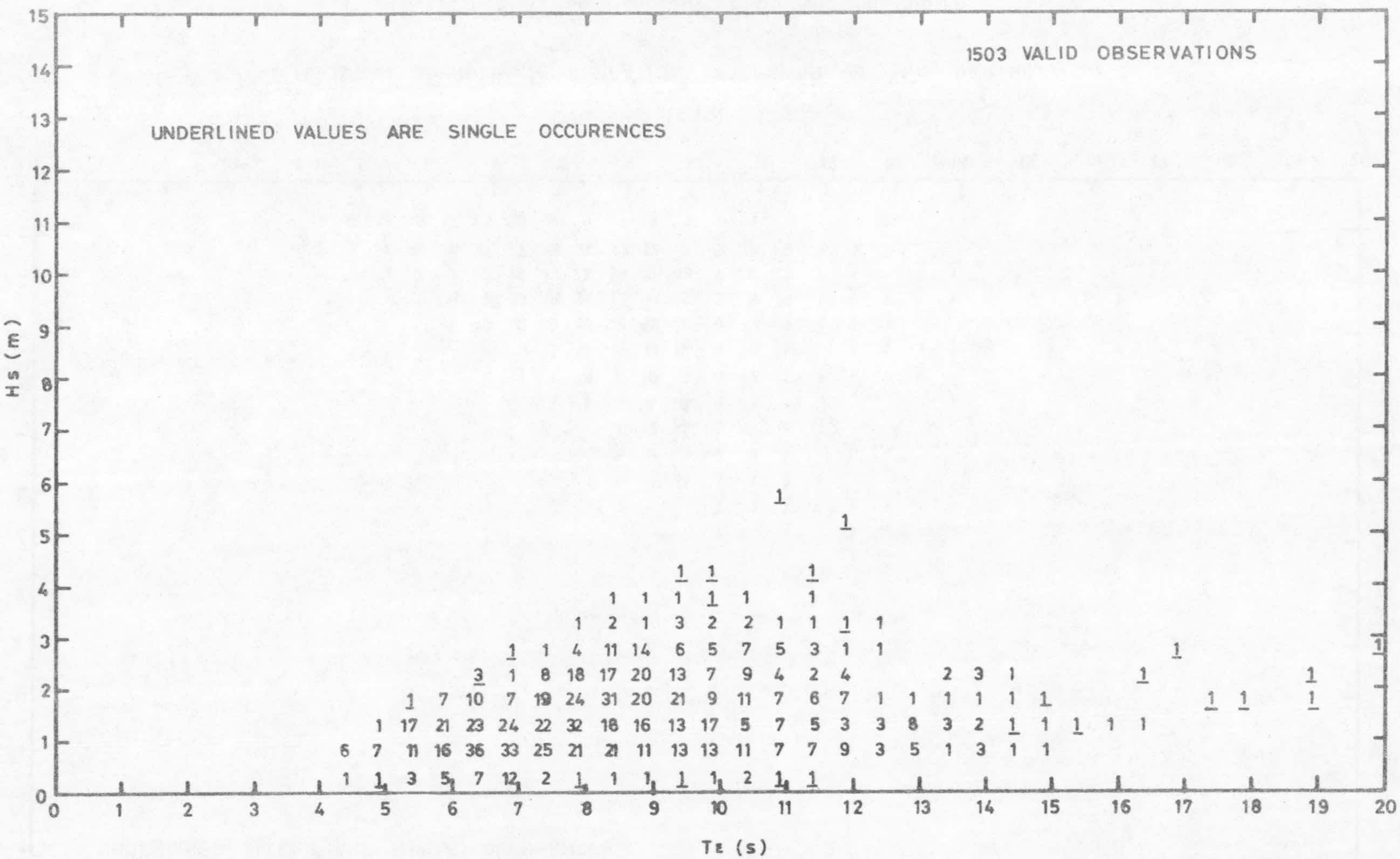


FIG. 7.5 SCATTER DIAGRAM OF Hs AND Te (ppt)

OFFSHORE SOUTH UIST AUGUST 1978-JUNE 1979



(Te IS NOT A RELIABLE INDICATION OF POWER AT THIS DEPTH)

FIG. 7.6 SCATTER DIAGRAM OF Hs AND Te (ppt)

INSHORE SOUTH UIST AUGUST 1978-JUNE 1979

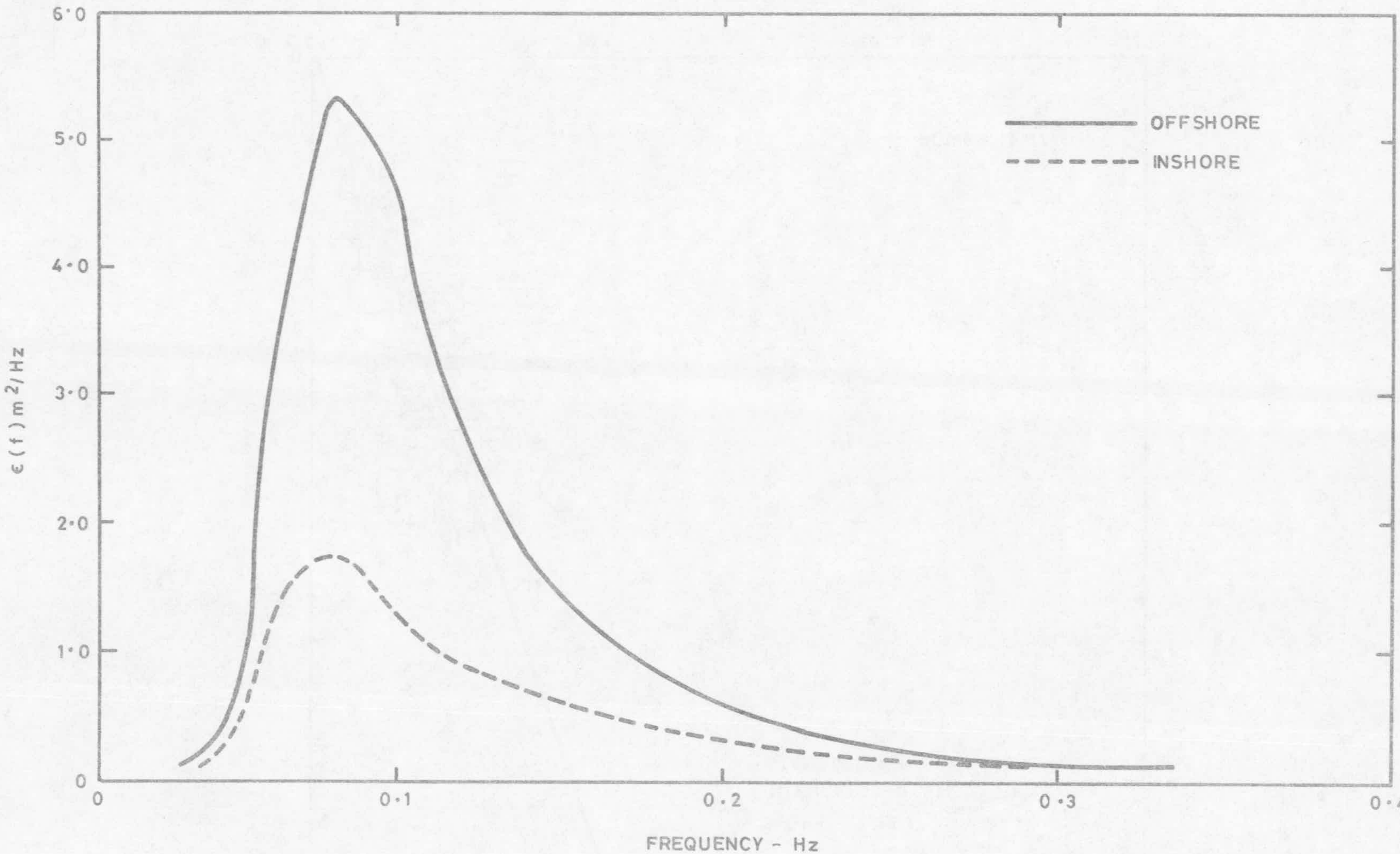


FIG 7:7 AVERAGE SPECTRA FOR SOUTH UIST AUGUST 1978 - JUNE 1979

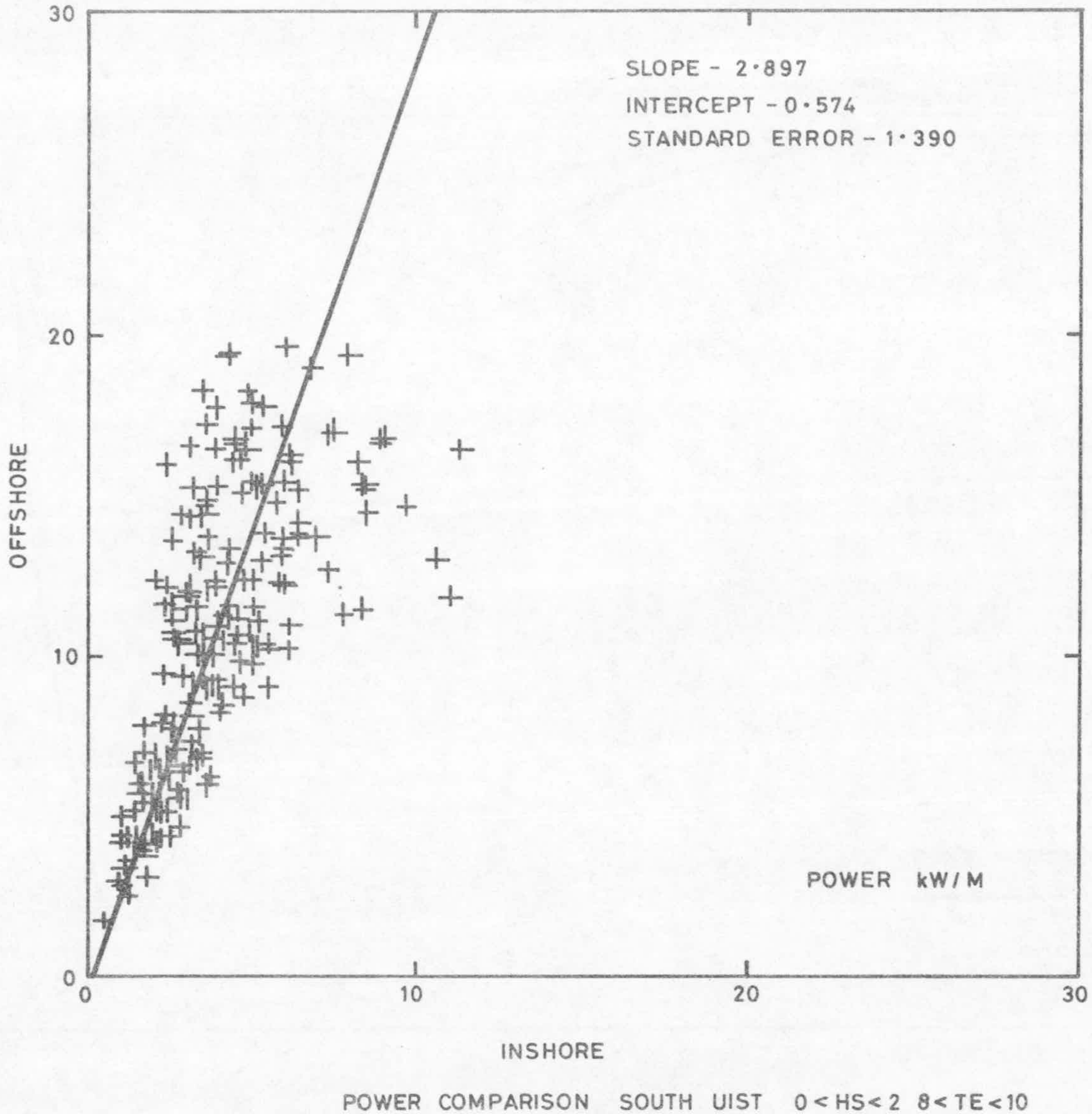


FIG 7:8 TYPICAL POWER COMPARISON GRAPH

7.3 Tidal Currents

There is no published information giving detailed current measurement for the area west of South Uist. However, a limited amount of general information is available from Admiralty publications, charts etc. It should be noted that current information from charts refers only to the surface water layers and cannot be assumed to be uniform in either speed or direction throughout the water depth. Specific site investigations will be necessary to determine current profiles, especially currents at the seabed, since they will have particular bearing on scour and deposition of seabed material at the base of the structure.

Current velocity information for the UK as a whole is given in Figures 7.9 and 7.10. Data for the approximate location of the offshore wave rider buoy is given in Table 7.3. This data was taken from Admiralty Chart No. 2722 for location 57° 19'N, 7° 38.5' i.e. approximately 8 miles west of South Uist.

Hours		Rate (m/s)		Direction °T
		Spring tides	Neap tides	
	6	0.05	0	270
Before	5	0.10	0.05	000
high	4	0.15	0.05	032
water	3	0.21	0.10	037
	2	0.21	0.05	042
	1	0.10	0.05	041
High Water		slack	slack	
	1	0.05	0	235
After	2	0.10	0.05	211
high	3	0.15	0.05	204
water	4	0.21	0.10	207
	5	0.15	0.05	216
	6	0.10	0.05	230

Table 7.3

Current Data at South Uist

7.4 Tidal Levels

Tidal information for the area west of South Uist is of a general nature. However, from Figure 7.11, which shows the tidal ranges for the UK, it can be seen that the area of interest lies in a relatively stable tidal regime. An extract from Admiralty Tide Tables based on the Standard Port of Ullapool showing tidal variations adjacent to the area of interest is given in Table 7.4. Analysis of this table gives an average tidal range of 3.6 metres between mean high water and low water levels during spring tides (i.e. MHWS-MLWS).

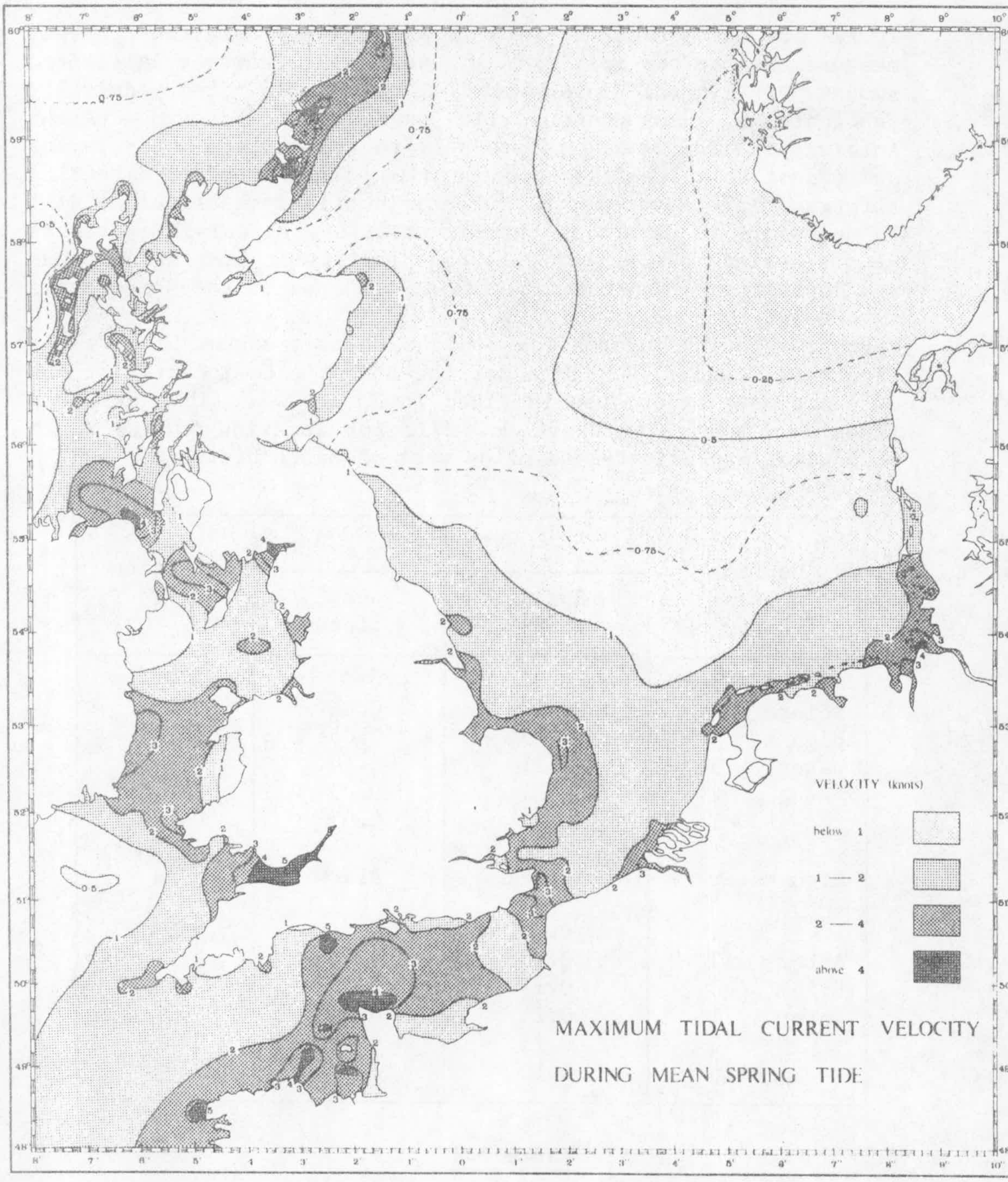


FIG. 7.9 MAXIMUM TIDAL CURRENT VELOCITY
DURING MEAN SPRING TIDE

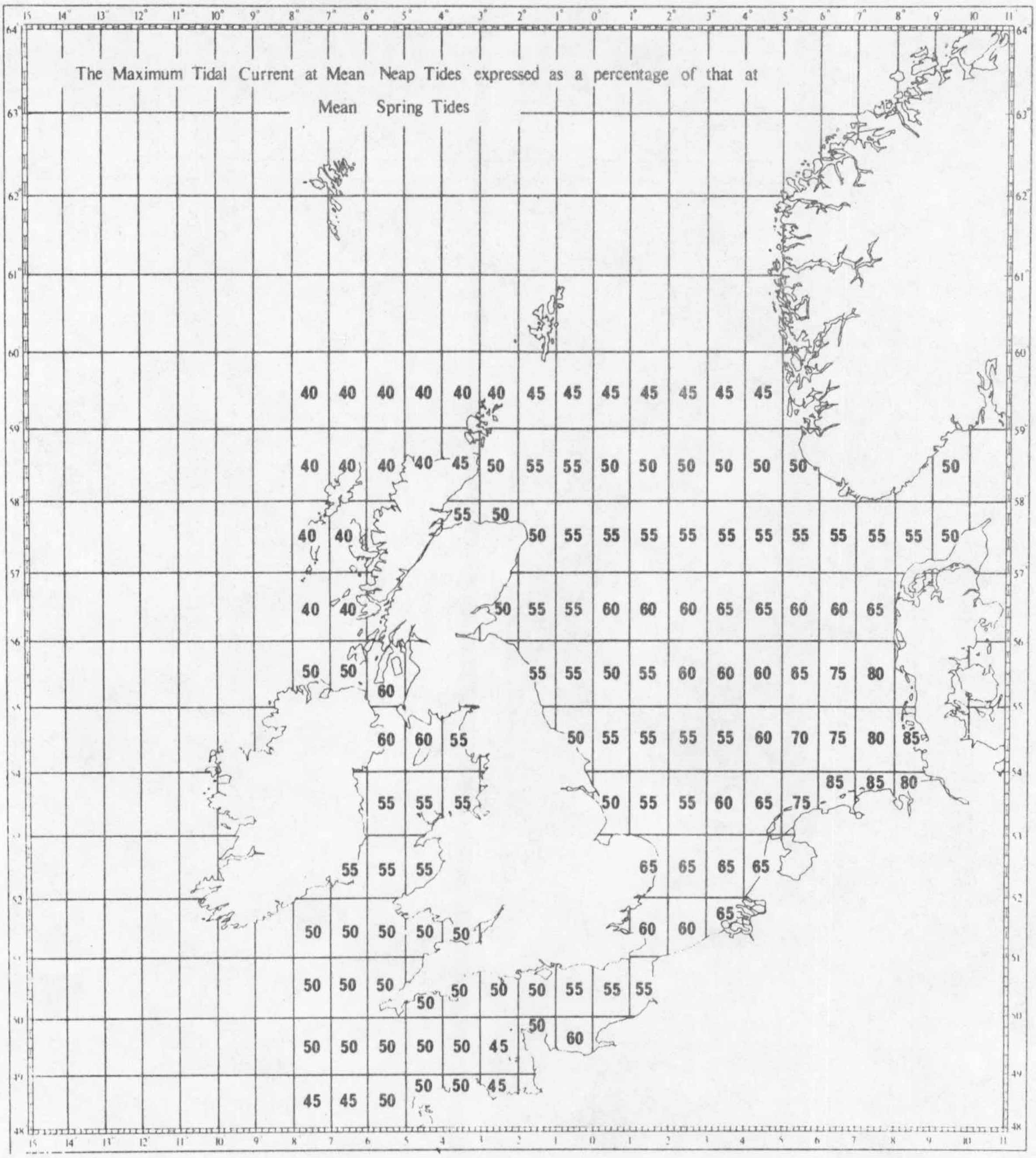


FIG 7.10 MAXIMUM TIDAL CURRENT AT MEAN NEAP TIDES EXPRESSED
AS A PERCENTAGE OF THAT AT MEAN SPRING TIDES

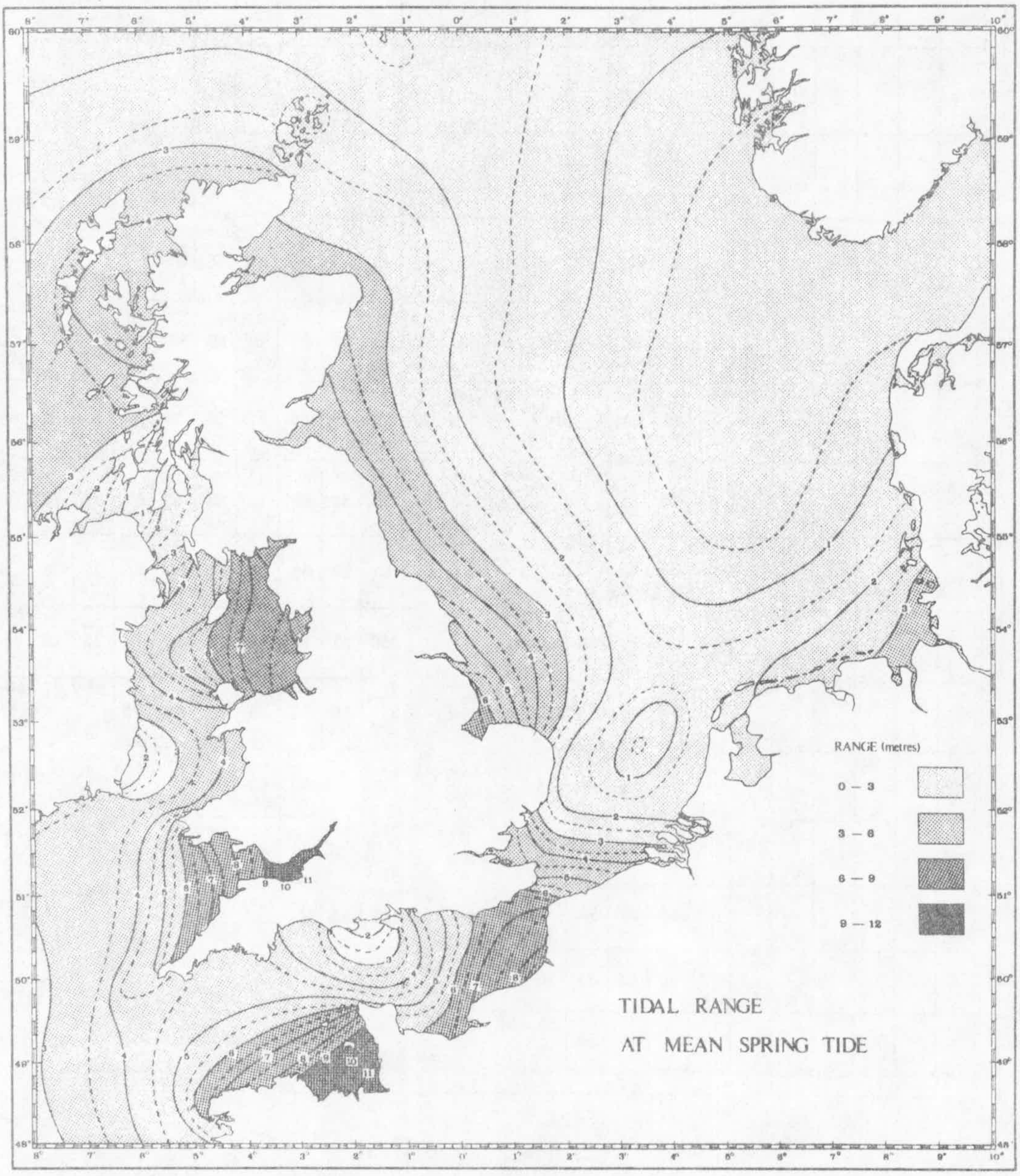


FIG. 7.11 TIDAL RANGE AT MEAN SPRING TIDE AROUND THE UNITED KINGDOM

Location	Position		Height above datum (m)			
	Latitude N	Longitude W	MHWS	MHWN	MLWN	MLWS
Ullapool Standard Port	57° 54'	5° 10'	5.2	3.9	2.1	0.7
Barra Head	56° 47'	7° 38'	-1.2	-0.9	-0.3	+0.1
Shillay	57° 31'	7° 41'	-1.0	-0.9	-0.8	-0.3
Bernera Harbour	58° 16'	6° 52'	-0.9	-0.8	-0.5	-0.2
Uachdair	57° 29'	7° 23'	-1.1	-0.8	-0.6	-0.2
Average	-	-	-1.0_5	-0.8_5	-0.5_5	-0.1_5

Note: MHWS - Mean high water springs
MHWN - Mean high water neaps
MLWN - Mean low water neaps
MLWS - Mean low water springs

Table 7.4

Tidal Constants West of South Uist

7.5 Storm Surge, Seiche, Tsunami

Storm surge results from severe storm conditions which, as the storm advances, forces the water ahead of it and causes the mean water level to rise above the normal tidal range. The mean level can rise by several metres if the water is forced down a narrowing channel, and in this context, the southern part of the North Sea can be regarded as a narrow channel. The area west of the Hebrides, however, is an open sea area and severe storm surges should not occur. No actual records exist for this area so the probability of surge occurrence is based on predictions by the IOS Hydrographic Office and Metrological Office Storm Warning Service. The consensus of opinion is that a surge of up to 1m would occasionally occur west of the Outer Hebrides, but the elevation on top of high water is likely to be not more than 0.75m. A surge of 2m is thought to be a once in a lifetime occurrence and a 3m surge too rare to warrant consideration.

Seiche or harbour surge is often generated by long waves caused by atmospheric disturbances but can also be caused by nonlinear effects of irregular wind waves which have the period of wave trains. This effect can set up resonances in enclosed harbour areas which cause variations in water level. It is unlikely that the envisaged configurations of the OWC would enclose areas of water which would be subject to seiche effects.

Tsunami, or seismic waves, are the result of earthquakes, landslides or subterranean explosions. They travel at several hundred knots and can build up to 30m or more in height as they approach a coastline. Tsunami are rare on Atlantic coasts, and since they are of the same order of magnitude as 50 year storm waves, should be within the design specification of the device.

8. THE BREAKWATER STRUCTURE

8.1 Structural Design Philosophy

The preliminary design of the structure for the Breakwater Device was carried out using limit state (i.e. semi-probabilistic) methods. The objective of these methods is to achieve an acceptable probability that the structure will never reach a state at which it is unfit for the use for which it has been designed. Any such state is known as a limit state.

It is necessary to consider two main groups of limit states. The first group is the ultimate or failure limit states which includes:

- Rupture or yielding of the section;
- Buckling;
- Implosion or explosion;
- Sliding and overturning.

The second group is the serviceability limit states which ensures that the structure performs satisfactorily under working loads. Typical checks at serviceability levels include:

- Deflection:
- Cracking;
- Vibration;
- Corrosion;
- Durability;
- Fatigue.

The main parameters governing the design are basically random variables for which full statistical information is generally not available. However, the method overcomes this lack of basic data by incorporating partial safety factors in both the load effect and the section resistance calculations, so that a satisfactory overall factor of safety is obtained.

In order that the design process can be carried out, values of loading which will rarely be exceeded in practice, and values of material strengths below which only a specific number of test results lie, are determined. These are known as characteristic values. In the analysis and design of the structure, or an element of the structure, characteristic values of different loadings are combined using different sets of partial load factors. The values of the partial load factors within each set depend upon the return period of the loading and the accuracy with which its value can be determined. In the same way the resistance of the section is reduced by a partial material factor which varies according to the particular limit state being checked.

Economical design of the structure is then achieved by carefully establishing detailed limiting criteria (at the various limit states) on the effect that the combined loading has on the individual element of the structure. The position of the element under consideration and the way that it is reinforced also affects the specification. The detailed criteria used for the design of the Breakwater Device are given in Appendix I - Design Specification.

The design of the structure is broken down into phases. These cover the anticipated life of the structure from construction through transportation to the site, installation and operation at the site, and removal when the structure is obsolete. The design phases, their reference numbers and the return period for assessment of the loading conditions are given in Table 8.1.

Phase			Loading Conditions	Design Return Period
1.	Construction	A	In drydock	
		B	During float out	*
		C	At inshore floating berth	10 years
2.	Transportation	A	To holding area	10 years
		B	During connection of emplacement barge	*
		C	To site location	10 years
3.	Installation	A	Emplacement	*
		B	Post emplacement	10 years
4.	Operation	A	Normal environmental and system conditions, maximum imposed loads	1 month
		B	Extreme environmental and system conditions, maximum imposed loads	50 years
		C	Extreme environmental and system conditions, minimum imposed loads	50 years
		D	Damage or overload conditions	50 years
5.	Retrieval	A	Removal operations at site	10 years
		B	Transportation to disposal site	10 years

N.B. * Return period to be assessed.

Table 8.1

Design Phases

Loadings are split into six categories as follows (see section 5 of Appendix I)

- Dead loads (denoted by G_k)
- Imposed loads (denoted by Q_k)
- System loads (denoted by S_k)
- Environmental loads (denoted by V_k)
- Deformation loads (denoted by D_k)
- Accidental loads (denoted by A_k)

The partial load factors that apply to various combinations of these loads for ultimate limit states are given in Section 7.3 of Appendix I. Load factors are always taken as unity for serviceability limit states.

In general, the most important serviceability limit states for concrete offshore structures are those of cracking, corrosion and durability, and fatigue. In the case of the first three states the location of the part of the structure under consideration is very important. Sections located in the splash zone generally require higher criteria than those that are fully submerged or those that are completely sheltered from the salt spray.

Fatigue in reinforced concrete in the marine environment is poorly understood at this time and is currently the subject of considerable research. The approach used for the design of the Breakwater structure is to establish separate endurance limits for the concrete and steel components, and then to restrict the stresses in the components to less than these endurance limits. The level of loading used to derive the stresses is that which occurs at least 20,000 times. The most critical component is usually the non-prestressed reinforcement.

The corrosion and durability serviceability limit states are generally satisfied by ensuring that the concrete has certain minimum properties and that the reinforcement has satisfactory cover of a specified minimum thickness.

8.2 Structural Loads

The loading on the Breakwater structure occurs in two separate and distinct ways. The first is during the floating phases of construction, tow out and emplacement. In this phase the device is subject to a combination of stresses resulting from the "hull girder" action and from the local panel hydrostatic effects. The second type of loading occurs once the structure has been installed on the seabed. In this mode the device acts like a breakwater and is loaded by the horizontal action of the waves.

The assessment of the loads for the floating condition was carried out using the provisional Lloyds Rules for concrete ships (Reference 7). These rules give minimum design values for the hull girder moment and the local panel hydrostatic loading. The hull girder moment computed from the formulae given in the Rules compared reasonably well with the value found using the formulae deduced from model testing of the Clam device by Sea Energy Associates (Reference 8).

The second type of loading condition is that which occurs during the operation of the device. At low wave amplitudes the water column extracts a considerable proportion of the energy from each wave, resulting in the horizontal wave momentum being translated into the vertical direction and resisted by the head of the wave piston. At higher wave amplitudes a greater amount of energy is reflected thereby creating typical breakwater conditions.

The loads acting on the structure in the operating phase were therefore calculated by assuming that the device acts entirely as a breakwater and reflects all approaching waves. The combination of incident and reflected waves creates a standing wave pattern which was analysed using second order wave theory. The resulting pressure distribution was simplified and integrated to produce a surge force and overturning moment.

A subsequent series of tests in the narrow tank at the National Engineering Laboratory on a model of the Breakwater provided information on the horizontal and vertical forces acting on the structure and on the associated air pressures in the water column. The results have been reported in NEL Progress Report 20 (Reference 9). The results of these tests have shown that the theoretical analysis is reasonably conservative.

A summary of the main structural loadings used in the design is given in Appendix II.

8.3 Structural Materials

The two main structural materials considered for the construction of the Breakwater Device were steel and concrete, both being widely used in offshore structures. However, it has generally been found that the choice of material is highly influenced by the nature of the proposed structure. This can readily be illustrated by an examination of North Sea structures. Where the purpose of the structure is mainly to provide a working platform clear of the sea, the transparent multi-element four or more legged jacket predominates. Steel is generally used for this type of structure because it has a very high strength to weight ratio and is easily fabricated by welding etc. In the case of the structures also incorporating oil storage cells, the shell and plate-like elements necessary to provide the storage envelope have been more readily constructed in concrete.

In the case of the Breakwater structure the choice of concrete is more readily made partly because of the extensive plate-like elements required to form the water column chamber, but also because of the requirement to provide as much weight as possible for stability purposes. The reduced maintenance requirements of concrete in the marine environment are also significant over the life of the structure.

An additional consideration is the lower input energy content of concrete as opposed to steel. Comparative designs in both materials were prepared for the 1978 Floating Reference Design. The steel design required 12,000t of materials whilst the concrete design required 80,000t of material. However, very basic calculations show that the total energy required to construct the steel structure is 6.7×10^5 GJ as opposed to 2.6×10^5 GJ for the concrete structure, thereby showing that there could be a considerable saving of input energy by the use of concrete as the main structural material.

As a structural material, concrete has poor resistance to tensile stresses. In practice these are normally either carried directly by steel reinforcement embedded in the concrete (i.e. reinforced concrete) or are prevented from occurring by using steel wires or tendons to impose a compressive stress of suitable magnitude on the section (i.e. prestressed concrete).

There appears to be a wider range of materials available for secondary structures on the Device including steel, aluminium/manganese/magnesium and other alloys, concrete with various types of reinforcement and reinforced plastics. Of these materials the range of reinforced plastics appears to offer the highest strength to weight ratios combined with extremely good resistance to the corrosive effect of sea water. As noted in Section 4.2 it is expected that glass reinforced plastic (GRP) will be used for the ducting rectifying valve, turbine casing and runner components in the power generation equipment.

The modularised boxes enclosing the alternator and other items of control equipment will be constructed from steel, primarily because of the ease with which it can be fabricated in structures of this size.

8.4 Corrosion protection

Corrosion of exposed steelwork on and embedded reinforcement in concrete structures in the marine environment can be very rapid due to the build up of electrical potential and resulting anodic currents. In wave energy devices this could be compounded by the presence of stray leakage currents from the power generation equipment.

This corrosion can be controlled in several ways. One method is to suppress the anodic reactions on the steel by applying an external current either by an external DC source (i.e. an impressed current scheme) or by an alternative anode fabricated from a metal higher on the galvanic series (i.e. a sacrificial anode scheme). A second method would be to ensure complete electrical isolation of the reinforcement from all exposed steelwork. In practice it is likely that a combination of these methods will be used.

9. STRUCTURAL DESIGN

9.1 Main Areas of Design

The design of the Breakwater structure has been resolved into two main areas. The first is concerned with the stability of the overall structure, both while floating and after emplacement. The second area is the detailed design of the components and elements of the structure. This includes the rock anchor and other prestressing requirements.

9.2 Overall Structure

The design of the overall structure is best illustrated by tracing the development of the cross section from its origin in the 1978 Floating Reference Design. This is shown visually in Figure 9.1. At an early stage in the development of the 1978 configuration it was seen that the location of the water column on one side of the transverse section created an assymmetry which necessitated considerable amounts of compensating ballast on the other side of the cross section. The use of concrete as the main structural material also caused the outside envelope of the device to become very large in order to provide sufficient supporting buoyancy.

The development of the Breakwater Device arose from the requirement to reduce the overall structure of the device. It was seen that this objective could be more readily achieved by making the device bottom standing, thereby eliminating the need for compensating ballast and the resulting extra buoyancy envelope structure. In the same way, it was also apparent that the power generation equipment had to be above sea level in order to keep the amount of hydrostatic resisting compartments to a minimum. This led to the the preliminary bottom standing device outlined in NEL Progress Report PR7 in May 1979 (Reference 10).

An important component of the preliminary (and subsequent) bottom standing proposals was the rock anchors. Their use was essential to provide adequate stability against sliding and overturning without requiring the extra structural content otherwise necessary.

Nevertheless, it was still essential to ensure that the bottom standing structure could float, in order to facilitate construction at the sheltered inshore sites as proposed for the floating device. This was achieved in the case of the May 1979 shape by temporary bulkheads across the mouth of the water column and at the extreme rear edge of the structure.

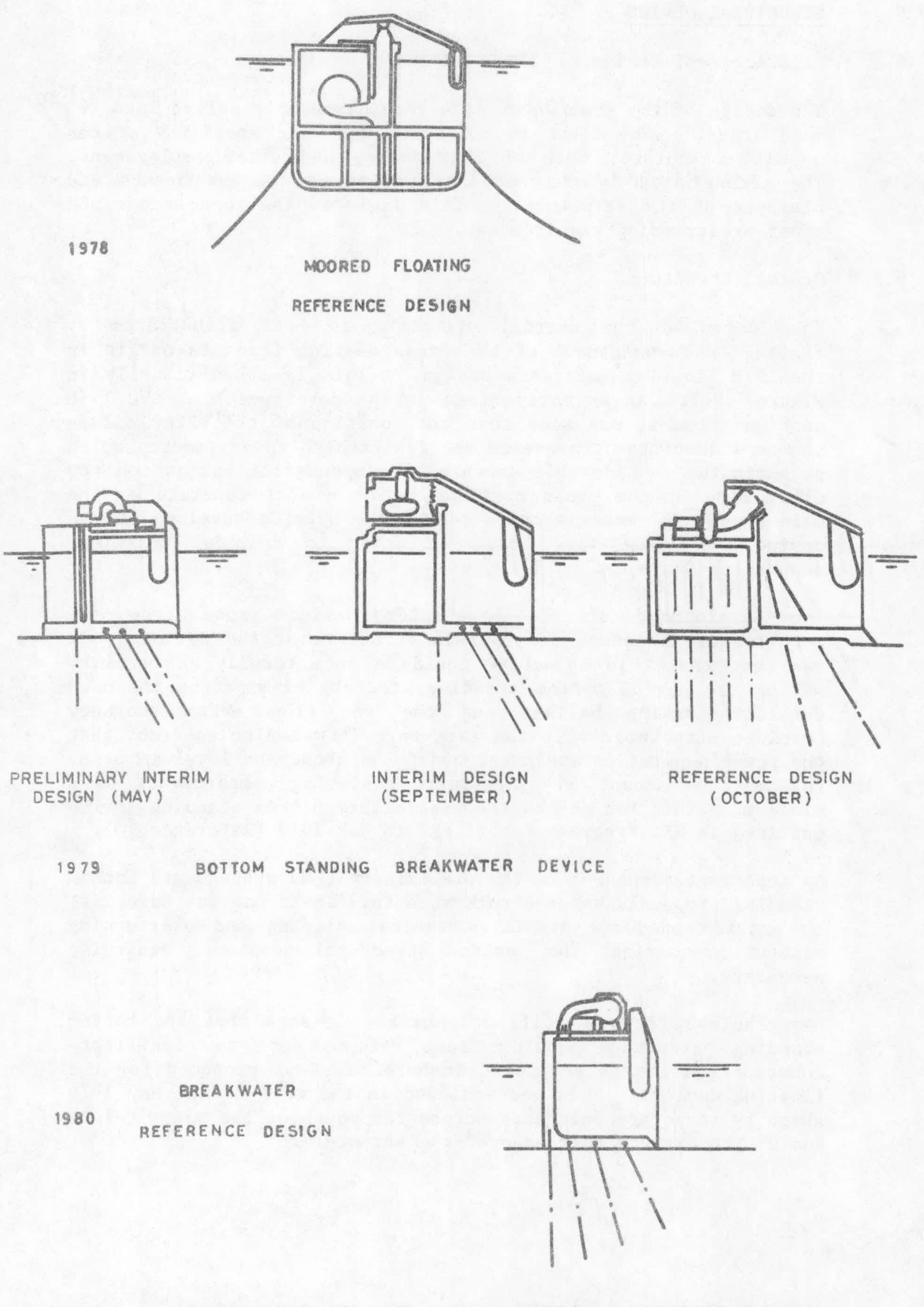


FIG 9:1 DEVELOPMENT OF THE BREAKWATER REFERENCE DESIGN

CROSS SECTION

However, as the more detailed study of the bottom standing device commenced, several areas required considerable attention. One of these areas was the complicated air ducting arrangement and gearbox necessitated by the requirement for the power generation equipment to present a low profile. This resulted in extra losses in the system. A second area was that the draft during tow out had to be unacceptably large in order to maintain satisfactory floating stability. This was partly due to the provision of a roof slab in order to protect the power generation equipment.

A further complication was that the structure had a very low resistance to sliding immediately after emplacement. At the time of the preliminary proposal it was expected that this could be solved by a combination of weather window and extra on board temporary ballast. However, on further investigation this approach was considered to be unacceptable.

The first problem to be tackled following the selection of the bottom standing device as front runner for 1979 was the reduction of the draft to not more than 12m in order to ensure satisfactory emplacement at any state of the tide. This was done by moving one of the centre spine panels in the preliminary proposal to the rear of the device. This also improved the floating stability by increasing the water plane area.

In order to provide a sheltered but clear working platform for installation of the rock anchors and other post emplacement work, it was decided that the power generation equipment should be installed after emplacement. In order to reduce the amount of work offshore, the equipment was packaged inside a large steel box containing several chambers which would act as air ducts. However, even with the use of double skinned orthotropic steel plates, the resulting 'big box' structure was found to be uneconomic.

Nevertheless, the concept of modularisation was retained and a power generation equipment layout incorporating four plant packages and two separate portions of air ducting was devised. This layout required a deck area which was unfortunately too large to be located directly on top of the water column. It was therefore located to the rear of the column just above high water level on a cellular platform. This location did give additional advantages, the first being the relative shelter to the equipment provided by the water column structure, and the second being the extra buoyancy offered by the support structure in the floating mode.

The problem of temporary stability immediately after emplacement was also solved at this time by the proposed provision of a catamaran type straddle barge. This barge provided the extra weight necessary to assure the stability of the device until the construction of the rock anchors was complete by pumping water into high level tanks.

The resulting transverse section shape was presented in NEL Report PR18 in November 1979 (Reference 11). However, due to the increase in water column dimensions that had occurred during the evolution of the structure, the volume of structural concrete required had not significantly decreased compared with the 1978 commencing point. At the same time further information regarding availability of power at the inshore bottom standing sites had resulted in the final cost of power produced being only half the cost of the Floating Reference Design as opposed to the 1/5th factor that had been anticipated at the time of commencing the detailed study.

In the search for further economy a review of the efficiency of the water column in terms of its main dimensional parameters showed that a larger water column area combined with increased damping in the form of a reduction in turbine size would provide increased output. This had the benefit of allowing a more compact power generation equipment layout to be located directly on top of the water column roof slab, which then removed the necessity for any structure at the rear of the water column. However, this meant a reduction in water plane and an increase in draft so that once again the floating draft became too large for direct emplacement.

The function of the straddle barge was, therefore, extended to also provide extra buoyancy sufficient to raise the structure to the minimum acceptable draft for emplacement. The end result of these studies is the Reference Design presented in this report.

The above section on the evolution of the structure for the Breakwater Device shows the complexity of the design process, and how small changes in the conceptual arrangements create larger changes in one or other of the controlling parameters in the design process.

A summary of the calculated stability parameters for the Reference Design in both floating and fixed modes is given in Appendix III.

9.3 Rock Anchors and Foundations

In contrast to floating devices, foundation design for the bottom standing Breakwater Device is relatively more important. However, at the South Uist site, it is made considerably simpler in some respects due to the presence of the strong and homogeneous Lewisian Gneiss immediately under the seabed. The best available information at the time of writing suggests that the rock is generally sound with a very shallow zone of weathering at its surface. Therefore, assuming that even contact between the base of the structure and the exposed rock surface is obtained by the underbase grouting, the pressures exerted by the structure are well within the capacity of the rock at seabed level.

The design calculations carried out for this report show maximum pressures under the base of up to 150 kN/m^2 which may be compared with the recommended permissible maximum of 10,000 kN/m^2 given in British Standard CP2004 (Reference 12) for this type of rock.

Resistance to sliding is considered to be provided by friction between the base of the structure and the bed rock, with the vertical component of the rock anchor loading being necessary to increase the net downwards load so that sufficient frictional resistance is mobilised. The value of 0.45 taken for the coefficient of friction is reasonably conservative, in comparison to values of 0.65 to 0.90 used in the verification of wave pressure formulae in Japan (Reference 13).

Although the contact area between the base of the structure and the prepared rock surface is likely to be uneven immediately after emplacement, it is envisaged that local overstressing and resulting deformation of both surfaces will quickly result in a more even and adequate contact area. At a later stage, the cement grout will increase the area of contact and may also provide additional horizontal resistance by a shear mechanism.

The presence of the Lewisian Gneiss also facilitates the use of rock anchors because the homogeneous and massive rock provides good anchorage with reasonably short anchor lengths.

The working load capacity of the anchors was chosen taking account of stresses in the structure and in the foundation, the availability and size of installation and tensioning equipment and the necessity for rapid installation. The selected capacity of 360t is provided by a tendon consisting of 29 no. 15.4mm diameter high strength prestressing strands.

The overall stability of the structure against sliding and overturning is provided by 90 vertical anchors situated in the rear and transverse walls, and 60 inclined anchors in the base of the water columns. The stability of the structure/rock slab system is assured by inserting the anchors to a depth of about 25m.

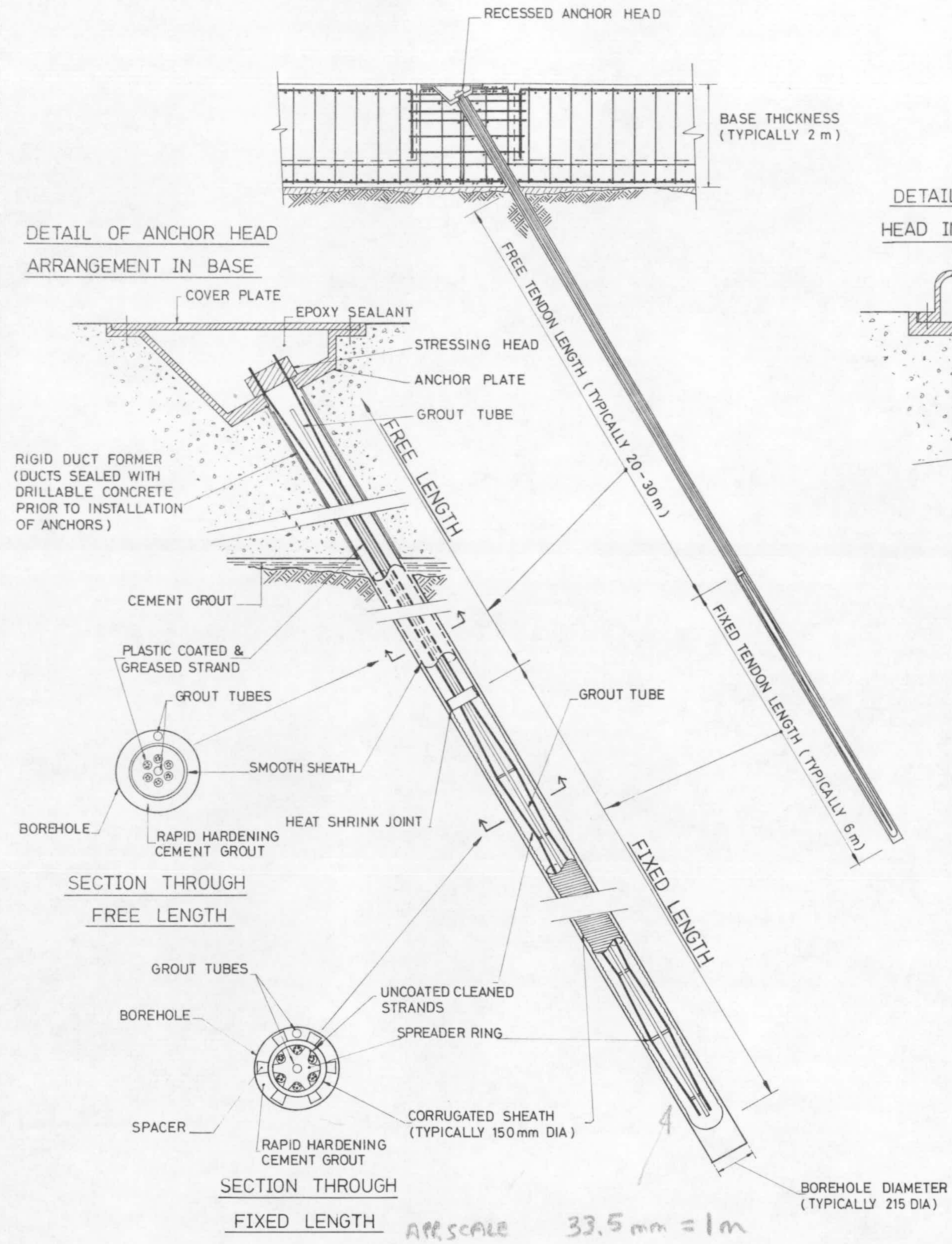
A detailed evaluation of the anchors was carried out by Colcrete Ltd (Reference 14). They made preliminary recommendations for borehole diameter, grout strength and fixed anchor length so that a minimum factor of safety of 2.0 against failure is obtained not only in each individual anchor, but also in the overall structure/anchor/rock mass system. Details of the rock anchors are shown in Drawing No. 3RD/104.

An important aspect of the design of the rock anchors is the provision of protection against corrosion of the tendon. This must be sufficient in order to provide a maintenance free working life of at least 25 years. In the fixed length at the bottom of the anchor, the tendon is encased within a corrugated plastic sheath. A high strength epoxy or polyester resin grout both inside and outside the sheath provides not only the stress transfer from the tendon to the rock, but also adds an extra layer of protection. In the free length up to the anchor stressing head in the structure, individual strands in the tendon are greased and sheathed in 1 to 2mm thick polythene. The composite tendon is then enclosed in a plain polythene outer sheath of similar diameter to the corrugated sheath in the fixed length. In the free length, only the area outside the sheath is grouted but the resulting annulus does provide another stage of corrosion protection. The protection of the anchor head is provided by epoxy based sealant encapsulations. In the walls the heads are encapsulated in steel domes which are bolted onto the anchor plates and then pressure filled with the epoxy sealant. In the base of the water columns, the heads are recessed into the concrete and covered with plates which are flush with the adjacent concrete surface. There is therefore a minimum of two corrosion prevention barriers in any part of the proposed rock anchor arrangement.

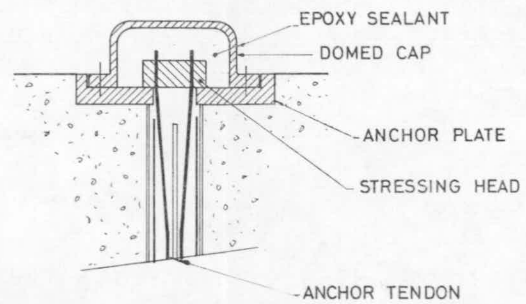
The long term behaviour of the production rock anchors is also relevant. Previous experience (Reference 15) shows that when anchors are installed into good competent rock, loss of prestress due to non tendon phenomenon is relatively small. In addition, the type of strand chosen has excellent long term relaxation characteristics, which are enhanced by the choice of a prestress level between 50 and 60% of the characteristic strength of the tendon (compared with prestress levels of between 70 and 80% used in more conventional prestressed concrete applications).

The rock anchor and foundation proposals outlined above have been specially developed for the conditions that apply at South Uist. However, it is more than likely that rock will not be present in other suitable locations, and therefore different foundation designs will be necessary.

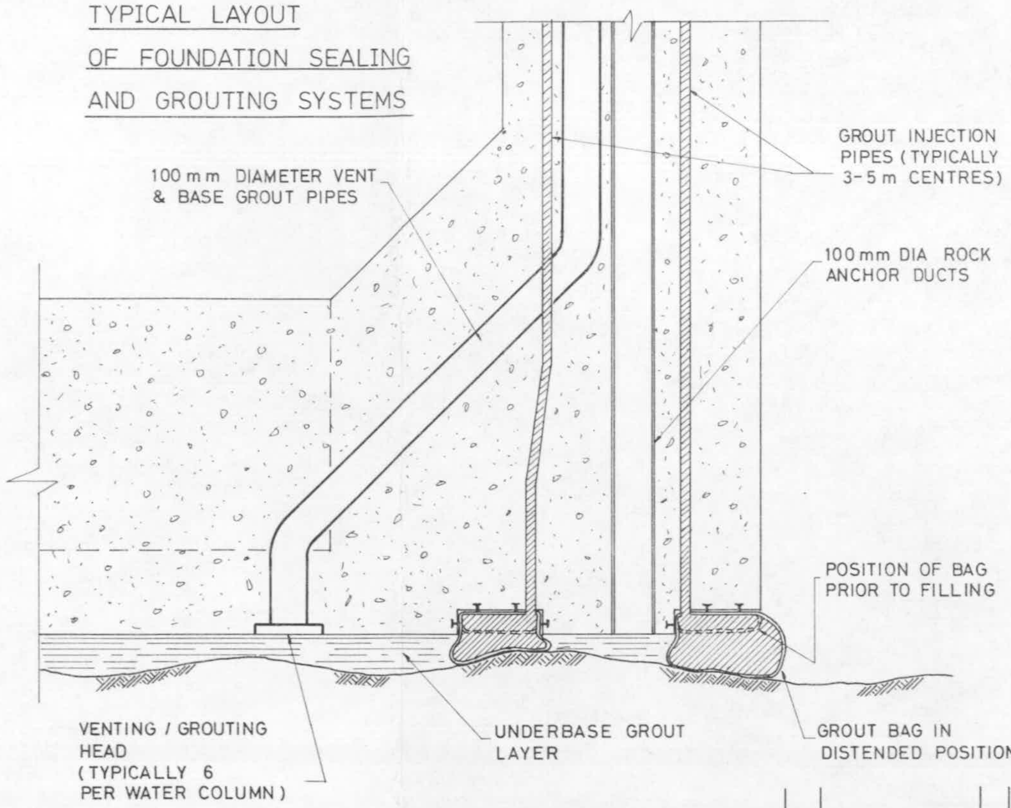
TYPICAL LAYOUT OF ROCK ANCHOR
SHOWING ARRANGEMENT OF ANCHORAGE
REINFORCEMENT FOR BASE ANCHORS

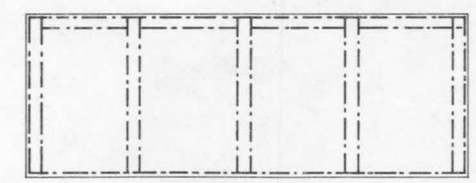


DETAIL OF ANCHOR HEAD IN TOP OF WALL



TYPICAL LAYOUT OF FOUNDATION SEALING AND GROUTING SYSTEMS





LOCATION OF GROUT SEALING BAGS ON UNDERSIDE OF BASE

(LOCATION SHOWN THUS —·—·—)

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NATIONAL ENGINEERING LABORATORY
OSCILLATING WATER COLUMN (WAVE PISTON)

MARCH 1980
REFERENCE DESIGN
BREAKWATER TYPE

DETAILS OF ROCK ANCHORS & FOUNDATION SEALING & GROUTING SYSTEMS

ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST
PAISLEY SCOTLAND
Tel. 041-889-0044
Telex 779684 ROXBJR G
ABERDEEN LONDON

Job №	Drg №	Revision
1678/71	3RD/104	
Scale nts	Date APRIL '80	
Drawn By	Passed By	Issued For
RPR	R	MARCH 80 REPORT

In softer seabeds horizontal sliding stability can be improved by the provision of a steel skirt around the perimeter of the base. In the event that tension ties (i.e. the rock anchors used in this Reference Design) are necessary, recourse can be made to lower capacity ground anchors (designs for which are available for both cohesive and non cohesive soil types) or to larger diameter bored or driven piles. However, the use of these or any other applicable alternatives would probably change the structural arrangement of the device, and therefore the design of the structure will have to be re-assessed for each type of foundation material.

9.4 Structural Elements

The primary structure for the Breakwater Device is relatively simple, consisting of fairly thick reinforced concrete continuous slab elements for the base and walls, reinforced concrete beam and slab construction for the column roof and a pre-cast prestressed concrete hollow section nose arrangement. Details of Reinforcements and Precast Elements are shown in Drawing Nos. 3RD/102 and 3RD/103.

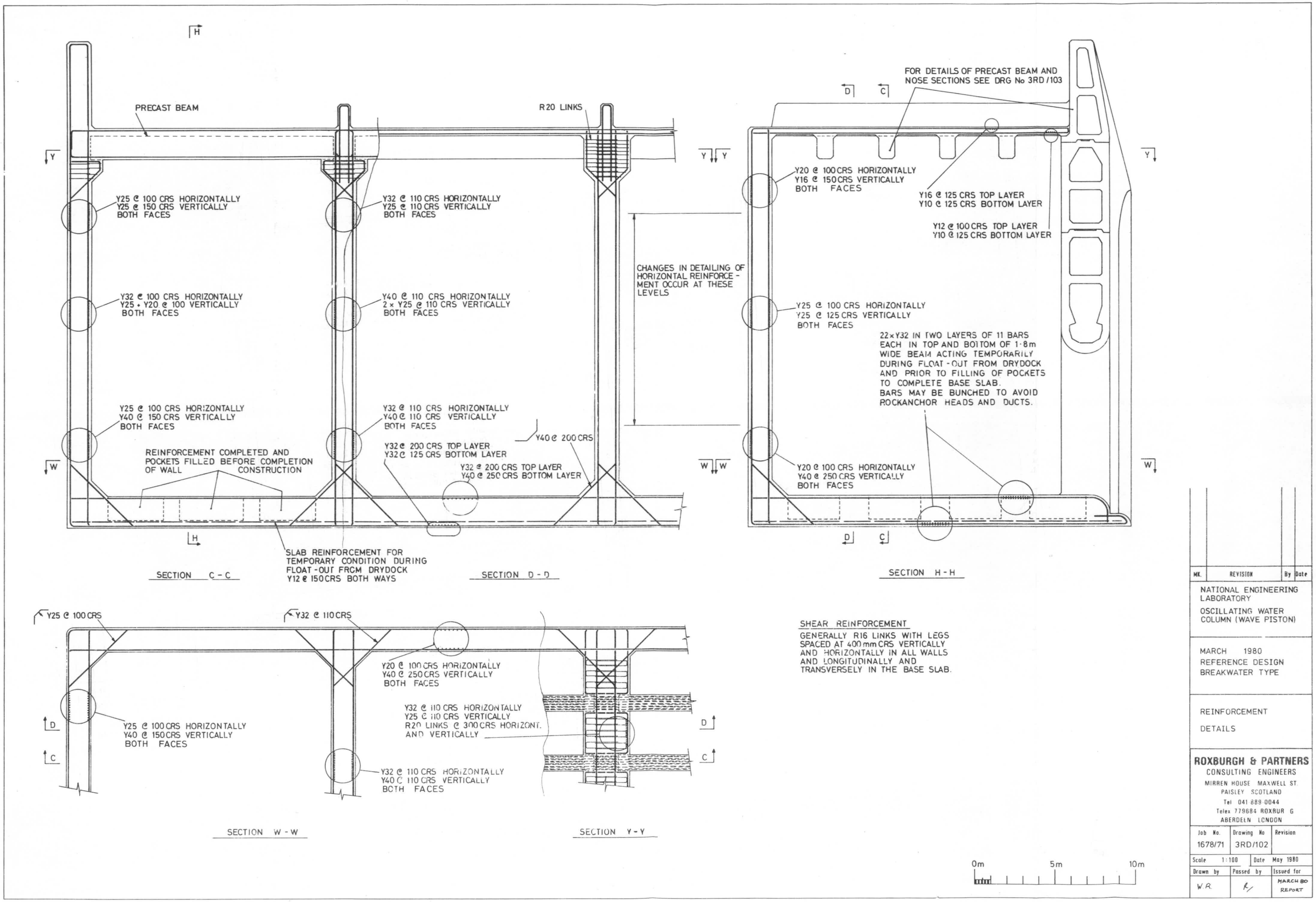
The detailed design considerations for each of the elements are as follows:

(i) Base

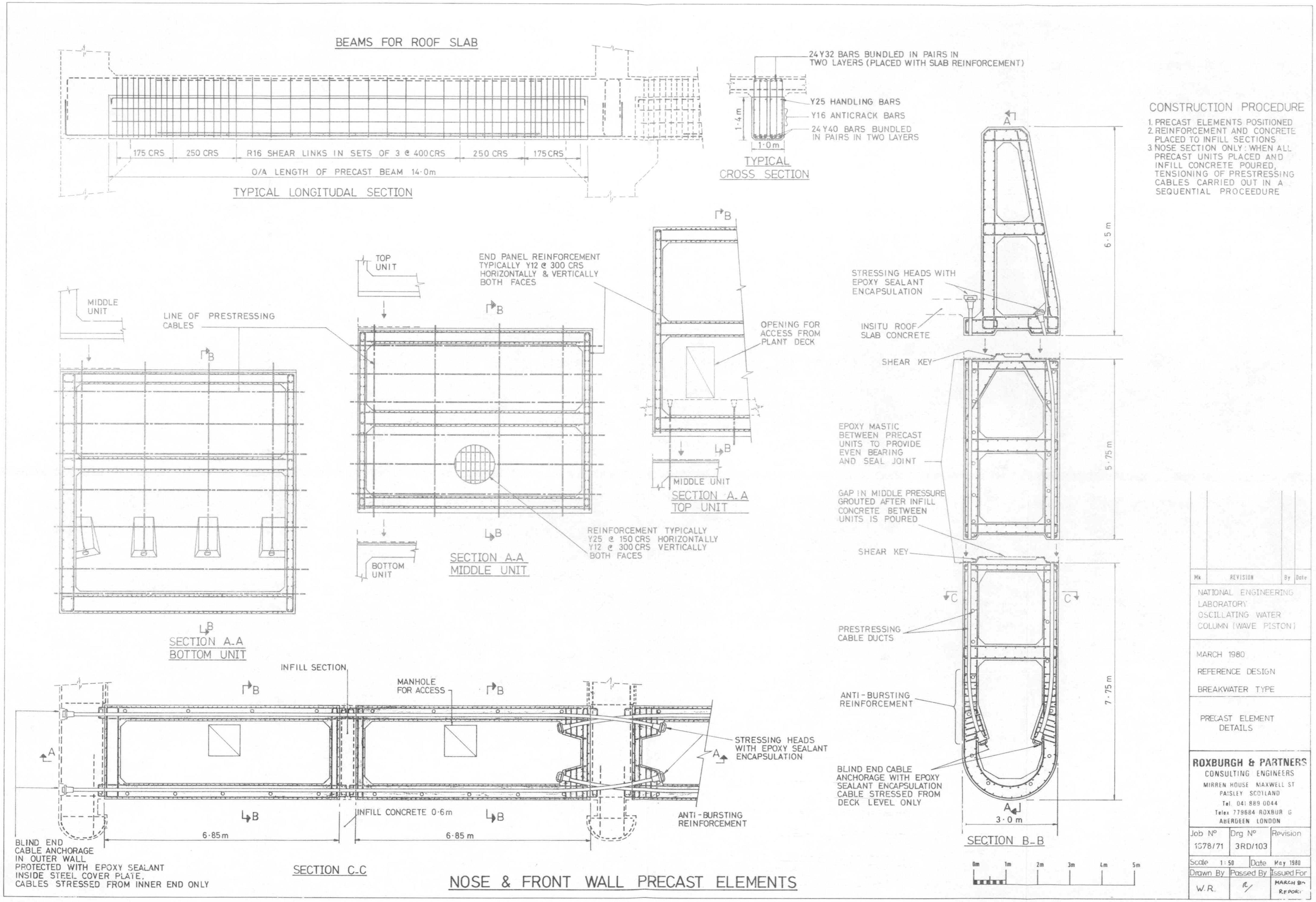
The base is designed for the transportation and operation phases as a continuous slab. However, during the construction phase it is necessary to use a reduced thickness in order to lighten the structure at float out from drydock, and the resulting thin slab is stiffened with a set of beams. These beams are coincident with the rock anchors that are located in the base and therefore help to spread the local stresses arising from the anchors.

The two most important loading phases found were, firstly, the stresses arising from the local hydrostatic and dynamic pressures which result from the device acting like a ship during the transportation phase, and secondly, the upwards pull of the transverse walls against the restraining action of the rock anchors on the base when the structure is being loaded by the extreme horizontal wave force.

This latter condition gives the maximum ultimate bending moment on the base, with a resulting reinforcement area of 0.9% of the overall section. The transportation phase provides the largest cracking serviceability moment which is satisfied by the above steel quantity. Due to the even distribution of rock anchors in the base, fatigue is not considered to be a problem.



PRECAST BEAM
R20 LINKS
Y25 @ 100 CRS HORIZONTALLY
Y25 @ 150 CRS VERTICALLY
BOTH FACES
Y32 @ 110 CRS HORIZONTALLY
Y25 @ 110 CRS VERTICALLY
BOTH FACES
Y32 @ 100 CRS HORIZONTALLY
Y25 + Y20 @ 100 VERTICALLY
BOTH FACES
Y40 @ 110 CRS HORIZONTALLY
2 x Y25 @ 110 CRS VERTICALLY
BOTH FACES
CHANGES IN DETAILING OF HORIZONTAL REINFORCE - MENT OCCUR AT THESE LEVELS
Y25 @ 100 CRS HORIZONTALLY
Y40 @ 150 CRS VERTICALLY
BOTH FACES
Y32 @ 110 CRS HORIZONTALLY
Y40 @ 110 CRS VERTICALLY
BOTH FACES
Y40 @ 200 CRS
REINFORCEMENT COMPLETED AND POCKETS FILLED BEFORE COMPLETION OF WALL CONSTRUCTION
Y32 @ 200 CRS TOP LAYER
Y32 @ 125 CRS BOTTOM LAYER
Y32 @ 200 CRS TOP LAYER
Y40 @ 250 CRS BOTTOM LAYER
SLAB REINFORCEMENT FOR TEMPORARY CONDITION DURING FLOAT - OUT FROM DRYDOCK
Y12 @ 150 CRS BOTH WAYS
SECTION C - C
SECTION D - D
FOR DETAILS OF PRECAST BEAM AND NOSE SECTIONS SEE DRG No 3RD /103
Y20 @ 100 CRS HORIZONTALLY
Y16 @ 150 CRS VERTICALLY
BOTH FACES
Y16 @ 125 CRS TOP LAYER
Y10 @ 125 CRS BOTTOM LAYER
Y12 @ 100 CRS TOP LAYER
Y10 @ 125 CRS BOTTOM LAYER
Y25 @ 100 CRS HORIZONTALLY
Y25 @ 125 CRS VERTICALLY
BOTH FACES
22 x Y32 IN TWO LAYERS OF 11 BARS EACH IN TOP AND BOTTOM OF 1·8m WIDE BEAM ACTING TEMPORARILY DURING FLOAT - OUT FROM DRYDOCK AND PRIOR TO FILLING OF POCKETS TO COMPLETE BASE SLAB.
BARS MAY BE BUNCHED TO AVOID ROCKANCHOR HEADS AND DUCTS.
Y20 @ 100 CRS HORIZONTALLY
Y40 @ 250 CRS VERTICALLY
BOTH FACES
SECTION H - H
Y25 @ 100 CRS
Y32 @ 110 CRS
Y20 @ 100 CRS HORIZONTALLY
Y40 @ 250 CRS VERTICALLY
BOTH FACES
Y25 @ 100 CRS HORIZONTALLY
Y40 @ 150 CRS VERTICALLY
BOTH FACES
Y32 @ 110 CRS HORIZONTALLY
Y25 @ 110 CRS VERTICALLY
R20 LINKS @ 300 CRS HORIZONT.
AND VERTICALLY
Y32 @ 110 CRS HORIZONTALLY
Y40 @ 110 CRS VERTICALLY
BOTH FACES
SECTION W - W
SECTION Y - Y
SHEAR REINFORCEMENT
GENERALLY R16 LINKS WITH LEGS SPACED AT 400 mm CRS VERTICALLY AND HORIZONTALLY IN ALL WALLS AND LONGITUDINALLY AND TRANSVERSELY IN THE BASE SLAB.
0m
5m
10m
MK. REVISION By Date
NATIONAL ENGINEERING LABORATORY
OSCILLATING WATER COLUMN (WAVE PISTON)
MARCH 1980
REFERENCE DESIGN
BREAKWATER TYPE
REINFORCEMENT DETAILS
ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST.
PAISLEY SCOTLAND
Tel 041 889 0044
Telex 779684 ROXBUR G
ABERDEEN LONDON
Job No. 1678/71
Drawing No 3RD/102
Revision
Scale 1:100
Date May 1980
Drawn by W.R.
Passed by
Issued for MARCH 80 REPORT



BEAMS FOR ROOF SLAB
175 CRS
250 CRS
R16 SHEAR LINKS IN SETS OF 3 @ 400CRS
250 CRS
175 CRS
O/A LENGTH OF PRECAST BEAM 14·0m
TYPICAL LONGITUDAL SECTION
24Y32 BARS BUNDLED IN PAIRS IN TWO LAYERS (PLACED WITH SLAB REINFORCEMENT)
Y25 HANDLING BARS
Y16 ANTICRACK BARS
24Y40 BARS BUNDLED IN PAIRS IN TWO LAYERS
1·4m
1·0m
TYPICAL CROSS SECTION
CONSTRUCTION PROCEDURE
1. PRECAST ELEMENTS POSITIONED
2. REINFORCEMENT AND CONCRETE PLACED TO INFILL SECTIONS
3. NOSE SECTION ONLY: WHEN ALL PRECAST UNITS PLACED AND INFILL CONCRETE POURED, TENSIONING OF PRESTRESSING CABLES CARRIED OUT IN A SEQUENTIAL PROCEEDURE
MIDDLE UNIT
LINE OF PRESTRESSING CABLES
TOP UNIT
END PANEL REINFORCEMENT TYPICALLY Y12 @ 300 CRS HORIZONTALLY & VERTICALLY BOTH FACES
OPENING FOR ACCESS FROM PLANT DECK
MIDDLE UNIT
SECTION A-A TOP UNIT
REINFORCEMENT TYPICALLY Y25 @ 150 CRS HORIZONTALLY Y12 @ 300 CRS VERTICALLY BOTH FACES
BOTTOM UNIT
SECTION A-A MIDDLE UNIT
SECTION A-A BOTTOM UNIT
STRESSING HEADS WITH EPOXY SEALANT ENCAPSULATION
INSITU ROOF SLAB CONCRETE
SHEAR KEY
6·5 m
EPOXY MASTIC BETWEEN PRECAST UNITS TO PROVIDE EVEN BEARING AND SEAL JOINT
GAP IN MIDDLE PRESSURE GROUTED AFTER INFILL CONCRETE BETWEEN UNITS IS POURED
5·75 m
SHEAR KEY
PRESTRESSING CABLE DUCTS
ANTI-BURSTING REINFORCEMENT
7·75 m
BLIND END CABLE ANCHORAGE WITH EPOXY SEALANT ENCAPSULATION CABLE STRESSED FROM DECK LEVEL ONLY
3·0 m
SECTION B-B
INFILL SECTION
MANHOLE FOR ACCESS
STRESSING HEADS WITH EPOXY SEALANT ENCAPSULATION
INFILL CONCRETE 0·6m
ANTI-BURSTING REINFORCEMENT
6·85 m
6·85 m
BLIND END CABLE ANCHORAGE IN OUTER WALL PROTECTED WITH EPOXY SEALANT INSIDE STEEL COVER PLATE. CABLES STRESSED FROM INNER END ONLY
SECTION C-C
NOSE & FRONT WALL PRECAST ELEMENTS
0m 1m 2m 3m 4m 5m
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NATIONAL ENGINEERING LABORATORY
OSCILLATING WATER COLUMN (WAVE PISTON)
MARCH 1980
REFERENCE DESIGN
BREAKWATER TYPE
PRECAST ELEMENT DETAILS
ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST
PAISLEY SCOTLAND
Tel 041 889 0044
Telex 779684 ROXBUR G
ABERDEEN LONDON
Job No 1378/71
Drg No 3RD/103
Revision
Scale 1:50
Date May 1980
Drawn By W.R.
Passed By
Issued For MARCH 80 REPORT

(ii) Rear and Transverse Walls

The main loadings considered for the walls are, firstly, as for the base, hydrostatic and dynamic loads under transportation conditions, and secondly, various combinations of external wave loading and internal water column pressures in the operational phase. Included in this latter group is a maintenance condition with one water column empty under normal environmental wave loadings, and an accidental case of one column empty under extreme environmental wave loadings.

The analysis of the loadings resulted in steel areas of 0.75% maximum in the rear wall, 1.0% in the end transverse walls and 1.4% in the internal transverse walls. Large 45^{o} angle splays are detailed at continuous edges in order to reduce the peak local bending moments occurring at these points.

In general the choice of wide concrete sections with fairly low reinforcement areas has meant that all the various ultimate and serviceability limit states are satisfied without requiring extra reinforcement or section depth to satisfy one individual state. The resulting sections are therefore very economical in their use of ordinary reinforcement.

A further factor in favour of low reinforcement quantities in the walls is the ease of placement during the slipforming process. This necessitates bars with diameters and lengths small enough to be placed quickly by hand. The use of only one layer of reinforcement is also considered preferable.

The use of prestressing was considered in order to reduce the overall reinforcement area. However, the loadings in the operational phase are cyclic in nature with almost equal amplitude in either direction. This means that concentric prestress must be provided, which nullifies most of the advantages normally obtained by the use of eccentric prestressing. In addition, for this structure, it would also have been necessary to increase the width of the concrete section in order to reduce to acceptable levels the maximum compressive stresses resulting from the combination of prestress and design loads.

(iii) Nose Section

The main loadings on the nose section occur in the operational phase. Although they are cyclic in nature, the magnitude when a wave crest is at the face of structure is considerably greater than when a trough is at the structure. Therefore it is possible to use eccentric prestressing to obtain a reduction in concrete section. Thus, the nose was detailed in precast hollow sections, which are held together and located in place in the structure by post tensioned prestressing cables.

The hollow sections are designed on the assumption that they are full of water. This has been done partly to reduce differential hydrostatic loadings and partly to modify the natural frequency of the section which could be close to the breaking wave impulse period. Flooding will be ensured through the access manholes, with additional ducts or inlets located in the front face to provide topping up as necessary.

The resulting prestress provisions are 19/15 Dyform strands horizontally at 1m centres in the rear panel and 1.3m centres in the front panel. Similar vertical prestress at 2.0m and 1.5m centres respectively is intended mainly to hold the cantilever top section in place and to keep the joints between the other sections firmly closed. The tendon anchorage heads are all fully encapsulated in a similar fashion to the rock anchors.

(iv) Water Column Roof

The maximum loadings on the water column roof are caused by the air pressures inside the column during its operating cycle. Due to the reversal of stresses, reinforced concrete was once again found to offer the most economical solution.

In order to facilitate construction operations the roof beams are precast, and are designed as simply supported in order to avoid the need for continuity reinforcement at the supports. The associated deck slab is then cast directly onto permanent concrete plank soffit shutters supported by the beams.

The extreme environmental and system loading phase gave the maximum moments resulting in reinforcement areas of 1.5% in the beams, and 0.4% in the main transverse direction in the slab.

10. SITE PREPARATION

The first operation carried out in the construction of a power station using Breakwater Devices is a detailed site survey. This survey includes establishment of suitable surface reference points and subsea transponder stations, a detailed topographical survey using direct and side scan sonar, and a geological investigation of the bed rock.

Following the survey, the detailed layout of the devices is determined taking account of the necessity to following the contour of the seabed, to provide navigation gaps (at gullies where possible), and to facilitate changes in the foundation level to minimise the amount of rock cut.

Work at the site commences with the clearance of sea weed and other loose material by a combination of pressure jetting and conventional dredging methods. Any protruding rock outcrops are removed at this stage by selective blasting. The rock surface is then fractured to near the design foundation level by either plaster shooting or by drilling and blasting explosive methods. The width of the fractured strip is approximately 35m wide to allow reasonable emplacement tolerances.

With the initial preparation of the seabed complete, the fractured rock is removed and the foundation surface trimmed to the required tolerance by a special rock cutting dredger. This dredger is similar to the 'Simon Stevin' (Figure 10.1), which is a semi-submersible jack up walking dredger developed by a Dutch dredging contractor for use in offshore locations (Reference 16).

'Simon Stevin' has the capability of cutting a strip up to 80m wide in depths of 30m while subject to waves of 2.5m significant height. It can remain on station on its jack up legs in waves of up to 4.0m significant height, and is capable of weathering worse storms under its own power when floating in its semi-submersible mode.

The dredger developed for the wave energy contract would be fitted with a hard or rock cutting head and would be capable of cutting to a 20cm level tolerance over the required 35m width at the specified 15 to 20m depth range.

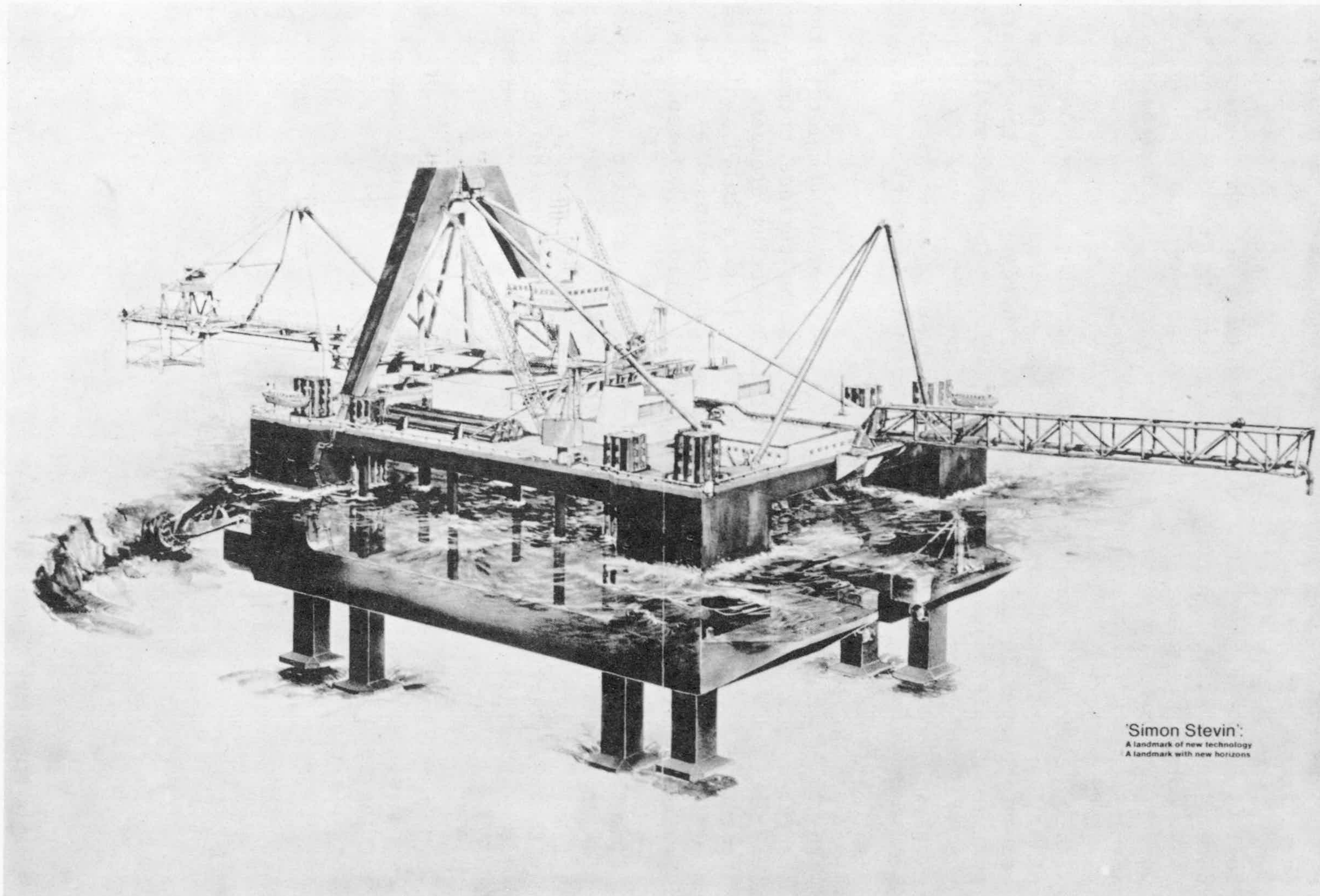


FIG. 10.1 'SIMON STEVIN' DREDGER

11. CONSTRUCTION OF THE STRUCTURE

This section is based on the premise that most of the concrete construction will be by insitu methods. However, it would appear that significant economies could be achieved by using pre-casting techniques and assembly line methods. These methods merit further research and would probably result in lower costs being estimated for construction.

Construction of the concrete primary structures for a Breakwater Power Station will take place in two phases at various oil platform yards located throughout Scotland i.e. Nigg on the East Coast and Ardyne, Hunterston, Portavadie and Kishorn on the West Coast (Figure 11.1).

The first phase is the construction of part of the base and lower walls in dry conditions within the basins, followed by a second phase of completion of the walls, installation of the various precast members and completion of outstanding insitu concrete pours at a sheltered inshore floating berth adjacent to the yard.

Several of the yards require installation of removable dock gates. This would be done concurrently with the construction of the first units.

The sequence of operations in the first phase in the drydock is as follows. The base slab, with large portions omitted in order to lighten the structure at float out, and the bottom portion of the walls are constructed using conventional insitu methods. Then the main rear and transverse walls are slip formed to a height of approximately 13m. The removable steel bulkheads that are used to seal the water column mouths are positioned and the structure is ballasted down ready for flooding of the basin. When water has penetrated evenly under the base of the structure, it is de-ballasted carefully so that it floats with a draft of about 9m. The dock gate is then opened and the second phase of the construction commences with float out of the structure to the deep water construction berth adjacent to the yard.

Prior to commencing construction in the first phase, careful preparation of the base of the dock is required. A 500mm thick layer of gravel is placed over the area of the structure to provide an even substrate and to allow the water pressure to distribute freely under the base at float out. The gravel is covered by a polythene or hardboard membrane to prevent loss of cement grout during concreting of the base.

The heavy duty inflatable tubes required to seal the edges of the foundation after emplacement and their associated grout pipes, a second set of grout and vent pipes for the underbase grouting and the ducts for the rock anchors are all carefully located during concreting. The rock anchor ducts in the base are plugged with drillable concrete prior to float out, while those in the walls are left open to assist water penetration at float out and venting at emplacement.

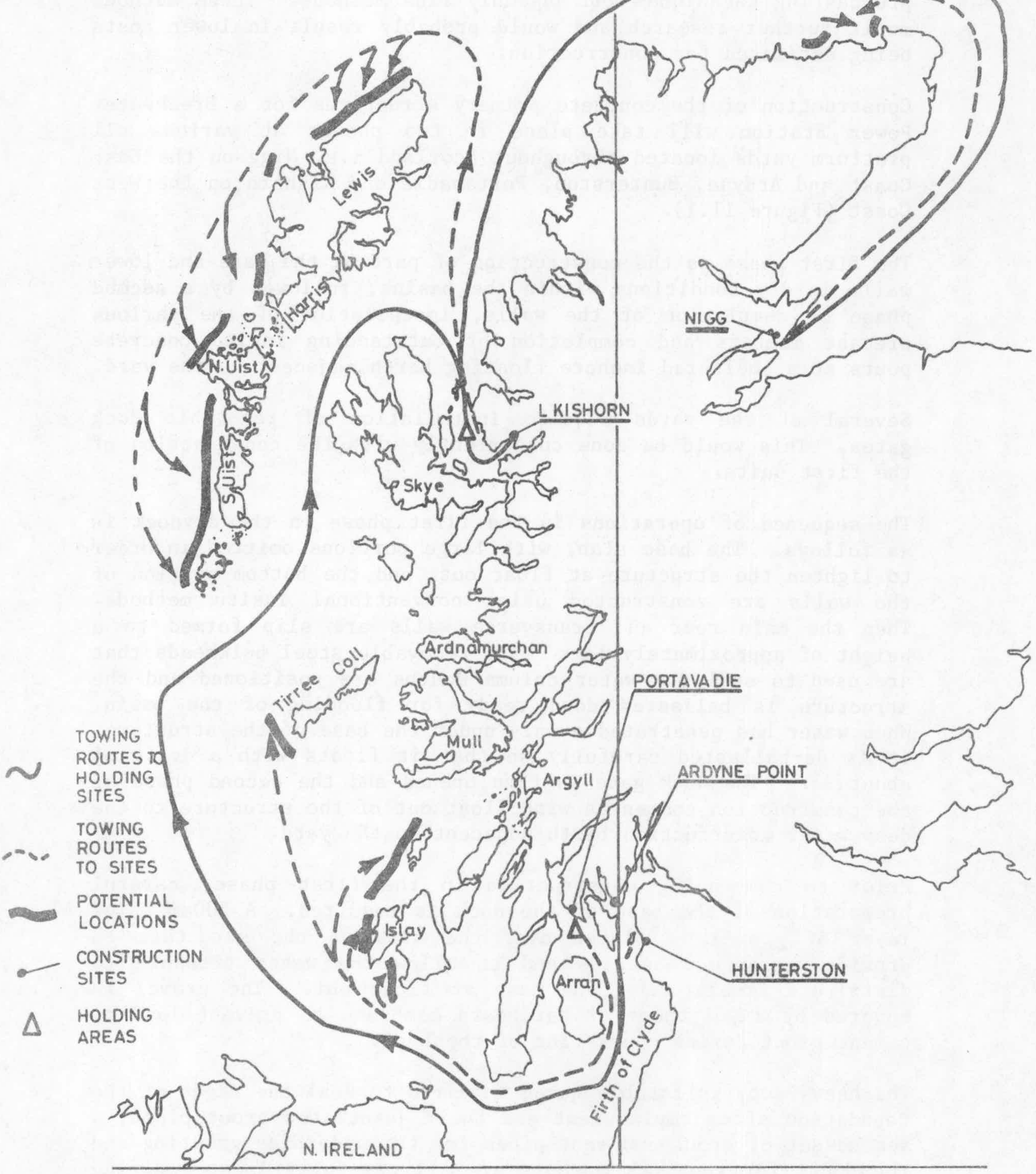


FIG 11:1 CONSTRUCTION AND ASSEMBLY SITES

Following float out from the basin the second phase of the construction is carried out from a fixed or floating jetty. The slipforming of the walls and the infill portions of the base are completed, the precast nose sections and roof support beams are located and insitu joints made, and the insitu column roof slab poured onto permanent precast concrete plank soffit shutters.

During this phase careful control of the equilibrium of the floating structure is required, particularly during slip forming operations. This is facilitated by additional concrete block kentiledge which is positioned as required on the floor of the water column. This kentiledge also allows final trimming to be carried out prior to tow out to the installation site.

In order to facilitate prestressing of the nose sections in one operation, the insitu joints between the precast units are conventionally reinforced so that the structure can resist all loadings likely to be imposed during the construction operations. This allows all the nose sections to be positioned and the roof slab to be poured before commencing any prestressing work, thus minimising problems with differential strain.

Finally, before tow out to the holding area, and eventually the emplacement site, the various openings in the water column roof slab are sealed to ensure the watertight integrity of the structure during its open sea journey.

12. INSTALLATION

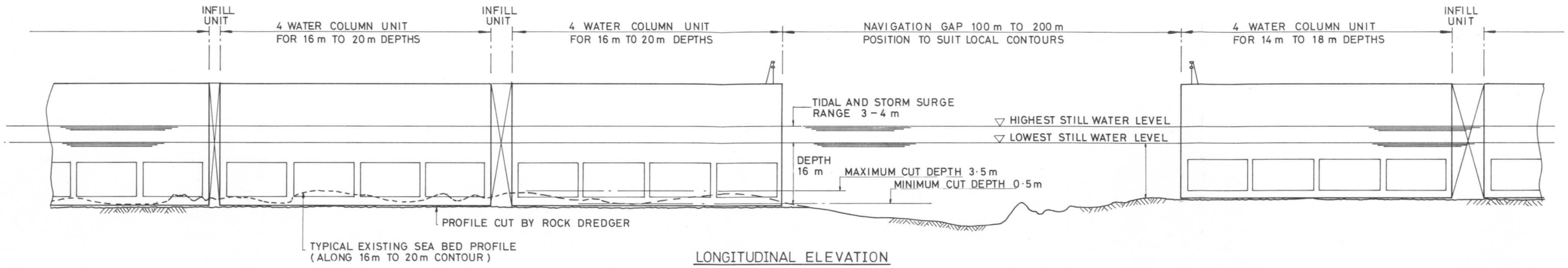
12.1 Installation of the Structure

The completed caisson-like structure, with its bulkhead gates in place and the bypass valves and duct openings on the roof closed off, is a watertight box with a draft of approximately 21m. It is towed by two 10,000 hp tugs from the construction site to a holding area situated at the Crowlin Islands near Loch Kishorn. The routes between the construction site and the holding area will be specially surveyed to identify wide obstruction free channels. Site locations, tow routes and installation procedure are shown in Drawing Nos. 3RD/100 and 3RD/105.

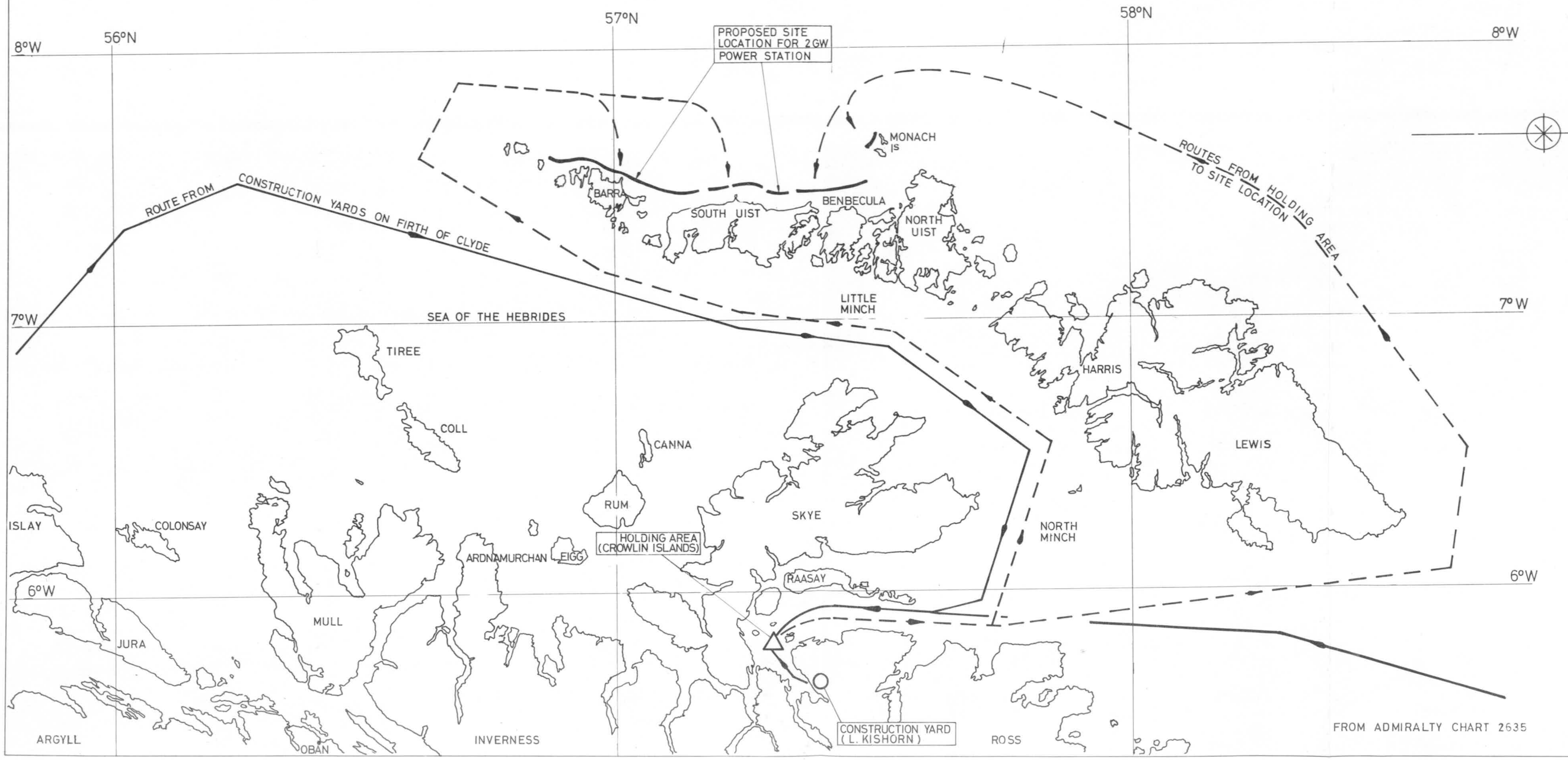
At the holding area the straddle barge for emplacement is connected to the floating Breakwater structure. The barge is a specially constructed catamaran type vessel with pontoons connected by large portal frames. The frames support a high level working deck which provides accommodation, workshops, storage space and extra ballast tanks. The straddle barge is connected to the floating Breakwater structure by being floated over and ballasted down onto the Breakwater Device. When the two structures have been rigidly connected together, the straddle barge is deballasted until a maximum draft of 12m under the composite structure is achieved. The composite structure is then towed to the emplacement site by two large tugs.

At the emplacement site the large tugs are replaced by four smaller ships to carry out the precise manoeuvering required for caisson installation. The site is located by use of the transponders installed prior to the site preparation. Once in position the caisson is placed quickly unto the prepared seabed foundation surface by flooding the water columns and ballasting the pontoons of the straddle barge. In order to provide sufficient stability after emplacement against storms that occur before completion of installation of the rock anchors, ballasting of the pontoons and additional tanks in the working deck is continued until they are all full. This provides adequate security against a storm with an annual return period.

Following completion of the ballasting operations the installation of the rock anchors (Reference 14) commences using the emplacement barge as a working platform and support facility. Drilling of the 215mm diameter rock anchor hole in the anticipated strong Lewisian Gneiss bedrock is carried out using a 'down-the-hole' percussive drilling hammer located immediately above a rotary drill bit. The hammer is powered by high pressure compressed air ducted through the drill stem, with rotary action of the drill being imparted by the rig at the top of the hole.



LONGITUDINAL ELEVATION



Mk	REVISION	By	Date

NATIONAL ENGINEERING LABORATORY
OSCILLATING WATER COLUMN (WAVE PISTON)

MARCH 1980

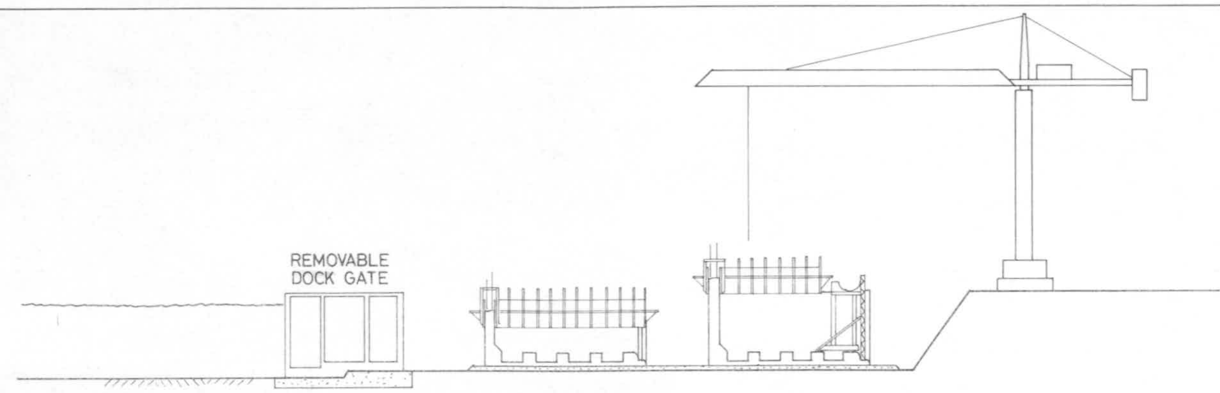
REFERENCE DESIGN

BREAKWATER TYPE

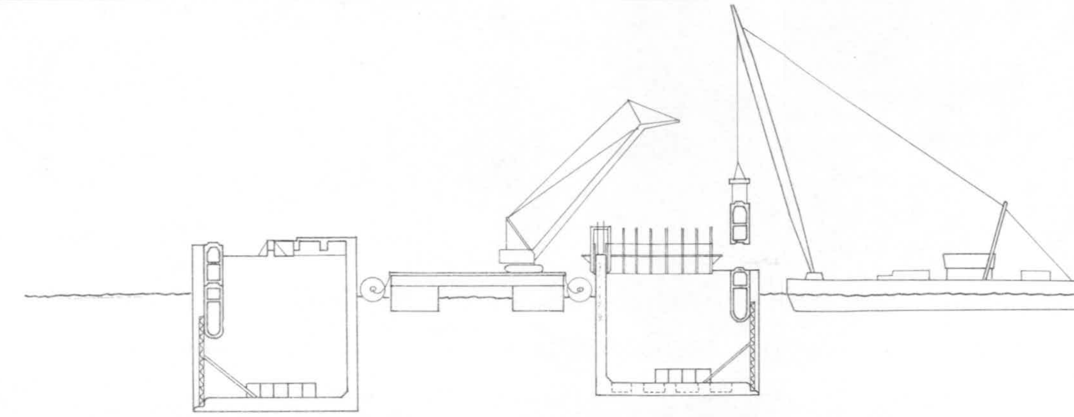
SITE LOCATION AT OUTER HEBRIDES AND LONGITUDINAL ELEVATION

ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST
PAISLEY SCOTLAND
Tel. 041 889 0044
Telex 779684 ROXBUR G
ABERDEEN LONDON

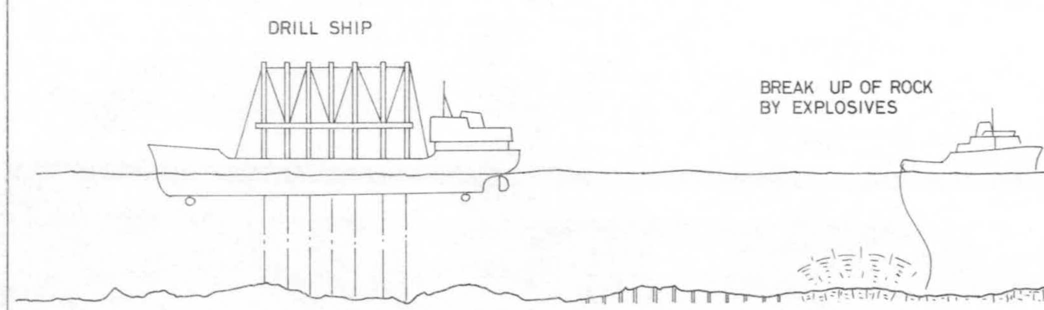
Job Nº	Drg Nº	Revision
1678/71	3RD/100	
Scale n.t.s	Date APRIL '80	
Drawn By	Passed By	Issued For
RPR		MARCH 80 REPORT



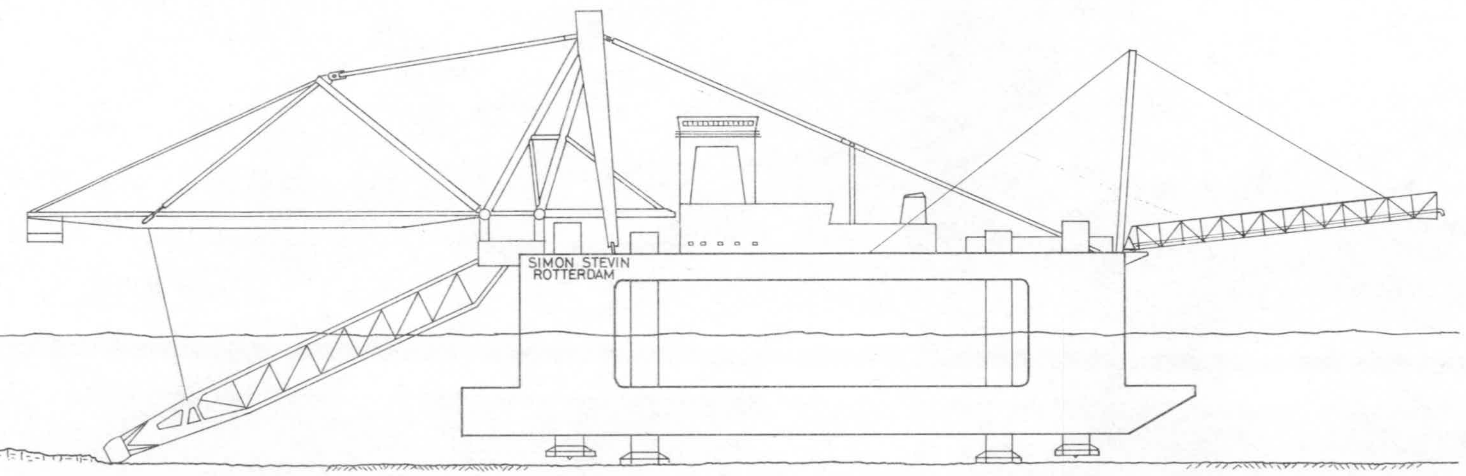
1. CONSTRUCTION IN DRYDOCK



2. COMPLETION OF CONSTRUCTION AT
INSHORE SHELTERED FLOATING BERTH

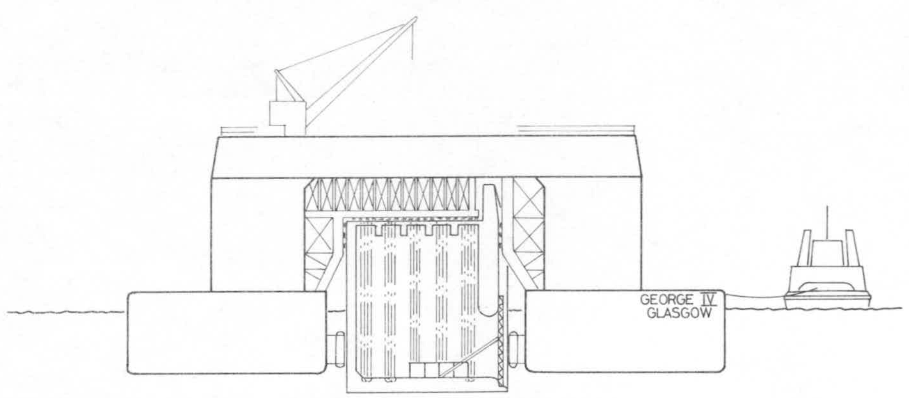


(a) Initial preparation of seabed

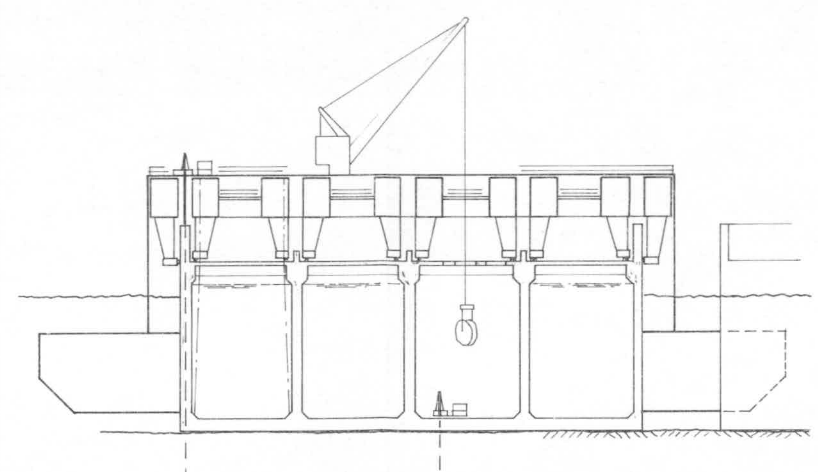


(b) Simon Stevin trimming formation

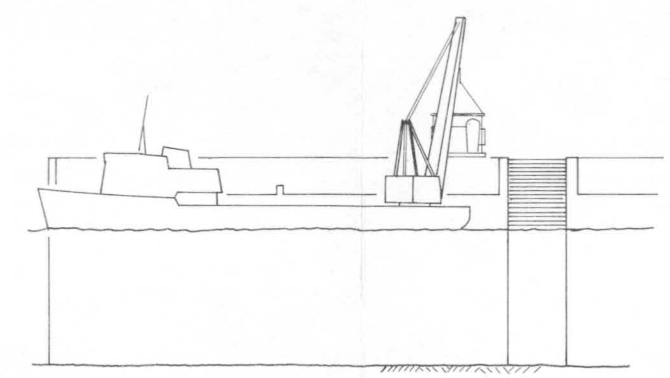
3. SITE PREPARATION



4. TOW OUT FROM HOLDING AREA
TO SITE LOCATION



(a) Installation of rock anchors



(b) Installation of M & E plant

5. POST EMPLACEMENT OPERATIONS

0m 10m 20m 30m

MK.	REVISION	By	Date

NATIONAL ENGINEERING LABORATORY

OSCILLATING WATER COLUMN (WAVE PISTON)

MARCH 1980
REFERENCE DESIGN
BREAKWATER TYPE

CONSTRUCTION &
INSTALLATION
PROCEDURE

ROXBURGH & PARTNERS
CONSULTING ENGINEERS
MIRREN HOUSE MAXWELL ST
PAISLEY SCOTLAND
Tel 041 889 0044
Telex 779684 ROXBUR G
ABERDEEN LONDON

Job No.	Drawing No	Revision
1678/71	3RD/105	

Scale	1:500	Date	May 1980
Drawn by	Passed by	Issued for	
RPR	R	MARCH 80 REPORT	

Utilisation of the high output available from the drilling equipment necessitates the sealing of any gap between the underside of the structure and the prepared rock surface at an early stage. This is done by inflating the flexible tubes (Reference 17) located on the underside of the base (Roxburgh & Partners' Drawing 3RD/104) so that narrow strips along the line of the rock anchor ducts in the rear and transverse walls can be cement grouted. With inflation of the tube along the front edge of the structure, the completion of the underbase grouting is carried out concurrently with the rest of the post emplacement operations.

The anchor tendons are prefabricated in controlled conditions at the manufacturer's factory and are delivered coiled onto large diameter drums. When the borehole is ready, the precoiled tendons are loaded onto a special handing machine which facilitates their careful installation in the hole. The fixed anchor length is then grouted up using a high strength reasonably slow setting epoxy or polyester resin delivered through tubes incorporated in the tendons. Following a curing period of up to 24 hours the anchor head is fitted and stressing and proof testing commenced using multi-strand jacks.

On completion of approximately 50% of the anchors in the walls, up to two water column chambers are emptied at one time and the various anchor installation steps carried out in sequence.

Following completion of the installation of the rock anchors, the emplacement barge is disconnected, partly deballasted and removed from the structure. It is then returned to the holding area for further work. Installation of the mechanical and electrical plant then follows.

12.2 Installation of the Mechanical and Electrical Plant

In order to minimise the amount of installation work offshore, the mechanical and electrical plant items are assembled into a number of modularised packages. The main modules are made up as follows:

(i) the air turbine and alternator package, with the turbine guide vane control mechanism and the alternator located inside a watertight offshore module;

(ii) separate inlet and outlet rectifying valve packages;

(iii) the control and monitoring equipment together with the transformer and rectifier in a second multi-compartmented watertight module.

The air ducting bends, inlet and outlet tubes etc., are the main non-modularised plant items.

Each module is manufactured, assembled and tested in an onshore engineering works. They are then dismantled and shipped out directly to the site location.

The modules are placed in position on the Breakwater structure using a crane barge situated on the lee side. Steel frames with locating flanges are provided to facilitate speedy and accurate installation and positioning. Following preliminary location of the items, the air ducting is placed and the joints made before completing the fixing down.

The installation of the modules will not be particularly weather sensitive due to the excellent shelter provided by the structure. With reasonable conditions, it is estimated that up to two main modules could be positioned per day by a single crane barge, with the lighter air ducting items being positioned at a later stage by a portal crane temporarily located on the structure.

Upon completion of the main mechanical installation work, connection of the various electrical elements is carried out, with the main power take off cables being fed through watertight openings in the various containers. On completion of the electrical installation work and connection to the prelaid submarine cables, the bulkheads across the mouths of the water columns are removed in turn and the system commissioned.

The installation work for the M & E equipment is essentially similar to that involved in major maintenance operations, and could provide useful familiarisation at an early stage. It is therefore considered desirable to build and commission a number of the permanent maintenance vessels to assist in the installation work.

The modularisation and resulting installation of the M & E plant at the site location offers a significant advantage over installation at an earlier stage, because it provides a clear and encumbrance-free working space on top of the structure during the tow out and emplacement phases.

13. REMOVAL AND SALVAGE

Scrapping of the installation at the end of its useful life is, in effect, a reversal of the installation techniques. Bulkhead gates are installed in each water column and the mechanical and electrical plant removed by crane barge and transport ship. This is followed by an emplacement barge being placed over the end structure in a group and ballasted down and attached to the structure. The water columns are then pumped dry, rock anchor heads are exposed and the tendons released. Cover plates are placed over the anchorage holes to make the structure watertight. The tendons in the walls are then released and the composite structure lifted clear of its foundation by deballasting the emplacement barge. The loose rock anchor tendons are severed by strings of explosive charges (see Reference 26) and the composite structure is towed away for scrapping.

14. CONSTRUCTION AND INSTALLATION PROGRAMME

14.1 Introduction

The evaluation of the construction and installation programme for the 2GW installed capacity power station has been carried out assuming that 782 no. 4 cell primary units are required. Constructional procedures and output rates used have been based mostly on current technology.

14.2 Construction of the primary structure

The proposed programme of operations at the construction yard is given in Figure 14.1. The anticipated construction time of 12 months for one unit is based on concreting rates of 50 to 60 m^3 per hour and makes suitable allowance for holidays, weather downtime, flooding and recovery of the basin. However, due to the nearly equal split between the work in the drydock and the work at the floating berth the cycle time for yard output is effectively six months.

The size and resulting capacity of existing oil platform production yards is given in Table 14.1. This shows that an annual production rate of up to 120 units can be achieved.

Construction Yard	Basin Size m	No. of Units per cycle
Ardyne No. 1 Elf	100 x 120	3
No. 2 Brent	120 x 120	3
No. 3 Cormorant	120 x 170	6
Hunterston	150 x 150	8
Portavadie	450 x 80	12
Kishorn	160 diameter	8
Nigg	305 x 176	20
Total per 6 month cycle		60

Table 14.1

Summary of Capacity of Existing Oil Platform Yards

The output in the first year of the programme will be less than 120 units because only Kishorn is capable of immediate production. The other yards will require varying amounts of lead time to instal required facilities including concrete plant, dock gates and suitable floating berths.

No.	Month No / ACTIVITY	1	2	3	4	5	6	7	8	9	10	11	12
1.	Prepare/recover basin												
2.	Construct 1st stage base slab												
3.	Precast nose and beam units												
4.	Fix slipform												
5.	Slipform lower walls				H			H					H
6.	Instal centre pier & bulkheads				O			O					O
7.	Flood basin, check structure				L			L					L
8.	Deballast, tow out & moor				I			I					I
9.	Complete base slab				D			D					D
10.	Place lower nose sects				A			A					A
11.	Continue slipforming walls				Y			Y					Y
12.	Place upper nose sects												
13.	Complete slipforming walls												
14.	Place roof beams & insitu deck												
15.	Prestress nose sections												
16.	Complete caisson												
17.	Tow to site												

Figure 14.1 Construction programme for Breakwater Device

Therefore, the overall rate of construction given in Table 14.2 is based on 50% production in year 1.

Year	Annual production	Total production
1	60	60
2-7	120	720
Overall time 7 years		780

cement 614000 t/y

Table 14.2
Output from Construction Yards

14.3 Wave Climate

Due to the extremely exposed location of the site, most of the site preparation and installation operations are extremely weather dependent. At present, as noted in Section 7.2, there is only limited measured data available for the site location. However, a preliminary analysis of this data has been made to obtain a first estimate of how the weather will affect the programme.

The statistics must be reviewed from two separate view points. The first relates to the site preparation operations where information is required on (i) the number of days annually when significant wave heights exceed 2.5m thereby preventing the dredger from working, and (ii) the number of days when wave heights exceed 4m signficant thereby requiring the dredger to move off station. Data on the period August 1978 to June 1979 is given in Table 14.3.

Month	H_s greater than	
	2.5m	4.0m
	(No. of days)	(No. of days)
August 1978	0	0
September	15	1
October	No records	No records
November	No records	No records
December	5	0
January 1979	3	0
February	11	0
March	No records	No records
April	2	0
May	3	0
June	5	0
July	0	0
Total over 9 months	44	1

Table 14.3
Preliminary Weather Statistics for Site Preparation Operations

The second view point is the number of weather windows of suitably calm weather over periods of several consecutive days. Data is presented in Table 14.4 on weather windows of 1 and 2m significant wave height over periods of 3 to 5 days.

Month	H_s not greater than		
	1m	2m	2m
	over 3 days	over 3 day	over 5 days
August 1978	1	4	1
September	0	2	0
October	No records	No records	No records
November	No records	No records	No records
December	1	5	1
January 1979	1	5	1
February	1	4	1
March	No records	No records	No records
April	3	6	3
May	1	6	2
June	3	8	4
July	0	7	2
Total over 9 months	11	47	15

Table 14.4

Preliminary Weather Statistics for Emplacement Operations

It is important to set the above projections into proper context. They are effectively based on very elementary calculations on data measured over an extremely short period of time, and therefore cannot be considered very reliable. It is essential to have a much greater amount of data before commencing design and construction of the Power Station.

14.4 Site Preparation

The rate of preparation of the offshore site for the Breakwater Device, and consequently the number of dredgers required, is based on handling the typical output of the construction yards in years 2 to 7. With an estimated average cut of 2m depth, over an area of 70m x 30m per 4 cell unit, the amount of rock to be removed annually is approximately 504,000m^3. It is expected that up to 80% of this volume will be fractured by the explosives, thereby facilitating rapid removal by the dredger, with the remaining 20% being directly cut at a much slower rate.

Production rates of 30,000m^3 per week in fractured rock and 3,000 m^3 per week in solid rock have been assumed for the dredger. These have been based on the projected output for the Simon Stevin, and on production figures reported by a second dredger of similar type i.e. the 'Al Wassl Bay' (References 18 and 19). The overall annual programme then requires 47 dredger weeks (see Table 14.5).

Material	Annual production required	Dredger production capacity	Time
	(m^3)	(m^3/week)	(weeks)
Fractured rock	403,000	30,000	14
Solid rock	101,000	3,000	33
Total	504,000		47

Table 14.5

Rock Dredger Production Requirements

Individual dredgers are estimated to have an effective working year of 32 weeks at the offshore location. This figure includes a loss of 16 weeks annually for weather downtime (estimated from Table 14.3) and four weeks annually for maintenance, breakdown, inspection, overhaul, unforeseen problems etc. Therefore, two dredgers will be required to match the output capacity of the yards as calculated in section 14.1.

14.5 Installation of Structure at Offshore Site.

The offshore installation operation commences prior to the mating of the straddle barge and the floating Breakwater structure at the holding area, and is broken into three phases. The first phase is the connection of the barge and structure at the holding area, the towout to the offshore site location and the emplacement operation at the site location. This phase is estimated to have an average duration of one week, assuming presence of a suitable weather window.

The second phase is the installation of the rock anchors to provide permanent stability of the structure. This is estimated to require approximately 3½ weeks from the initial sealing of the foundations to drilling, installation and tensioning of the anchors in the base of the fourth water column. Installation of the anchors in the base of the water columns is not commenced until at least half the wall anchors are tensioned. With completion of at least three quarters of the wall anchors work can be carried out in two water columns simultaneously.

The third phase is the removal of the barge from the emplaced structure and the return trip to the holding area. This is estimated to require five days.

The structure installation operation is thus estimated to require approximately five weeks under normal working conditions with no undue delays in the programme. However, it is considered necessary to incorporate a 20% provision for weather downtime and mismatching of weather windows with the emplacement sequence. With inclusion of a four week period for maintenance, breakdown, inspection, overhaul and unforeseen problems etc., it is anticipated that one straddle barge will instal eight structures per year. Therefore, the overall programme requires a minimum of 15 barges to maintain progress during years 2 to 7.

14.6 Installation and Commissioning of Mechanical and Electrical Plant.

It is anticipated the installation of the mechanical and electrical plant modules, the installation of the generation, control and monitoring systems and the commissioning of a 4 cell unit will take up to eight weeks.

14.7 The Overall Programme

The overall programme for the 2GW power station is dictated by the output capacity of the existing Scottish oil platform production yards. The five yards can produce the required 782 structures in a period of 6½ years. However, it is necessary to allow a lead time of up to 12 to 18 months for site surveys, design of the structures, preparation of the construction yards and building of the specialist construction and installation plant. In practice, there will also be a final lag time following installation of the last structure of a further six months to enable final construction and commissioning works to be carried out.

Therefore, the overall programme for a 2GW power station is envisaged to be 8 to 9 years, although there could be significant quantities of power produced at a fairly early stage in the programme.

15. OPERATION

A preliminary evaluation of the method of control of the Breakwater Devices shows that unmanned operation is preferable to manned due to the cost of providing suitable accommodation on the structure, the communication and environmental conditions in winter and the labour requirements for an 'offshore' installation. Therefore, 'on-load' control of the generating equipment is carried out by an onboard operating system which monitors and responds to system demand and instantaneous input power levels.

An outline of the operating and control system is given in Figure 15.1. The central control system regulates the alternator operating voltage and sets the turbine inlet guide vanes by monitoring water column pressure and displacement, pressure head across the turbine, and speed, voltage and current conditions in the alternator. The opening time of the valves is also controlled. The system objective is to obtain optimum turbine efficiency and water column damping at all alternator speeds.

Protection of the generating system is provided by a hydraulic system (Figure 15.2) which over-rides and closes the turbine inlet vanes and opens the water column by-pass valves in response to exceedance of certain preset operating limits and indication of fault conditions.

The basis of the hydraulic control system is a set of two way directional control valves which are operated by a solenoid. When the solenoid is energised hydraulic pressure is directed through the control valves so that the by-pass valves are held closed and the guide vane over-ride mechanism is not operational. This allows normal operation of the generating system. When the solenoid is de-energised the hydraulic pressure is directed so that the by-pass valves open and the guide vane over-ride mechanism operates to close down the turbine.

The protection system is 'fail safe' in operation, in that the solenoid must always be energised during normal operation of the generating system. Additional protection against potential major fault conditions is provided by inclusion of a circuit breaker in the solenoid control circuit. When the circuit breaker is tripped by the fault indication, the solenoid is de-energised and cannot be re-energised until the circuit breaker is re-set. The settings of the components in the protection system under various operational limit and fault conditions are given in Table 15.1.

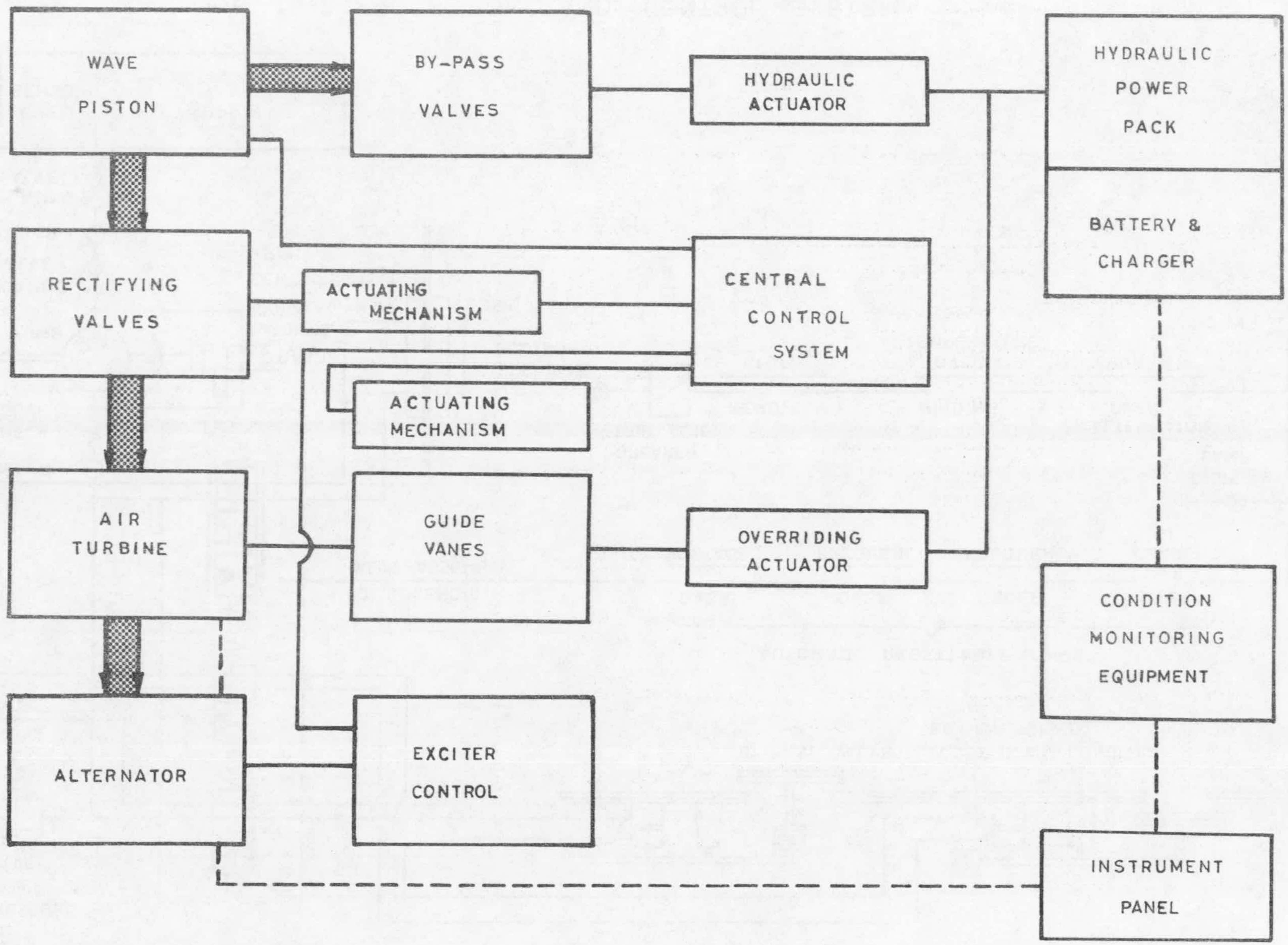


FIG 15:1 OUTLINE OF OPERATION AND CONTROL SYSTEM

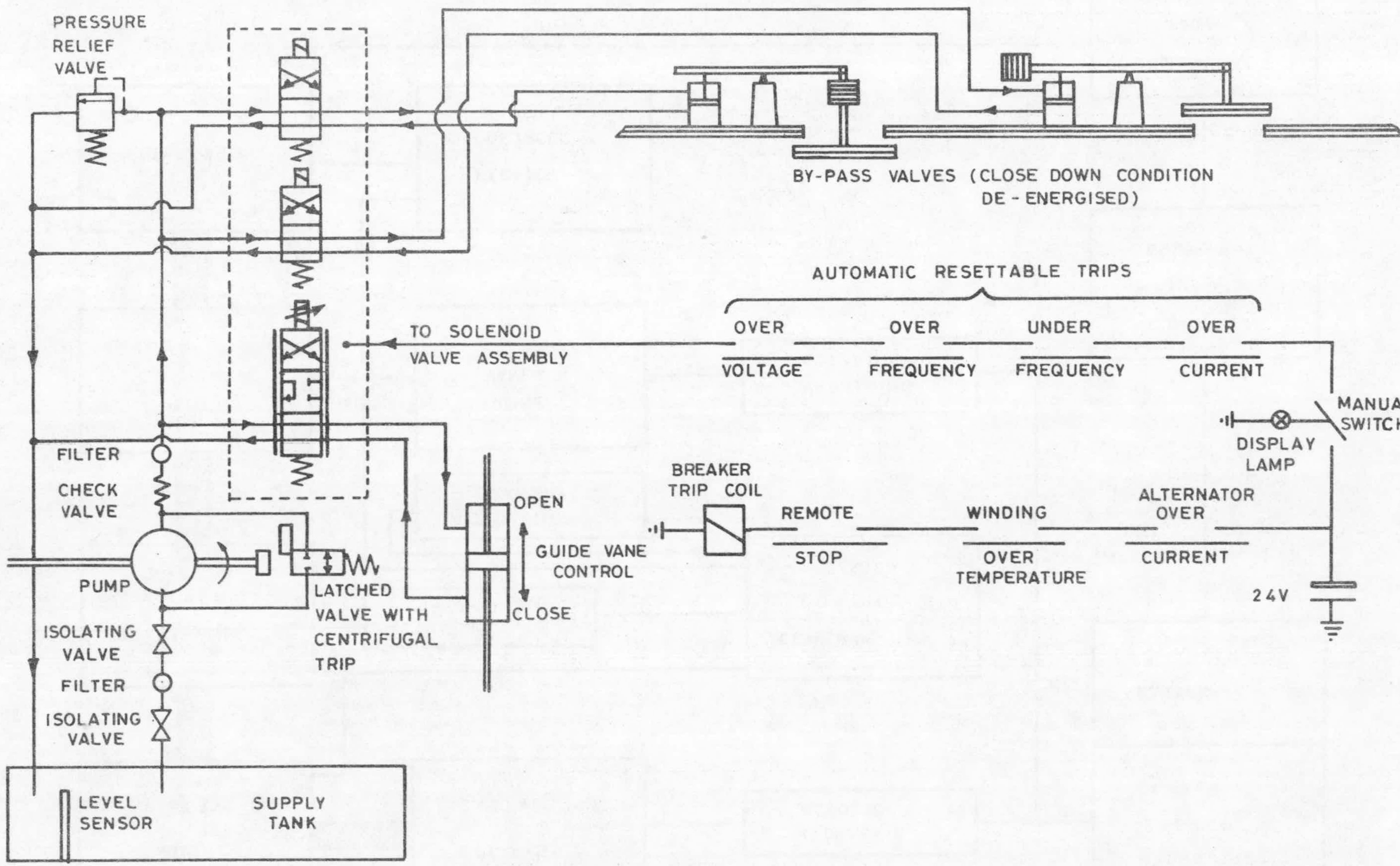


FIG 15:2 PROTECTION AND CONTROL SYSTEM
(SCHEMATIC LAYOUT)

Condition	Solenoid	Circuit Breaker	Water Column Bypass Valves	Turbine Inlet Guide Vane Over-ride Mechanism
Normal Operation	energised	set	closed	off
Normal operation limits exceeded (i) Under frequency (ii) Over frequency (iii) Over voltage (iv) Over current Local stop (for maintenance etc)	de-energised	set	open	on
Fault conditions (i) High alternator current (ii) High alternator winding temperature Remote stop	de-energised	tripped	open	on

Table 15.1

Protection System Settings

16. MAINTENANCE CONSIDERATIONS

16.1 Mechanical and Electrical Plant

The maintenance philosophy for the M & E plant is one of preventive maintenance, which implies regular servicing and replacement of components as necessary before they fail. In order to achieve this objective a service life for the main components within the system is determined so that replacement can be carried out in a planned sequence. It is anticipated that the in service period will be about three years.

Therefore, in the design of the plant layout it is important to provide easy access to items that are likely to require regular replacement offshore. This is particularly necessary to ensure minimum loss of energy production.

Plant items that are removed from the structure are returned to a shore based workshop for a complete overhaul. On completion of the overhaul they are then available for re-use.

A certain amount of very minor maintenance and repair work will be carried out insitu, with support from a suitable service tender. Due to lack of space on the structure for permanent accommodation, each maintenance crew is withdrawn at the end of a shift. However, emergency shelter will be provided in a specially constructed survival module fixed to a strong point on the structure at commencement of the maintenance work.

16.2 Structural Maintenance

The use of concrete as the main structural material results in very little maintenance being necessary. In general the greatest work is the removal of debris and marine fouling from the water column chamber. This is carried out by sealing the chamber using steel stop logs located in special grooves at either side of its inlet opening. The column is then drained and marine growth etc removed by high pressure water jets.

The maintenance of the power station requires a special shore base with suitable workshop and storage facilities located in a sheltered position in the adjoining Hebridean islands. It is anticipated that a fleet of two crane ships, four supply/module transport ships and six to eight service tenders will be required. A number of helicopters are also necessary to provide a rapid communication and emergency transport service between the vessels, the wave energy devices and the shore.

17. PREDICTION OF POWER OUTPUT

17.1 Introduction

The analysis of the performance of the Breakwater Device falls into three main parts. The first is the efficiency of primary conversion of sea power into air power through the wave piston mechanism. This is computed using the South Uist inshore scatter diagram assuming Pierson-Moskowitz spectra, in conjunction with the monochromatic frequency response of the water column. The second part is the efficiency of the secondary conversion system from air power to shaft power. Finally, the maximised generator and transmission efficiencies are calculated to enable establishment of component ratings and computation of the average output.

17.2 Efficiency of Primary Conversion

(i) The Water Column

The monochromatic efficiency curve for the Reference Design was obtained using the theoretical analysis described in NEL Progress Report No. 8 (Reference 20), with experimental verification of the method having been carried out prior to RPT's November 1979 assessment (Reference 21). A model of the Breakwater Device is shown on page 86 undergoing tests in the N.E.L. two-dimensional wave tank.

The theoretical method gives an idealised efficiency within the limitations of the analysis, whilst small scale modelling generally produces efficiencies which are approximately 5% lower at the natural frequency. This latter effect is probably due to the loss mechanism, which being Reynolds number dependent, diminishes as the scale approaches full size. At frequencies away from the natural frequency both experiment and theory are subject to increasing error. However, on balance it is considered that the theoretical prediction is more realistic than small scale model testing and, in addition, is more convenient to use because it can cover a much wider range of frequencies.

The present Reference Design shown in Figure 17.1 has a predicted monochromatic efficiency curve as shown in Figure 17.2. This curve is sub-optimal in two respects. Firstly, the selected applied damping is twice the theoretical optimum, which gives rise to improved efficiency at extreme frequencies at the expense of diminished efficiency at the natural frequency. The resulting overall effect is then a marginal reduction in sea efficiency which is balanced by a reduced air flow rate. This allows turbine diameter and duct areas to be reduced, and also reduces air friction losses. Secondly, the characteristic dimension is greater than need be for optimium hydrodynamic performance, but is beneficial in improving structural stability, material content, and does permit the siting of the power pack on the upper deck of the column rather than at the rear as envisaged in the November 1979 Reference Design (Reference 11).

MODEL BREAKWATER DEVICE UNDERGOING TESTS IN THE NEL TWO DIMENSIONAL WAVE TANK

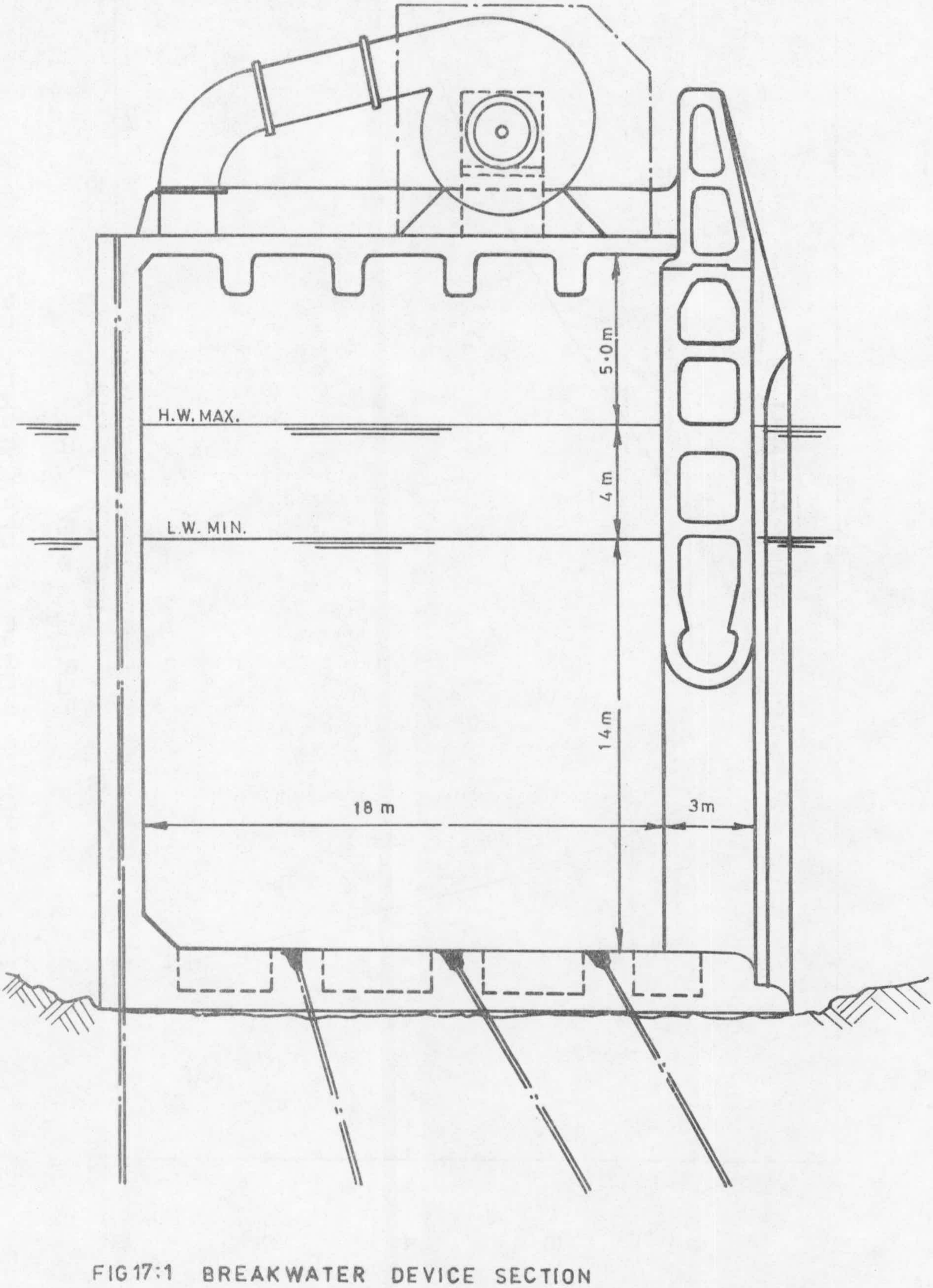


FIG 17:1 BREAKWATER DEVICE SECTION
REFERENCE DESIGN 1980

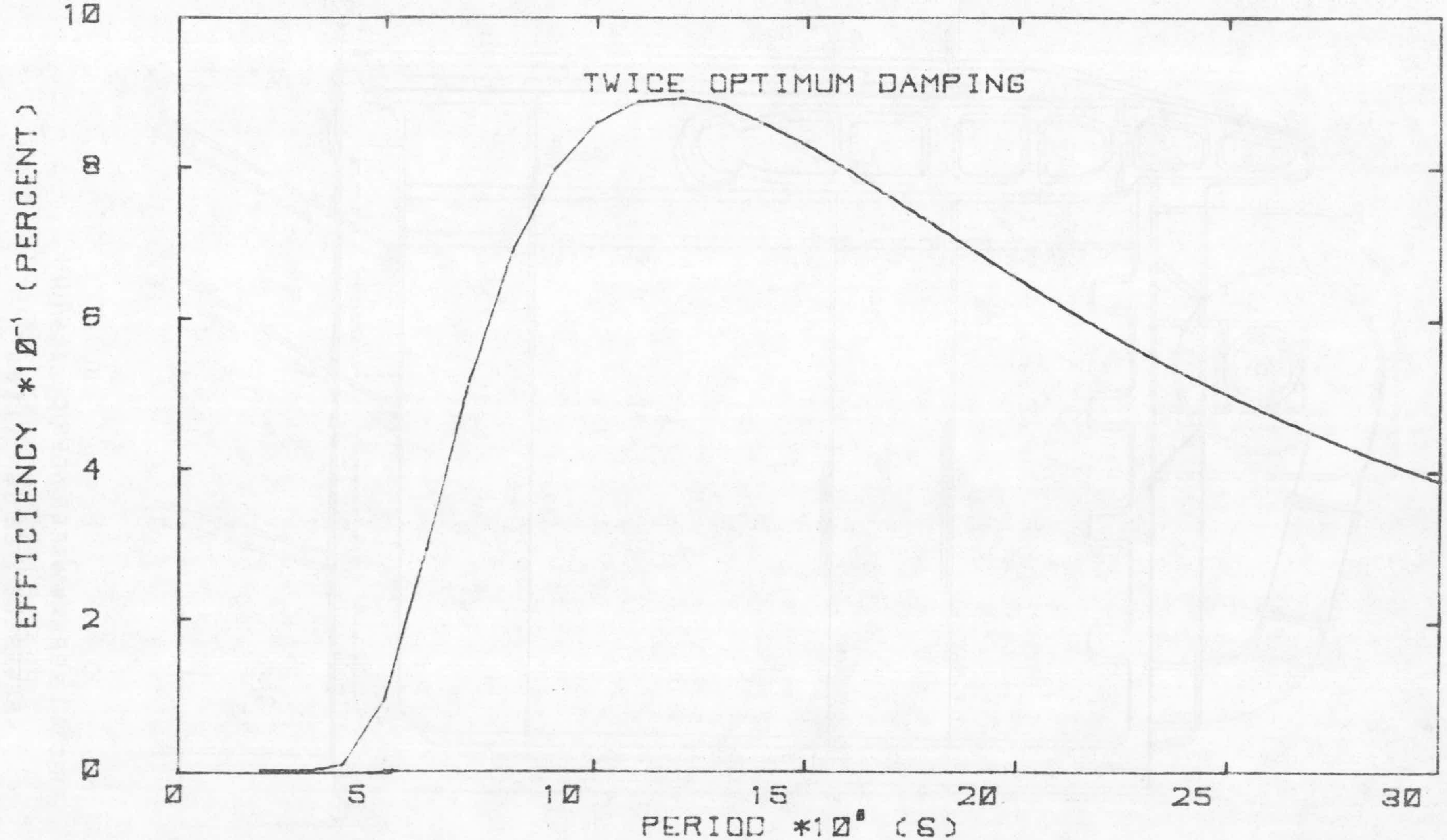


FIG. 17.2 MONOCHROMATIC EFFICIENCY OF COLUMN

The efficiency curve (shown in Figure 17.2) is for a device located in 18m of water. Tidal variations of plus or minus 2m give rise to increased efficiency at low tide and decreased efficiency at high tide, with the behaviour at mean water level being therefore representative of the overall efficiency.

(ii) The Power in the Sea

The data presently available consists of the scatter diagram (Figure 7.6) and the average spectrum (Figure 17.3) for the inshore site at South Uist for the period August 1978 to June 1979. This data was obtained from a wave rider buoy located in water of 15m depth. The proposed siting of the present Reference Design is in 18m of water. The fact that spectra, average power, device efficiency are all functions of water depth in shallow water makes it difficult to transfer data from one depth to another.

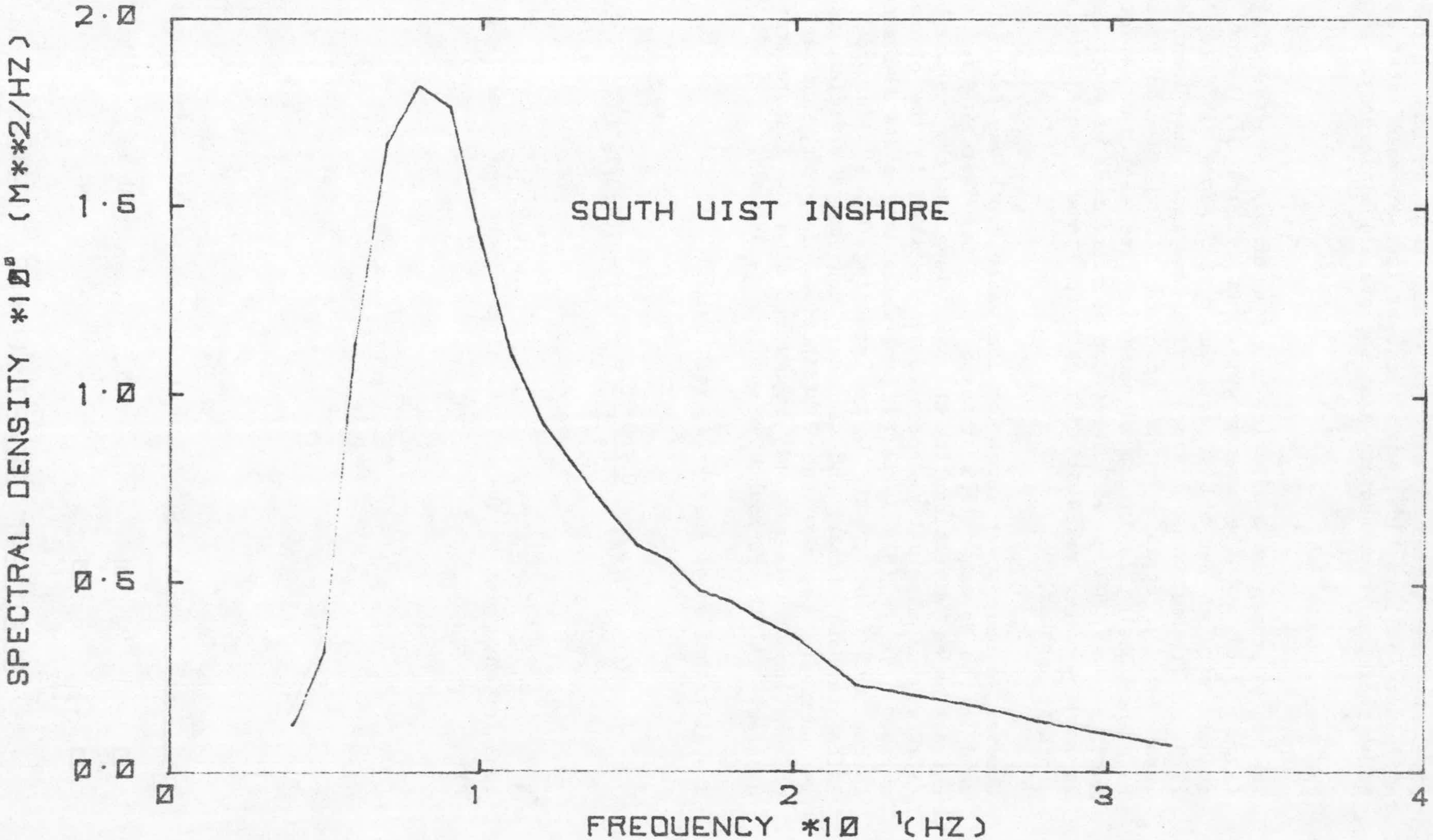
However, a mean power level for 18m water depth was derived and used in the November 1979 assessment. After factoring for site correction and directionality the power level at the inshore buoy (17kW/m) was marginally decreased to 16.9kW/m in 18m of water depth. It is this latter figure which represents the annual theoretical input to the bottom standing OWC. In order to calculate both primary and secondary conversion efficiencies it is necessary to know the distribution of power, and at the present moment this may only be derived from the inshore scatter diagram. A Pierson-Moskowitz spectrum is assumed.

The distribution of energy is given by:

$$\varepsilon(f) = \frac{0.168\ 75}{T_e{}^4 f^5} H_s{}^2 \exp(-0.6750/T_e{}^4 f^4) \qquad (1)$$

Thus for any point on the scatter diagram, the sea power is given by:

$$P(H_s T_e) = \frac{\rho g^2}{4\pi} \int_0^{\pi} \varepsilon(f) f^{-1} \phi(mh) df \qquad (2)$$



FIG·17.3 AVERAGE SPECTRUM FOR AUG 1978 – JUN 1979 where the shallow water correction factor is:

$$\phi(mh) = \tanh(mh)\left\{1 + \frac{2mh}{\sinh(2mh)}\right\} \qquad (3)$$

$$h = \text{depth of water (15 m)}$$

$$\lambda = \frac{g}{2\pi f^2}\tanh(mh)$$

$$m = 2\pi/\lambda$$

The average power is obtained by the equation:

$$\bar{P} = \sum_{T_e}\sum_{H_s} P(H_s, T_e)q(H_s, T_e) \qquad (4)$$

where q is the fractional occurrence of (H_s, T_e) pairs.

For the scatter diagram shown in Figure 7.6 the average power is 12.0 kW/m.

(iii) The Efficiency of the Water Column In-situ

Since the Reference Design is located in 18m of water and the detailed wave data is for water of 15m depth, it is more convenient to apply device behaviour to the reduced water depth and then apply a gross correction factor to make the data applicable to 18m, than it is to attempt to convert the wave data to the increased depth. Small changes in water depth do not have a significant effect upon the monochromatic efficiency of the device. Thus the curve in Figure 17.2 may be applied directly to the inshore data, i.e.:

$$\eta_s(H_s T_e) = \frac{\int_0^\infty \varepsilon(f)f^{-1}\phi(mh)\eta(f)df}{\int_0^\infty \varepsilon(f)f^{-1}\phi(mh)df} \qquad (5)$$

Information on the variation of device efficiency with wave height is not available. In small and modest waves the efficiency will remain fairly constant. In big waves, the onset of eddy formation at the column inlet will cause a significant reduction in device efficiency, which, however, should only occur infrequently.

Thus, at this stage, the sea efficiency is assumed to be a function of energy period only and is shown in Figure 17.4. Using this curve and the inshore scatter diagram it is possible to compute the annual average efficiency, i.e.:

$$\bar{\eta} = \frac{\sum_{T_e} \eta_s(T_e) \sum_{H_s} P(H_s T_e) q(H_s T_e)}{\bar{P}} \tag{6}$$

The resultant value is 0.71.

A similar calculation using the measured average spectrum for the period August 1978 to June 1979 produced the same value. Therefore, a digital spectrum correction factor of 1.0 has been taken for the Breakwater Device, as opposed to the value of 0.95 used for the devices further offshore.

The occurrence distributions of air power and air flow rate are of particular relevance in the analysis of secondary system efficiency. They have been computed using the following equations applied to each (H_s, T_e) pair:

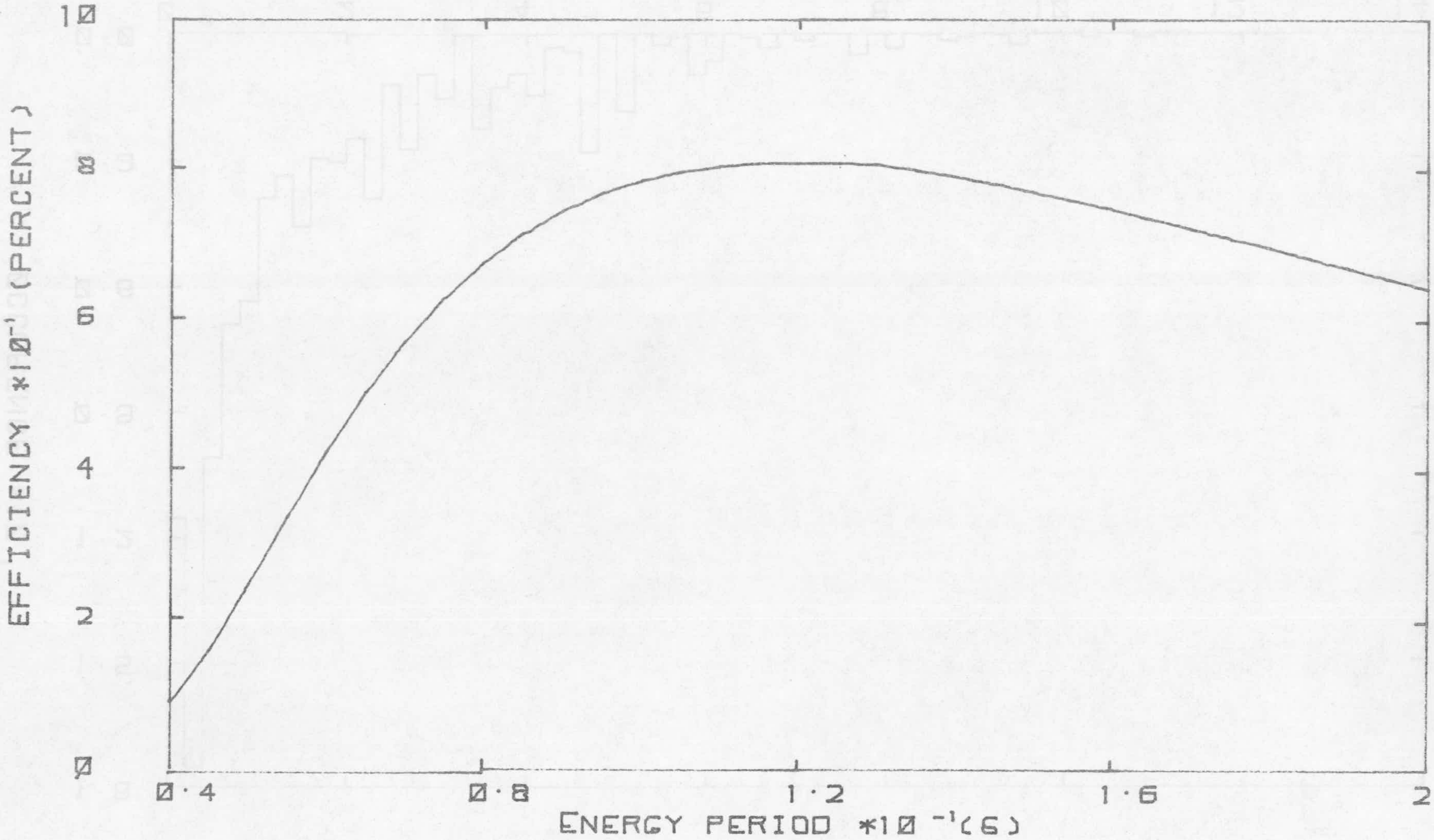
$$P_a(H_s T_e) = \eta_s(H_s T_e) P(H_s T_e) \tag{7}$$

and

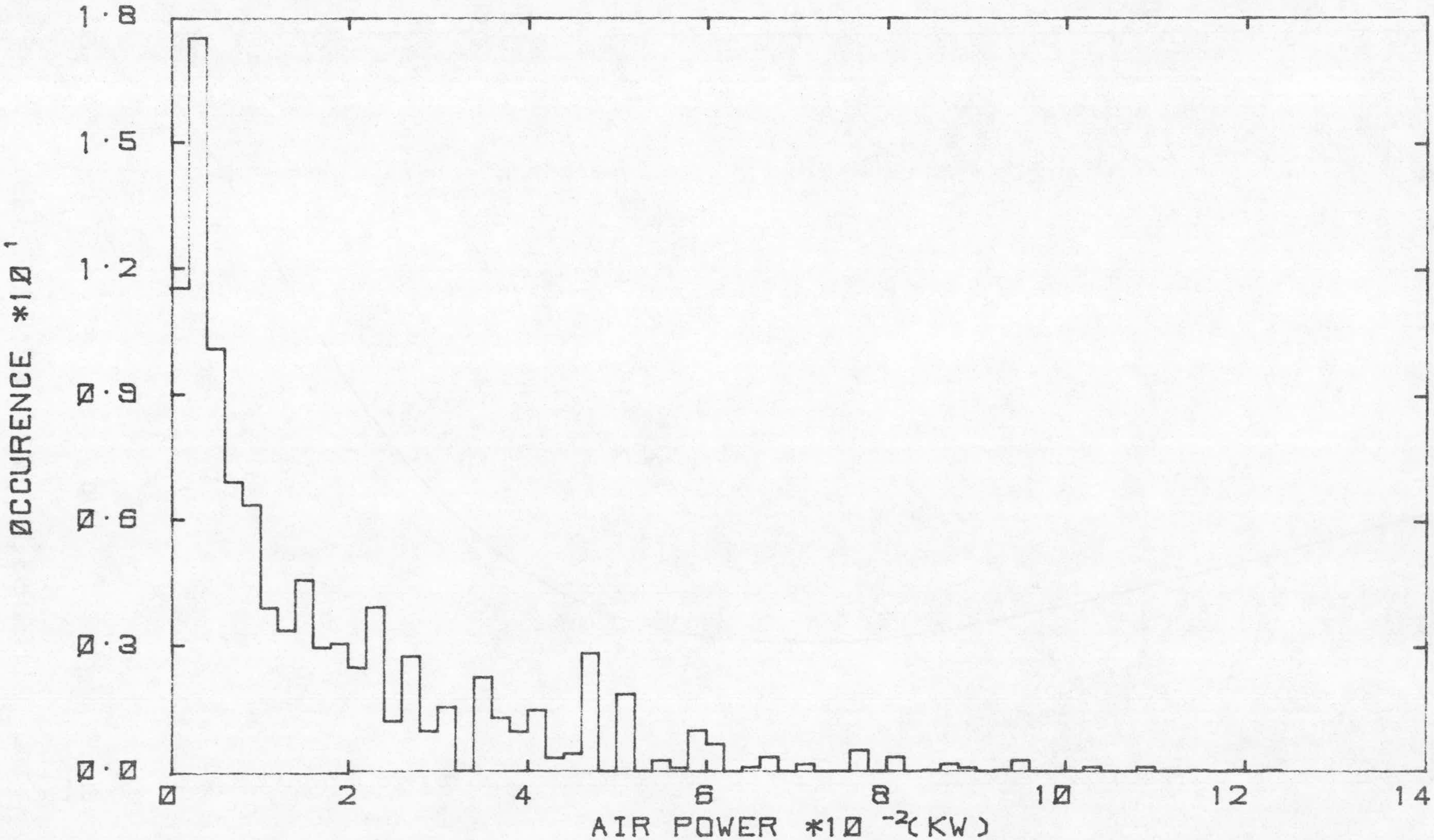
$$Q_{rms}(H_s T_e) = \left\{\frac{P_a(H_s T_e)}{\rho g K}\right\}^{\frac{1}{2}} \tag{8}$$

where K (= H_{rms}/Q_{rms}) is the applied damping coefficient at the column i.e. linear damping is assumed.

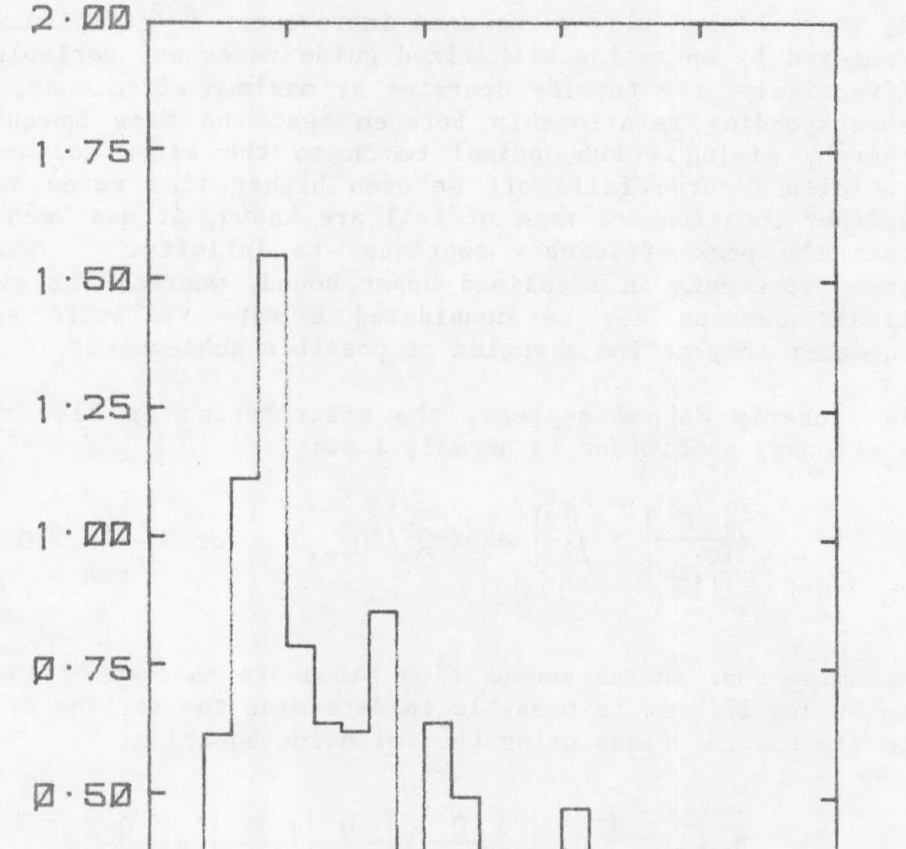
The power distribution has been sub-divided into intervals of 5kW, and the flow rate into intervals of $5m^3/s$, the appropriate occurrences being extracted from the scatter diagram, summed, and then plotted as shown in Figures 17.5 and 17.6.



FIG· 17.4 COLUMN EFFICIENCY FOR P-M SPECTRUM



FIG· 17.5 ANNUAL DISTRIBUTION OF COLUMN AIR POWER



FIG· 17.6 COLUMN FLOWRATE DISTRIBUTION

17.3 The Efficiency of Secondary Conversion

(i) Turbine Efficiency

The hill chart of unit flow against unit speed obtained by steady flow experiments on a small scale model Francis Turbine (Figure 17.7) enables a relationship between efficiency and flow rate to be extracted for almost any simple relationship between head and flow rate. Using variable inlet guide vane settings and variable turbine speed, the optimum efficiency curve for a linear head - flow relationship (i.e. the linear damping curve in Figure 17.8) is obtained by operating at constant unit speed. The curve is ill-defined for the very highest flow rates, but it is likely that the fall-off is approximately as shown.

At these higher flow rates some improvement in efficiency can be achieved by operating with fixed guide vanes and variable speed. Effectively, the turbine operates at maximum efficiency, but the corresponding relationship between head and flow is quadratic, thereby giving a sub-optimal match to the water column. This efficiency curve falls off at even higher flow rates but since neither location nor rate of fall are known, it has been assumed that the peak efficiency continues to infinity. This curve then represents an idealised upper bound, whereas the curve for linear damping may be considered a more realistic estimate. Together they define a region of possible achievement.

In linearly damped systems, the distribution of flow rate for stationary conditions is normal, i.e.:

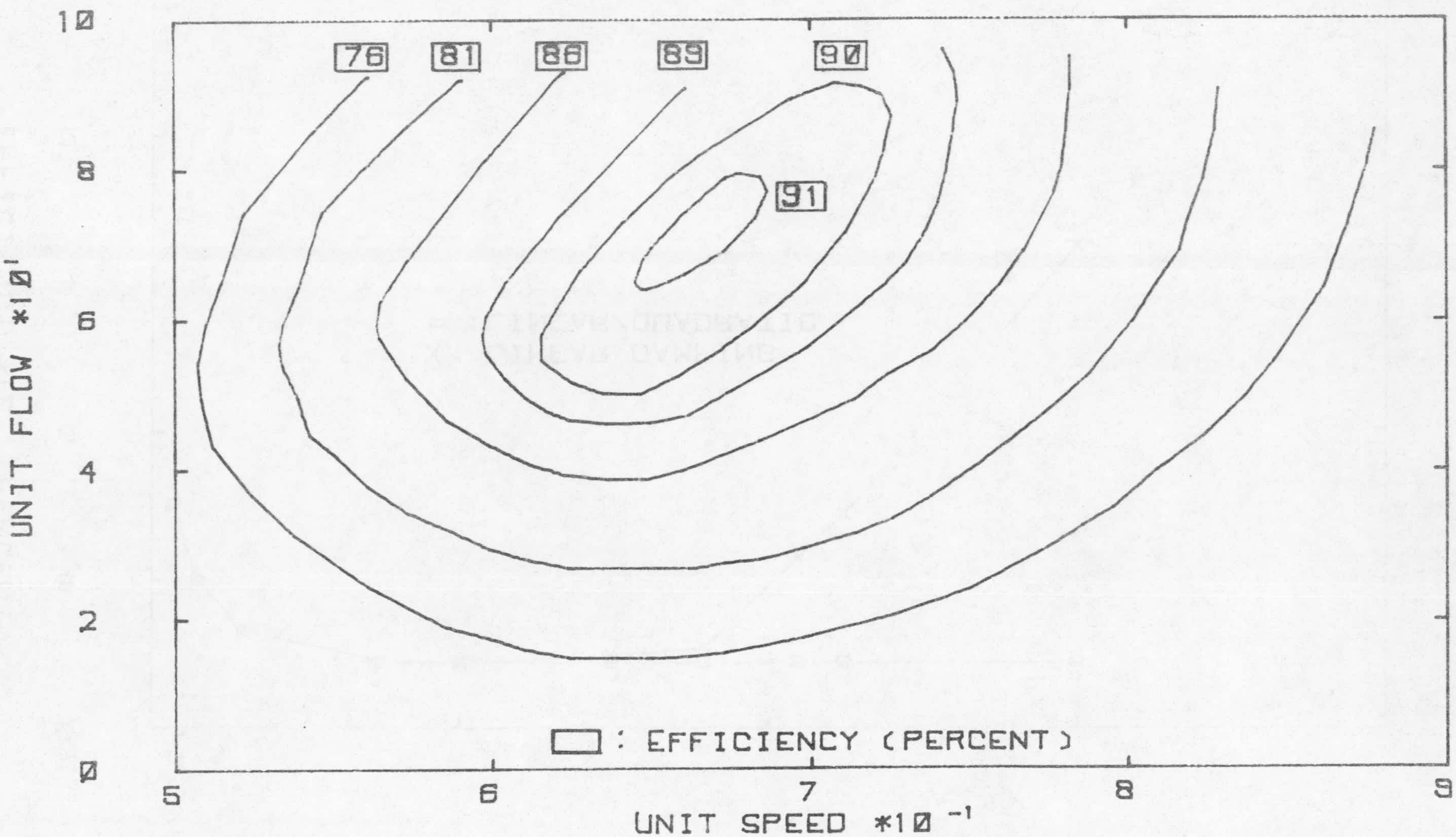
$$Z\left(\frac{Q}{Q_{rms}}\right) = \sqrt{\left(\frac{2}{\pi}\right)}\ \exp(-Q^2/2Q_{rms}^2) \quad \text{for} \quad \frac{Q}{Q_{rms}} \geqslant 0 \tag{9}$$

Assuming that instantaneous flow rates are matched by the curves in Figure 17.8 it is possible to determine the turbine efficiency in fluctuating flows using the following equation:

$$\bar{\eta}_{TURB} = \int_0^\infty \eta_{TURB}\left(\frac{Q}{Q_{rms}}\right) Z\left(\frac{Q}{Q_{rms}}\right)\left(\frac{Q}{Q_{rms}}\right)^2 d\left(\frac{Q}{Q_{rms}}\right) \tag{10}$$

where

$$\eta_{TURB}\left(\frac{Q}{Q_{rms}}\right) = \eta_{TURB}\left(\frac{Q}{Q_o} \times \frac{Q_o}{Q_{rms}}\right) \tag{11}$$



FIG·17.7 HILL CHART FOR FRANCIS TURBINE

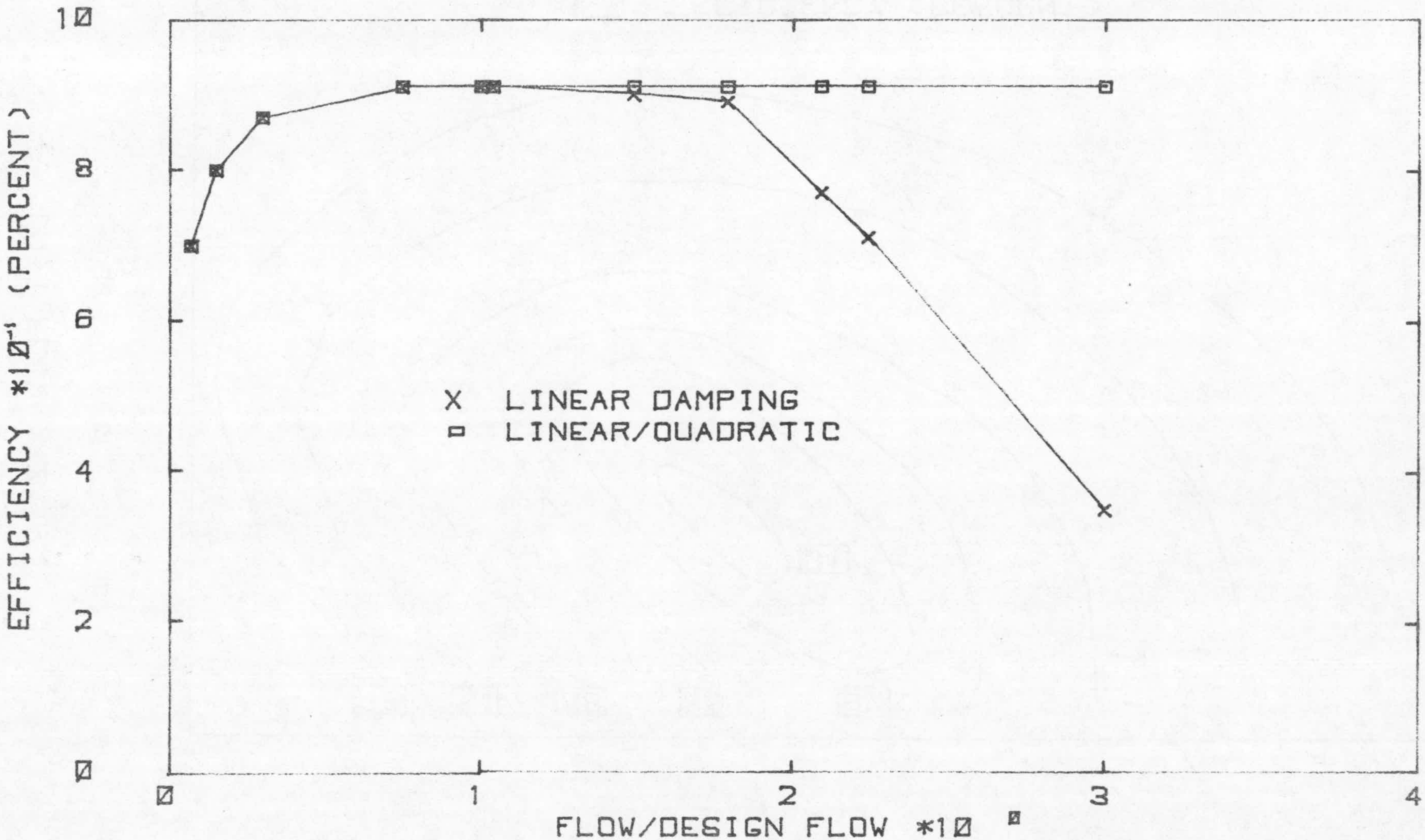


FIG. 17.8 TURBINE EFFICIENCY FOR STEADY FLOW

Integrating the above expression for various values of Q_o/Q_{rms} gives the relationships shown in Figure 17.9 for both pure linear and partial quadratic damping.

(ii) Losses

To complete the calculation of secondary system efficiencies, estimates of losses due to friction, flow through rectifying valves, and leakage must be made.

The instantaneous loss due to aerodynamic friction and diffusion is given by:

$$F_D = \frac{C\rho AV^3(t)}{2} \tag{12}$$

where
C is the loss co-efficient
A is the characteristic area
V is the instantaneous velocity

and for a normally distributed velocity, the average loss is:

$$\bar{F}_D = \int_o^{\infty} \frac{C\rho AV^3}{2} Z(V)dV \tag{13}$$

Therefore, it may be deduced after integration and normalisation that:

$$\bar{\eta}_D(Q_{rms}) = 1 - \frac{1.596}{2gKQ_{rms}} \sum_o^n C_n V^2_{rms,n} \tag{14}$$

where
n = no. of components (i.e. valves, bends etc.)
$K = H_{rms}/Q_{rms} = 6.98$ m/m^3/s

Therefore:

$$\bar{\eta}_D(Q_{rms}) = 1 - 0.011\ 65Q_{rms} \sum_o^n \frac{C_n}{A_n{}^2} \tag{15}$$

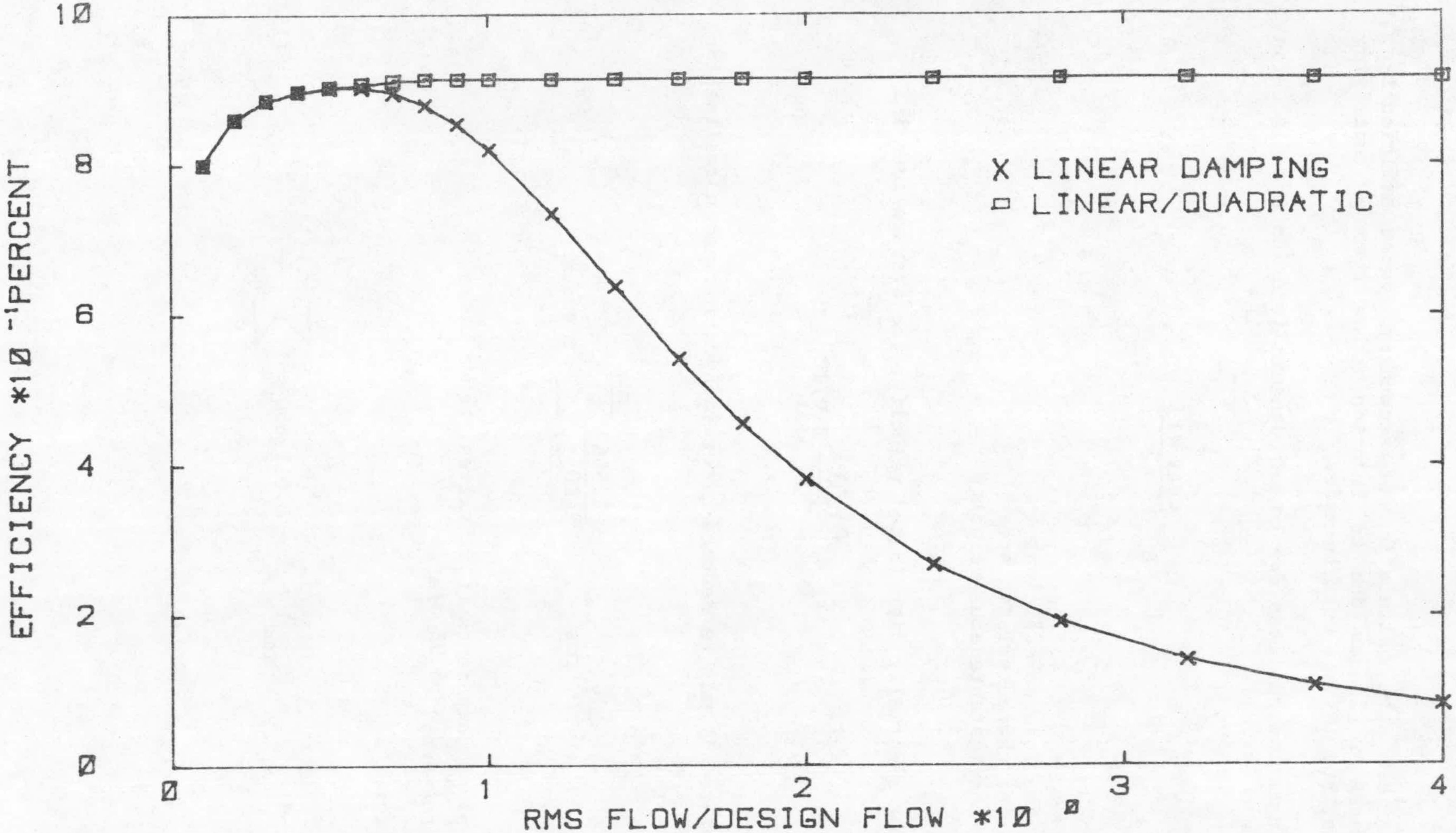


FIG. 17.9 TURBINE EFFICIENCY FOR NORMAL FLOW

The much simplified power take-off system of the present design has two main sources of significant loss, namely, the rectifying valves, and the diffusion of flow from duct to chamber. Characteristic areas are approximately $9\mathrm{m}^2$ with loss coefficients of 0.5 and 1.0 respectively. Therefore, equation (15) can be reduced to:

$$\bar{\eta}_D(Q_{rms}) = 1 - 0.0002Q_{rms} \tag{16}$$

Obviously such losses become really significant only for very high flow rates. In fact, further reductions in characteristic area (i.e. smaller ducts, valves) could be made without undue penalty to system efficiency.

The instantaneous loss due to leakage flow is given by:

$$F_L = \left(\frac{Q_L}{Q}\right)\rho gKQ^2 \tag{17}$$

and for a fluctuating flow the average power loss is given by:

$$\int_o^\infty Z(Q)\rho gKQ^2\left(\frac{Q_L}{Q}\right)dQ \tag{18}$$

If the leakage is given by the discharge coefficient, C_d, and the distribution of flow is normal, then:

$$\frac{Q_L}{Q} = \frac{dLC_D\ \surd(2gK)}{Q^{\frac{1}{2}}_{rms}} \tag{19}$$

and, after integration and normalisation, the leakage efficiency is given by:

$$\bar{\eta}_L = 1 - 0.86\ dLC_D\ \surd(2gK)Q^{-\frac{1}{2}}_{rms} \tag{20}$$

where
d = leakage gap (assumed to be 0.0015m)
L = seal line (36m)
C = 0.6
K = $6.98\mathrm{m}/\mathrm{m}^3/\mathrm{s}$

This gives:

$$\bar{\eta}_L = 1 - 0.326 Q_{rms}^{-\frac{1}{2}} \tag{21}$$

The leakage loss is more significant than that due to diffusion (Figure 17.10) and, though acceptable at present, should be considered carefully in future designs.

Finally a nominal mechanical efficiency of 0.99 has been taken for the bearing losses in the system.

(iii) Optimisation of the Secondary Air System Efficiencies

The efficiency of the secondary system for stationary sea conditions is given by the product of component efficiencies. Thus:

$$\eta_{SS}(Q_{rms}) = \bar{\eta}_{TURB} \times \bar{\eta}_D \times \bar{\eta}_L \times \bar{\eta}_{MECH} \tag{22}$$

The annual average efficiency may be computed from:

$$\bar{\eta}_{SS}(Q_D) = \frac{\int_0^\infty \eta_{SS}(Q_{rms}) M(Q_{rms}) \rho g K Q_{rms}^2 \, dQ_{rms}}{\int_0^\infty M(Q_{rms}) \rho g K Q_{rms}^2 \, dQ_{rms}} \tag{23}$$

where M (Q_{rms}) is the occurrence of rms flow rates (Figure 17.6). The relationship between this average efficiency and the turbine design flow is shown in Figure 17.11.

17.4 Establishment of the Ratings of the Power Chain Components

(i) The Turbine

With an annual rms flowrate of 45.1 m^3/s, the preferred turbine design flowrate is 67 m^3/s. This gives a machine of reasonable size and speed, with an acceptable efficiency of 0.80 (i.e. the average of linear and linear/quadratic as shown in Figure 17.11).

The machine parameters have been obtained from the specific speed and diameter formulae, i.e:

$$N_S = \frac{N Q_o^{\frac{1}{2}}}{(g H_o)^{\frac{3}{4}}} = 1.069 \tag{24}$$

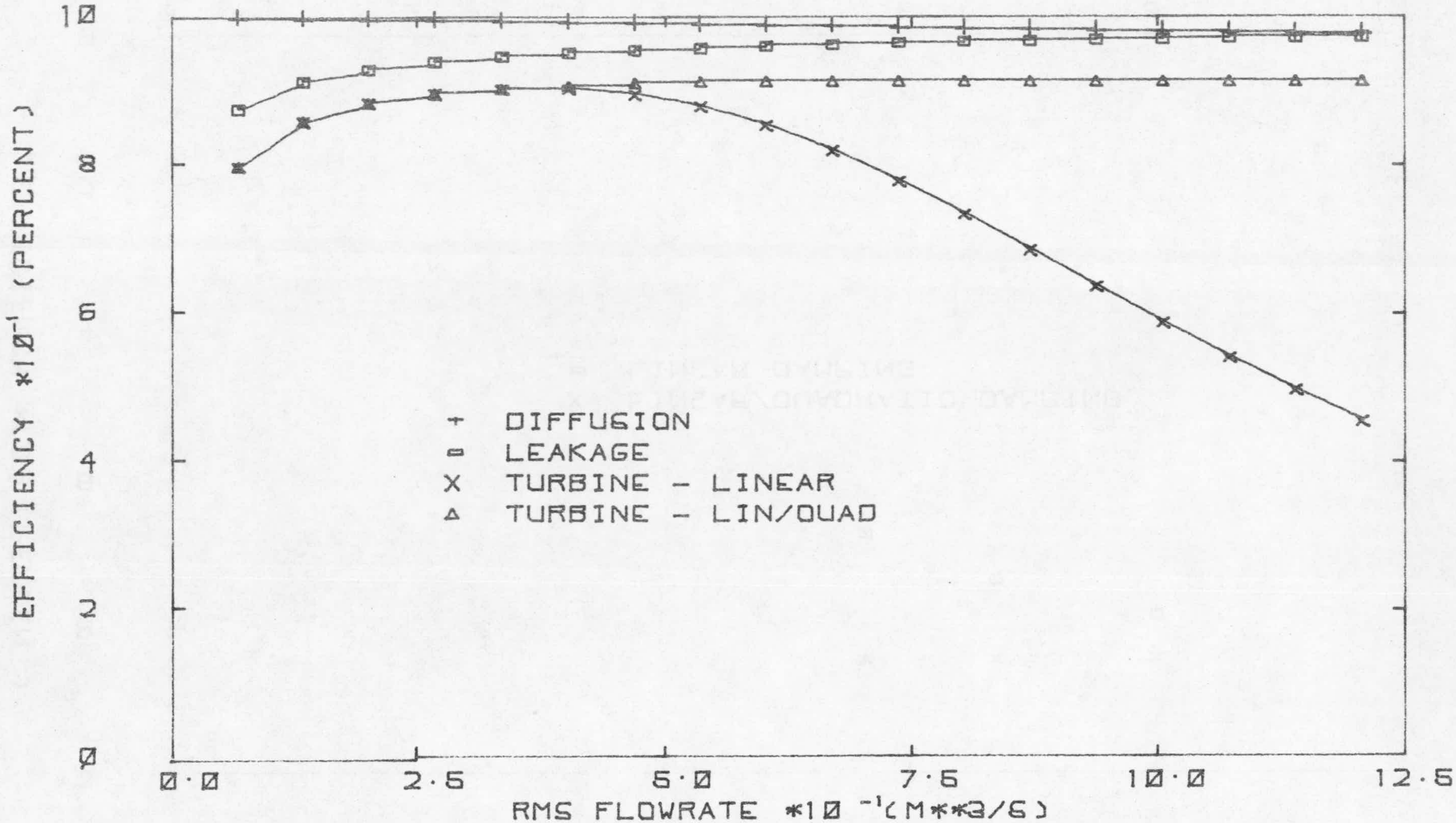


FIG 17.10 COMPONENT EFFICIENCIES OF SECONDARY SYSTEM

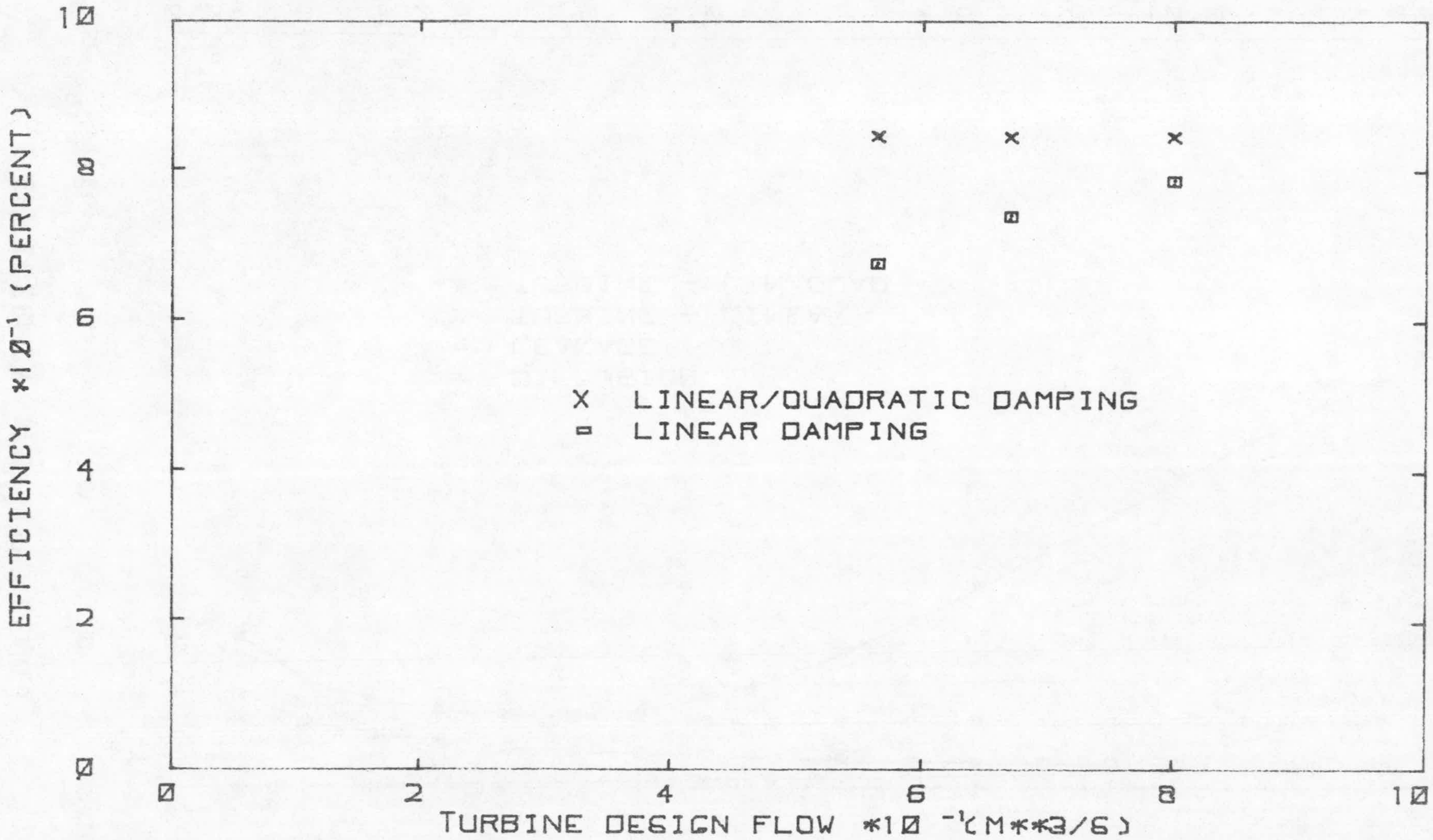


FIG 17.11 ANNUAL SECONDARY SYSTEM EFFICIENCY

$$D_S = \frac{D(gH_o)^{\frac{1}{4}}}{Q_o^{\frac{1}{2}}} = 2.081 \tag{25}$$

where the damping characteristic is $H_o = K Q_o$ with $K = 6.98 m/m^3/s$.

The design output power of the turbine is 336 kW, and although the machine is able to run continuously at 1200kW, a notional rating of 778kW input has been adopted to match the generator.

(ii) The Generator and Power Transmission Components

For each of the remaining power chain components i.e. the generator and the transmission to Perth, ratings have been chosen, following the method used by RPT in their November 1979 assessment (Reference 21), to suit the maximum annual efficiency given by:

$$\bar{\eta} = \frac{\int_o^\infty P\phi(P)\eta(P)dP}{\int_o^\infty P\phi(P)dP} \tag{26}$$

where ϕ is the annual distribution of power input,

$P = P/Pr$ is the power non-dimensionalised by rated power, and

η is the steady state efficiency.

(iii) The Overall Rated Output

The resulting component ratings, average and rated efficiencies and rated output of the power chain are summarised in Table 17.1.

Component	Rated input	Average efficiency	Rated efficiency	Rated output
	kW/m			kW/m
Turbine	55.6	0.80		
Generator	50.0	0.89		
Transmission	48.6	0.86	0.83	40.3

Table 17.1

Rated Output of Power Chain

17.5 Calculation of Power Delivered to the Grid

The calculation of the amount of power delivered to the grid is carried out in several stages. The first is to calculate the power incident on the device - Table 17.2.

	Mean power at South Uist in-shore buoy	Site Correction	Direction-ality Correction	Incident Power
	kW/m			kW/m
High Estimate	18.0	1.26	0.88	18.2
Most Probable	17.0	1.18	0.83	16.9
Low Estimate	16.0	1.15	0.78	15.6

Table 17.2

Average Annual Incident Power

The second stage is to calculate the power captured by the device and then, by taking account of the efficiency and reliability of the various components in the power chain, to calculate the average annual power delivered to Perth - Table 17.3.

	Incident Power	Device Capture Efficiency		Power Chain		Power to Perth
		Based on PM Spectrum	Digital Spectrum Correction	Efficiency	Reliability	
	kW/m					kW/m
High Estimate	18.2	0.75	1.05	0.66	0.95	9.0
Most Probable	16.9	0.71	1.00	0.61	0.92	6.8
Low Estimate	15.6	0.64	0.95	0.56	0.83	4.4

Table 17.3

Average Annual Power Delivered to Perth

Having established the output per unit length of device, the third stage is to calculate the length of device necessary to provide the specified requirements of the power station. For the 2GW installed capacity station discussed in this report, the rated capacity per unit length (from Table 17.1) is 40.3 kW/m and with an average length of 64m per 4 cell unit, the number of units required for the station is 782. This results in an overall length of device of approximately 50km which, when allowance is made for navigation and other gaps, requires a length of coastline of 60 to 80 km. The annual mean power delivered to Perth is then given in Table 17.4.

Predicted Annual Mean Power delivered to Perth	per m length	per 4 cell unit	per 2GW rated power station
	kW/m	kW	kW
Upper bound 95% confidence	9.0	572	447×10^3
Mean	6.8	432	338×10^3
Lower bound 95% confidence	4.4	279	218×10^3

Table 17.4

Mean Annual Power Delivered to Perth

The number of 4 cell units necessary to produce a mean output of 0.5GW is approximately 1160.

18. ANALYSIS OF COSTS

18.1 Introduction

In general, a number of the costs used to build up the overall figure in this report have been taken from RPT's Second Report (Reference 22) which had a base date of August 1978. Accordingly the costs must be updated to the base date of this report i.e. April 1980. The average annual rate of inflation in the Tender and Building Cost indices over the last eight years is approximately 15% per annum, which would result in an increase of 26.5% from August 1978 to April 1980. However, in their November 1979 Interim Report (Reference 21), RPT suggest that a reduced figure would be more realistic. Accordingly, using their figure of 16% from August 1978 to November 1979, an annual rate of inflation of 12.8% has been taken in this report, with a resulting increase of 21.33% from August 1978 to April 1980.

18.2 Provision of Facility

In 1978 RPT allowed £500,000 per 6 cell OWC unit for provision of the building facility. The cost updated for inflation is £605,000. However, with the Breakwater Device being constructed in four cell units, the number of units produced in a given time will increase and, therefore, the facility cost per unit has been reduced by 33%. In addition the volume of concrete in the Reference Design is approximately 33% less than the 1978 Floating Reference Design, which will enable the concrete throughout to be increased by 25%. The facility cost per unit has therefore been further reduced by 20%. The resulting unit cost for provision of facility is £324,300 and the total capital cost per power station is £254 x 10^6.

18.3 The Structural Cost

The original 1978 and the updated 1980 unit costs for the main structural materials and formwork are given in Table 18.1. The unit cost figures for concrete placed insitu, formwork and reinforcement include for the slipformed sections of the structure. The unit cost for precast concrete includes for all concrete, ordinary and prestressed reinforcement, and shuttering materials necessary, and also allows for delivery from the precast factory to the construction yard.

Cost centre	Unit	Unit cost	
		August 1978	April 1980
Concrete placed insitu	m^3	£ 46.22	£ 56.08
Formwork	m^2	£ 22.75	£ 27.60
Reinforcement	t	£408.86	£496.08
Precast concrete Insitu	m^3	£199.77	£242.39

Table 18.1

Unit Costs for Structural Materials

The summary of structural quantities and resulting costs for the 2GW power station (consisting of 782 no. 4 cell units) is given in Table 18.2. Costs for the rock anchors are based on an estimate prepared by Colcrete Ltd at October 1979 (Reference 14) and are for the materials and specialist labour content only. Rock anchor constructional plant items are included in the cost of the emplacement barge.

Item	Unit Rate	Quantity	Cost
Concrete placed insitu	£56.08/m^3	10,620m^3	£595,600
Formwork	£27.60/m^3	9,130m^2	£252,000
Reinforcement	£496.09/t	1,790t	£888,000
Precast concrete	£242.39/m^3	2,230m^3	£540,500
Rock anchors Sept 79 cost Update cost(7.5%)	 £2,600 ea £2,795 ea	 150 No	 £419,300
Total capital cost for primary structure per 4 cell unit			£2,695,400
Total for 2GW power station			£2,108x10^6

Table 18.2

Primary Structure Quantities and Costs

18.4 Mechanical and Electrical Plant

Since the preparation of the 1978 report the characteristic size of the air turbine, rectifying valves and ducts have been reduced and the layout has been simplified. Therefore, the overall cost has been reduced by 50%. However, due to the exposed location of the plant on the Breakwater Device it has been necessary to include for the various watertight housings. The summary of costs under this heading is given in Table 18.3.

August 1978 base cost excluding enclosures per 4 cell unit		× 0.60665
Turbine ducting and valving	£940,000	£570 251
Alternators	£69,000	£ 41 858
Auxiliary equipment	£41,000	£ 24 873
sub total	£1051,000	
update cost	+ 21.33%	} = 0.60665
allow for reduced size	- 50%	
Sub total for M & E plant items	£637,000	
Watertight enclosures	£120,000	
Total capital cost for M & E Plant per 4 cell unit	£757,000	
Total for 2GW power station	£592x10^6	

Table 18.3

Mechanical and Electrical Plant Costs

18.5 Site Preparation

The cost of site preparation is dominated by the provision of the rock cutting dredgers. The capital and operating costs for these vessels are based on estimates for the 'Simon Stevin' provided by Royal Volker Stevin in late 1979. The resulting capital and annual costs per power station are given in Table 18.4.

Programme requirements		
Initial preparation vessels	2 No.	
Dredgers	2 No.	
Dump vessels	4 No.	
Initial preparation vessels		
Long term charter cost per annum	£440,000	
(August 1979 base date) update cost	+ 8.5%	
each	£477,400	
Overall annual cost	£1.0x10^6	
Total cost over 7 year programme		£7x10^6
Dredgers		
Capital cost		
(April 1980 base date) each	£50x10^6	
Write off proportion (50%) set against programme each	£25x10^6	
Total capital cost to programme		£50x10^6
Annual operating cost each	£2.5x10^6	
Total cost over 7 year programme		£35x10^6
Dump vessels		
Long term charter cost per annum	£365,000	
(August 1979 base date) update cost	+ 8.5%	
each	£396,000	
Overall annual cost	£1.6x10^6	
Total cost over 7 year programme		£11x10^6
Total capital cost of site preparation for 2GW power station		£103x10^6

Table 18.4

Site Preparation Costs

18.6 Towage and Emplacement Operations

Towage costs are based on information provided by London Offshore Consultants Ltd (Reference 23) at August 1979. The capital and operating cost of the emplacement barge is based on comparisons of anticipated size and complexity with the dredgers. The costs are presented in Table 18.5.

Programme requirements		
Large ocean going tugs for towage from construction yards to holding area and from holding area to site location	16 No.	
Small coastal tugs for local towage operations at construction yards, holding area and site locations	24 No.	
Emplacement barges	15 No.	
Ocean going tugs		
Long term charter cost per annum	£1,200,000	
(August 1979 base date) update cost	+ 8.5%	
each	£1,300,000	
Overall annual cost	$£20.8x10^6$	
Total cost over 7 year programme		$£146x10^6$
Coastal tugs		
Long term charter cost per annum	£292,000	
(August 1979 base date) update cost	+ 8.5%	
each	£320,000	
Overall annual cost	$£7.6x10^6$	
Total cost over 7 year programme		$£53x10^6$
Emplacement barges		
Capital cost (April 1980 base date) each	$£8x10^6$	
Write off proportion (70%) set against programme each	$£5.6x10^6$	
Total capital cost to programme		$£84x10^6$
Annual operating cost each	$£0.5x10^6$	
Total cost over 7 year programme		$£4x10^6$
Total capital cost of towage and emplacement operations for 2GW power station		$£287x10^6$

Table 18.5

Towage and Emplacement Costs

18.7 Power Take-off Equipment

Power take-off costs are based on the latest proposals by Kennedy and Donkin (Reference 24) for DC series connection of the generating equipment. However, the K&D proposals are for a floating power station located approximately 20km offshore, whereas the Breakwater power station is a maximum of 5km offshore. In addition, the Breakwater primary structures are closely spaced, with the gaps normally bridged with solid infill pieces thereby eliminating the requirement for flexible cable. Therefore, the K&D cost for medium voltage DC cable has been reduced by 65%. The resulting costs are given in Table 18.6.

October 1979 base cost per 4 cell unit	
Device Rectiformers	£25,000
MV DC cables	£29,000
sub total	£54,000
update cost	+6.4%
Total capital cost for power take off per 4 cell unit	£57,000
Total for 2GW power station	£45x10^{6}

Table 18.6

Power Take-off Costs

18.8 Transmission Equipment

The cost of transmission equipment is also based on the K & D proposals (as in section 18.7). However, due to the location of the Breakwater Power Station along Barra, South Uist and Benbecula only, the number of power collection stations has been reduced to three. The resulting cost saving has been taken as 15%. The summary of costs is given in Table 18.7.

October 1979 base cost per power station	
Power collection and convertor station HV DC cables HV overhead line HV DC invertor	£192x10^6 £39x10^6 £30x10^6 £77x10^6
sub total update cost	£338x10^6 + 6.4%
Total capital cost for transmission equipment per power station	£360x10^6

Table 18.7

Transmission Equipment Costs

18.9 Operation and Maintenance Costs

The operation and maintenance costs for the 2GW power station are based on figures prepared by EASAMS Ltd (Reference 25) for the inshore bottom standing HRS rectifier device. These figures include repayment and interest on all capital cost items. The resulting costs for the Breakwater power station when updated from the November 1979 base date in the EASAMS Report are:

Offshore operations	£20.6x10^6
Onshore operations	£4.5x10^6
Total	£25.1x10^6

18.10 Overall Costs

A summary of the capital cost for the 2GW installed capacity power station is given in Table 18.8. A range of values is presented, based on the mean costs derived previously, but with an optimistic assessment being taken as basic less 10% and a pessimistic assessment being taken as basic plus 20%.

Cost centre	Minimum -10%	Basic	Maximum +20%	% of total
	£x10^6	£x10^6	£x10^6	
Provision of facility	229	254	305	7
Primary structure	1,897	2,108	2,530	56
M&E Plant	533	592	710	16
Site preparation, towage & emplacement operations	351	390	468	10
Power take off	41	45	54	1
Power transmission	324	360	432	10
Total capital cost	3,374	3,749	4,499	100

Table 18.8

Summary of Capital Costs for
2GW Installed Capacity Power Station

The percentage breakdown of capital cost given in Table 18.8 shows that the highest cost centre is the primary structure, with the mechanical and electrical plant coming second.

The overall annual costs of the power station are presented in Table 18.9, with a rate of 5% compound interest over 25 years being taken for repayment of the capital cost. This is equivalent to repaying 7.1% of the capital each year. The resulting annual cost ranges from £263 x 10^6 to £349 x 10^6.

The cost of energy produced is given in Table 18.10. This shows that the unit energy cost varies between 6.7 and 18.3p with a mean value of 9.8p.

	Minimum -10%	Basic	Maximum +20%
	£x10^6	£x10^6	£x10^6
Total capital cost	3,374	3,749	4,499
Annual repayment of capital and interest (at 5% compound over 25 years = approx. 7.1% simple interest annually	240	266	319
Annual maintenance	23	25	30
Total annual cost	263	291	349

Table 18.9

Annual Cost of Power Station

	Units	Pessimistic	Mean	Optimistic
Average annual power delivered to Perth	kW	218x10^3	338x10^3	447x10^3
Total annual energy delivered to Perth (based on 8760 hours annually)	kWh	1,910x10^6	2,961x10^6	3916x10^6
Total annual cost	£	349x10^6	291x10^6	263x10^6
Energy cost	p/kWh	18.3	9.8	6.7

Table 18.10

Energy Cost

19. ECONOMIC ASSESSMENT

19.1 Labour Requirements

Experience in the construction of concrete oil production platforms has shown that outputs of between 80 and 100m^3 of concrete can be obtained per man per year. Therefore, based on a volume of concrete of 13,000m^3 per device with approximately 780 devices forming the 2GW power station, the amount of concrete to be placed per year is about 1,500,000m^3. Thus, a labour force of between 15,000 and 20,000 men is required.

A labour force of this size is the equivalent of the total labour force of three or four of the larger civil engineering contractors at present operating in the United Kingdom. It would seem unlikely that any major contractor would be prepared to commit its whole labour force to any project for a period of 7 to 8 years and therefore consideration should be given to methods of achieving the required production. Some possibilities are suggested below:

(a) Formation of a National Contracting organisation by Central Government.

It is considered that this is impractical unless a flow of work of similar type can be foreseen after the seven years of this project. Similarly, it would seem impractical to consider that such an organisation could be set up within the time span available, unless the whole of the contracting and civil engineering industries were nationalised.

(b) Formation of a consortium composed of three of four of the major civil engineering contractors.

It is considered that there would be considerable advantages to be gained from centralised purchasing, prefabrication, etc. Furthermore, as a number of construction yards are required, a considerable number of contractors could participate.

(c) Individual contracts could be let to a large number of contractors each based on one or more construction yards.

19.2 Industrialisation

It is considered that, as a number of the existing construction facilities are located in areas remote from existing pools of labour and that any further facilitites would be similarly situated, consideration must be given to one or more centralised manufacturing and/or prefabrication factories.

These factories would be located in areas which offer reasonable labour pools, and which are also adjacent to suitable transport terminii. Where the raw materials are transported by sea, it would then be possible to use the same ships for delivery of the prefabricated elements to the fabrication/assembly facility.

19.3 Materials

In view of the vast quantities of materials that would be required for this project in relation to U.K. annual output, consideration must be given to the need for ensuring regular supplies. The relationship between project requirements and U.K. annual output for the reinforced concrete work is as follows:

Material	Project Requirement Per Annum	U.K. Annual Output	
Concrete	1,600,000m^3	33,000,000m^3	5%
Reinforcement	220,000t	1,000,000t	22%

It is considered unnecessary to set up manufacturing plants for such basic materials as cement and reinforcement, but consideration might be given to gaining control of sufficient sources of aggregate as may be necessary. Similarly, so far as the electrical plant is concerned, it is felt that most equipment will be purchased from established manufacturers. However, there could well be merit in setting up a centralised factory for manufacture of ducting.

19.4 Energy Costs

The Government's policy of increasing the cost of gas to the consumer by an annual amount considerably in excess of the current inflation rate has been noted with considerable interest. Should this policy be extended in the future to other forms of energy, such as electricity, the "Net Present Value" of each unit sold will be increased.

Indeed as the cost of oil has already been increasing at a rate well in excess of general inflation, there is good reason to believe that electricity may be similarly affected in the future. Therefore, the economics of the wave energy power station are likely to improve.

20. ADVANTAGES AND FEATURES

The Oscillating Water Column Breakwater Device combines the economy of state-of-the-art structural design with highly efficient energy conversion equipment. The amount of structure forming the envelope of the primary piston has been reduced to a minimal level thereby giving a low ratio of structural material to piston volume. The structure can also be readily constructed using current methods that have been tried and tested in the development of the offshore oil industry. The site preparation and installation work can be carried out using constructional plant similar to that currently in existence, and the long term fixity and security of the device is assured by rock anchors, which again are widely used in civil engineering practice.

The energy conversion plant, which consists of a simple air driven radial flow turbine directly coupled to an alternator, has been optimised to reduce the number of steps in the conversion chain and to reduce the number of mechanical components as far as possible. This helps to maintain high efficiency levels with low capital cost and maintenance requirements, particularly in the severe offshore environment. Where major maintenance is required, the plant is packaged in relatively small modules which have been designed to be easily removed and taken ashore. This reduces the cost penalty implicit in both the alternatives of (i) major maintenance and repairs insitu offshore, or (ii) a lengthy service free working life.

The electrical power generated on board the Breakwater Device can be transmitted to shore using conventional submarine cables of relatively short length. Due to the mounting of the device on the seabed, there is no requirement for a flexible electrical umbilical. In addition, the extremely expensive cost centre of moorings is eliminated.

The OWC Breakwater Device therefore could provide an early stepping stone in the exploitation of wave energy. It could allow valuable experience to be gained on the operational side without the additional problems of sea-keeping in the extremely hostile marine environment. In addition, even with the very low current estimation of available energy (i.e. 17kW/m), it is estimated that there is potential for an installed capacity around the shores of the United Kingdom of the order of 6 to 7GW. There are further side benefits to be obtained due to the construction of the Device in long continuous lengths, which would provide shelter for various activities including deep water jetty or single point mooring installations, intensive offshore fish farming industries and recreational pursuits.

21. FURTHER WORK AREAS

Preliminary two-dimensional tank experiments on a simple fixed oscillating water column have shown that careful control of the phase of the air valves can enable considerable improvement to be made to the power characteristics. This dynamic phase control, which was first promulgated by Budal and Falnes, while perhaps not significantly improving the already extremely high efficiency of the Breakwater Device, may allow reduction in water column dimensions for similar power outputs, thereby giving considerable structural economies. The hydrodynamic and modelling aspects of this work area are projected for the 1980/81 programme, but the pre-engineering design of the air valves and control systems is also necessary at an early stage in order to establish the full potential of the concept in relation to the Breakwater Device.

The current design utilises a single secondary power generation pack for each wave piston. However, it is considered that combination of air flows from several pistons into a single air turbine (i.e. manifolding) may offer further significant benefits. These benefits would be apparent, not only on the capital first cost side, but also in the reduced offshore operations necessary for installation, operation and maintenance.

An early item in an on-going investigation of the Breakwater Device is the implementation of a fairly large scale sea trial. This would be necessary to verify the smaller scale model tests and to gain experience in the operation of the system in the 'real' environment. The use of a Breakwater type configuration with its inherent security would provide an excellent test bed for the evaluation of different types of generation equipment and their constituent components.

On the full size engineering design side, the lack of site information is an obvious deficiency. It would be essential to carry out a detailed topographical and geological survey at an early stage in the further development of the Breakwater Device. It would be possible to implement this survey in two phases, the first phase being over a relatively short length of coastline to allow installation of a full size prototype, and the second phase being the completion of the survey over the entire power station complex.

A second important aspect of detailed design that should be studied at the earliest opportunity is wave slam. Due to the reduction of structural content in the search for economy, it is anticipated that the natural response of certain of the Breakwater panels could be concurrent with wave slam impact durations, with resulting considerable magnification of structural stresses. However, the mechanism of wave slam is very diverse and has not been investigated to any significant extent for structures of the nature of the Breakwater. It is therefore essential that this aspect be studied well in advance of implementation of design work for a full scale power station.

22. CONCLUSIONS

The Oscillating Water Column Breakwater Device is a means of harnessing the energy in the waves to provide electrical power to the National Grid in significant quantities. The concept and design are elegantly simple and could be created by current techniques in the field of marine and offshore technology. The cost estimates therefore have a high degree of credibility.

The simplicity of the design at this stage provides scope for further sophistication and application of new and developing technologies. It also provides the starting point for step-by-step development towards higher and more efficient energy capture in the future.

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APPENDIX I

Design specification for March 1980

Breakwater Reference Design

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1. DESIGN BASIS

1.1 Design Philosophy

In general the global analysis of stresses in the structure will be carried out using linear elastic theory. Relative stiffnesses of members will be calculated using the entire concrete cross section ignoring the reinforcement.

Local stress assessment will be done using elastic analysis, plastic analysis or elastoplastic with buckling analysis methods where appropriate.

Concrete section design will be carried out using limit state design methods. In general it is found that serviceability limit states under marine conditions are more onerous than the ultimate or failure limit state, and therefore, the design of the section will usually be carried out for serviceability limit states and checked at the ultimate limit state.

Steel section design will be carried out using the maximum permissible stress approach. However, assessment will be made to ensure that yielding, buckling, brittle facture and fatigue limit states are not exceeded. (Note: This permissible stress design approach may eventually be replaced with a limit state approach as embodied in the new British Standard Specification for the Structural Use of Steelwork).

1.2 Design Phases

	Phase		Loading Conditions	Design Return Period
1.	Construction	A	In drydock	
		B	During float out	*
		C	At inshore floating berth	10 years
2.	Transportation	A	To holding area	10 years
		B	During connection of emplacement barge	*
		C	To site location	10 years
3.	Installation	A	Emplacement	*
		B	Post emplacement	10 years
4.	Operation	A	Normal environmental and system conditions, maximum imposed loads	1 month
		B	Extreme environmental and system conditions, maximum imposed loads	50 years
		C	Extreme environmental and system conditions, minimum imposed loads	50 years
		D	Damage or overload conditions	50 years
5.	Retrieval	A	Removal operations at site	10 years
		B	Transportation to disposal site	10 years

* Return period to be assessed.

1.3 Design Codes and Recommendations

1.3.1 General The Breakwater structure will be designed in accordance with the following codes and guidance notes:

Department of Energy - Offshore Installations Guidance on Design and Construction, 2nd Edition.

Lloyds Register of Shipping - Guidance Notes for the Structural Design of Wave Energy Devices.

1.3.2 Concrete Structures The following codes and recommendations will be used as appropriate:

British Standards Institution - CP110 : Part I : 1972 - The Structural Use of Concrete.

FIP - Recommendations for the Design and Construction of Concrete Sea Structures, 3rd Edition.

Lloyds Register of Shipping - Rules and Regulations for the Classification of Offshore Installations, Fixed Concrete Installations.

Lloyds Register of Shipping - Rules and Regulations for the Construction and Classification of Concrete Ships.

1.3.3 Steel Structures Where steel sections are used in conjunction with a mainly concrete structure, their design will be carried out using the permissible stress approach embodied in the following codes:

British Standards Institution - BS449 : Part 2 : 1969 - The Use of Structural Steel in Building.

British Standards Institution - BS153 : Parts 3 and 4 : 1972 - Steel Girder Bridges.

Reference may also be made to the draft of the new BS limit state code for Structural Steelwork i.e.

British Standards Institution - Draft for Public Comment 1977 - Specification for the Structural Use of Steelwork.

1.3.4 Reserved for Glass reinforced plastic (GRP)

1.3.5 Foundations The following codes and recommendations will be used:

British Standards Institution - CP2004 : 1972 - Code of Practice for Foundations.

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2. ENVIRONMENTAL DATA

2.1 General Information

2.1.1 Location of site

Outer Hebrides to West of South Uist.
Latitude 57° 00′ to 57° 25′N
Longitude 7° 30′W
National Grid Reference NF 71-73

2.1.2 Water depth

15-20m

2.1.3 Wind

Wind speeds at 10m above sea level for an average recurrence period of 50 years:

(a) Maximum 3 sec gust 56m/s (120 mph)
(b) Hourly mean speed 40m/s

2.1.4 Waves

Scatter diagram H_s against T_e for South Uist prototype site given in Figure 2.1

Design Values	Maximum Wave Height	Typical Period
Normal environmental Return period 1 mth	8m	10 secs
Extreme environmental Return period 50 yrs	12-15m	13 secs

2.1.5 Tides

	Tidal Range
Mean Springs	3.3 - 3.8 m
Mean Neaps	1.2 - 1.7 m

2.1.6 Tide and Storm surges

2.1.7 Currents

	Rate
Spring max	2.0 m/s (0.4kn)
Neap max	1.0 m/s (0.2kn)

2.1.8 Temperatures

(a) Air temperature range 1 - 14°C

(b) Sea temperature range 6 - 14°C

2.1.9 Marine Growth

See Scottish Marine Biological Report "Environmental Factors relating to possible sites for bottom standing OWCs to West of Hebrides" Oct. 1979.

4. CONSTRUCTIONAL MATERIALS

4.1 Reinforced and prestressed concrete

4.1.1 Concrete

	Grade 30	Grade 40
Characteristic strength N/mm^2	30	40
Age Factors - 6mths	1.20	1.19
12mths	1.23	1.25
Modulus of Elasticity kN/mm^2	28	31
Poisson's ratio (Serviceability limit state)	0.2	0.2

4.1.2 Reinforcement

	Hot Rolled Mild Steel (to BS4449)	Hot Rolled High Yield Steel (to BS4449)
Characteristic strength N/mm^2	250	410
Modulus of Elasticity kN/mm^2	200	200

4.1.3 Prestressing Steel

Seven Wire Strand (to BS 3617)

	Normal and low relaxation					
Diameter mm	6.4	7.9	9.3	10.9	12.5	15.2
Characteristic Strength N/mm^2	1820	1840	1790	1760	1750	1640
Modulus of Elasticity	200 kN/mm^2					

Nineteen wire strand (to BS 4757)

	as spun			normal and low relaxation
Diameter mm	125.4	28.6	31.8	8.0
Characteristic Strength N/mm^2	11550	1540	1480	760
Modulus of Elasticity	175 kN/mm^2			

4.2 Structural Steel

4.2.1 Mechanical Properties (to BS 4360)

Exposed Locations

	Minimum Grade	Minimum Yield Stress		Charpy Impact Value
		up to 16mm	16-40mm	
		N/mm^2	N/mm^2	
Plate	43D	280	270	41J@ -10°C 27J@ -20°C
Sections and Bars	43D	255	245	27J@ -15°C
Hollow Sections	43D	255	to be agreed	27J@ -15°C

Non-Exposed Locations

	Minimum Grade	Minimum Yield Stress		Charpy Impact Value
		up to 16mm	16-40mm	
		N/mm^2	N/mm^2	
Plate	43C	245	240	27J@ 0°C
Sections and bars	43C	255	245	27J@ 0°C
Hollow sections	43C	255	to be agreed	27J@ 0°C

All grades	
Modulus of Elasticity	206 kN/mm^2
Shear Modulus	79 kN/mm^2
Bulk Modulus	172 kN/mm^2
Poisson's Ratio	0.30
Co-efficient of linear thermal expansion	12×10^{-6} per °C

5. LOADS

5.1 Load Categories

Loading on the structure from the following causes will be evaluated:

5.1.1 Dead Loads (G_k)

(a) Weight in air of structure and superstructure;
(b) Fixed equipment;
(c) Ballast, wet or dry;
(d) Stored liquids;
(e) Hydrostatic external pressure and uplift force due to buoyancy in calm sea conditions calculated for the highest anticipated water level.

Characteristic values of dead loads are defined as the expected average values based on accurate data for the unit weight of the material and the volume in question.

5.1.2 Imposed Loads (Q_k)

(a) Construction and launching loads;
(b) Mooring and towing forces (including dynamic effects);
(c) Moveable equipment;
(d) Maintenance equipment;
(e) Helicopters landing, taking off or parked, if appropriate;
(f) Mooring of vessels to device.

Characteristic values of imposed loads are defined as the maximum permissible loads determined from an evaluation of the specified method of operation of the equipment, helicopters or vessels.

5.1.3 System Loads (S_k)

System loads are those that occur in excess of normal hydrostatic or atmospheric conditions due to operation of the device as an energy convertor. The following will be evaluated:-

(a) Hydrodynamic pressures in the water column chamber and entrance;
(b) Air pressures in the water column chamber and ducts leading to the air turbine.

Characteristic values will be obtained from a consideration of the full operating cycle using model testing or computer evaluation as appropriate.

5.1.4 Environmental Loads (V_k)

(a) Hydrodynamic pressures due to waves (other than associated with the system), including the effects of wave slam, slap and breaking;
(b) Currents;
(c) Wind;
(d) Ice;
(e) Earthquake.

For environmental loads, which are normally considered as random, characteristic values are defined as the most probable largest value for a return period equal to the expected duration of the phase under consideration.

The combination and severity of environmental loads used in design will be consistent with the probability of their simultaneous occurrence. Earthquake loads will be combined only with normal environmental conditions (i.e. design stage 4A).

5.1.5 Deformation Loads (D_k)

(a) Prestress including that due to rock anchors;
(b) Shrinkage and expansion;
(c) Creep;
(d) Temperature variation;
(e) Differential settlement;
(f) Absorption.

Characteristic values of deformation loads will be derived from an evaluation using both maximum and minimum values of the governing parameters.

5.1.6 Accidental Loads (A_k)

(a) Collisions;
(b) Explosion or implosion;
(c) Dropped objects;
(d) Exceptional earthquake.

Accidental loads are generally ill-defined in intensity and frequency of occurrence. Their effects will be evaluated by combining the probability of occurrence with the likely extent of the resulting damage.

6. STABILITY ANALYSIS

6.1 Floating Stability

Floating stability will be assessed during appropriate phases of the construction and installation programme as follows:

6.1.1 Intact Stability

Design phase	1B,1C	2A,B,C 3A
Minimum Freeboard	4m	2m
Minimum Initial GM Value	0.3m	0.5m
Maximum list	$1\frac{1}{2}^{o}$	
Minimum range of stability	35^{o}	40^{o}
Area under statical stability curve* / Area under wind heeling moment curve (Minimum values)	1.4	1.4

* Notes

(i) Wind velocity (1 minute mean speed) taken for 10year return period storm, but not less than 35m/s for sheltered inshore waters or 50m/s for offshore waters.

(ii) Areas calculated up to either the second intercept of the two curves or to the point at which flooding commences, whichever is less.

In addition the structure will be water tight such that no ingress will occur during the 10 year return period storm.

Displacement, buoyancy and ballast values will be calculated at all stages of immersion during phases 1 and 3 to ensure that the stability criteria can be met.

6.1.2 Damage Stability

Design phase	All floating
Minimum freeboard (to any significant opening)	0.05 m
Minimum GM value (using constant displacement method)	0.05 m
Maximum list	15°
Minimum GZ value at damage condition	>0
$\frac{\text{Minimum residual righting moment}}{\text{Wind heeling moment}}$ *	>1.0

* Wind velocity (1 minute mean speed) not less than 25 m/s.

6.2 Post Emplacement Stability

6.2.1 Limit States

(a) Ultimate (i) Sliding
(ii) Overturning

(b) Serviceability (i) Deflection

6.2.2 Ultimate safety factors

Note - Values below are for overall factor of safety and are intended for use in the March 1980 Reference Design. For more advanced design work a fuller assessment of partial load factors for all load categories and partial material factors will be carried out.

Ultimate Limit States	Design Phase				
	3B	4A	4B	4C	4D
	NB. Values given are o/a values				
Sliding	1.2	1.7	1.4	1.3	1.1
Overturning	1.4	2.0	1.7	1.5	1.1

7. DESIGN OF CONCRETE SECTIONS

7.1 Limit States

(a) Ultimate limit states:
- (i) Strength
- (ii) Buckling
- (iii) Brittle failure
- (iv) Implosion
- (v) Progressive collapse

(b) Serviceability limit states:
- (i) Deflection
- (ii) Vibration
- (iii) Cracking
- (iv) Corrosion
- (v) Durability
- (vi) Fatigue

7.2 Serviceability Limit States

7.2.1 Partial Safety Factors

(a) Loads:
Partial load factor = 1.0 for all load categories and all design phases.

(b) Materials:

	Deflection Vibration	Cracking	Fatigue
Concrete	1.0	1.3	1.5
Steel	1.0	1.0	1.15

7.2.2 Deflection

Deflections will be limited to those given in CP110 Clause 2.2.3.1.

Long term deflection calculations will assume that the maximum level of stress is applied constantly. (This will allow for creep deflection and should yield reasonably accurate answers - Reference DoE Guidance Notes).

7.2.3 Vibration

In general, since the structures are normally unmanned, vibration limit state criteria will be set by the requirements of the M & E Plant. However, during periods when maintenance is being carried out vibrations should be limited to Category II as given in the DoE Guidance Notes Clause 5.9.2.3.

7.2.4 Cracking

Exposure zones will be classified as follows:

External surfaces

Category	Zone	Limits of Zone
Su	Submerged	Below lower limit of splash zone
Sp	Splash	10m under LWST level to HWST level + 50 year (average) wave amplitude
At_1	Atmospheric (exposed zone)	Above upper limit of splash zone but exposed to salt water spray
At_2	Atmospheric (sheltered zone)	Sheltered from salt water spray but exposed to rainwater

Internal surfaces

Category	Zone	Limits of Zone
Su	Submerged	Below highest level of retained liquid
At_3	Atmospheric	Above submerged zone but subject to heavy condensation or corrosive fumes

(a) Reinforced concrete

Cracking limit state criteria for reinforced concrete will be defined as follows:

Class I Maximum crack width anywhere to be 0.3mm
Class II Crack width above main reinforcement not to exceed 0.004 x nominal cover.
Class III Tensile stress in reinforcement not to exceed $0.8f_y$.

Design cracking criteria

Design Phase	Zone		
	Su	Sp	At_{1-3}
1A,B	III	III	III
C	II	III	III
2A,C	II	I or II	I or II
B	III	III	III
3A	III	III	III
B	II	II	II
4A	II	I or II	I or II
B,C	III	III	III
5A,B	III	III	III

The method used to calculate crack widths will be that given in CP110 Appendix A. The allowance for tension stiffening effect in concrete embodied in the method will not be taken where there are cyclic loads whose magnitude exceeds 50% of the steady loads at the design phase under consideration

i.e. $Q_k + S_k + V_k \ngtr \frac{1}{2}(G_k + D_k)$.

(b) Prestressed concrete

Cracking limit state criteria for prestressed concrete will be defined as follows:

Class 1 No flexural tensile stress
Class 2 Flexural tensile stresses limited to ensure no visible cracking
Class 3 Maximum crack width 0.1 mm
Class 4 Maximum crack width 0.2 mm

Design cracking criteria

Design Phase	Zone		
	Su	Sp, At_1	$At_{2\ 3}$
1A,B	3	2	4
C	2	2	3
2A,C	2	1	3
B	3	2	4
3A	3	2	4
B	2	2	3
4A	2	1	3
B,C	3	2	4
5A,B	4	4	4

For Class 1 and 2 structures the section remains uncracked at the limit state and the cracking criteria will therefore be satisfied if the flexural tensile stress values given below are not exceeded.

For Class 3 and 4 structures the section is cracked at the limit state and the method used to calculate crack widths will be based on that used for reinforced concrete sections. However, in certain cases (see CP110), the criteria may be satisfied by assuming that the section is uncracked and then limiting the flexural tensile stresses to the values given below.

Permissible flexural tensile stresses (N/mm^2)

Class	Crack Width	Group	Concrete Grade		
			30	40	50
2	0	a	–	2.9	3.2
		b	1.7	1.8	2.0
3	0.1	a,b	3.2*	4.1	4.8
		c	–	5.3	6.3
4	0.2	a,b	3.8*	5.0	5.8
		c	–	6.3	7.3

* Group b only

Group	Type of Construction
a	Pre-tensioned
b	Grouted post-tensioned
c	Pre-tensioned with tendons located at outer edge of tensile zone only

Class 3 and 4 structures only
Depth factors

Depth of member mm	$\leqslant 200$	400	600	800	$\geqslant 1000$
Flexural tensile stress modification factor	1.1	1.0	0.9	0.8	0.7

Additional reinforcement
(positioned at outer edge of tensile zone)
amount = 1% cross-sectional area of concrete

Group	Increase in permissible flexural tensile stress (N/mm^2)
a,b	4.0
c	3.0

Absolute maximum tensile stress - $0.25f_{cu}$

7.2.5 <u>Corrosion and durability</u>

In general the serviceability limit states of corrosion and durability will be satisfied by the following reinforcement cover and concrete specification requirements:

see 7.2.4. for zone definitions

Zone	Concrete Grade	Minimum Cover		Min cement content	Max w/c ratio
		non stressed	stressed		
		mm	mm	kg/cm^3	
Su	min 40	60	75	400	0.40*
Sp	min 40	75	100	400	0.40*
At_1	min 40	60	75	400	0.40*
At_2	min 30 40	40 30	40 30	360(320)**	0.45
At_3	min 30 40	40 30	40 30	360(320)**	0.45

Notes: * Preferable value, absolute maximum 0.45
** For aggregate maximum size 20(40)mm

Where subject to severe scouring or abrasion characteristic strength of concrete should be increased by 5 N/mm^2.

7.2.6 Fatigue

The following stress limits will be applied at design phases 1C, 2A, 2C, 3B, 4A to loading levels which only occur in excess of 20,000 times.

(a) Concrete

Maximum stress range $\not> \frac{1}{\gamma m} (0.4f_{cu} - 0.5f_{min})$

where f_{cu} = characteristic strength of concrete

f_{min} = minimum stress

(b) Non stressed deformed reinforcement

Maximum stress range $\not> \frac{1}{\gamma m} (0.33\ f_y).\ F_1$

up to f_{min} $0.4f_y$

where f_y = characteristic strength of steel

and F_1 = 1 for straight bars

½ for bent or welded bars.

(c) Prestressing tendons (strand)

Maximum stress range $\not> \frac{1}{\gamma m} (0.10f_{pu})$

for f_{min} $0.65f_y$

where f_{pu} = characteristic strength of tendons

Alternatively the fatigue limit state for prestressing tendons will be considered to be satisfied if the nominal tensile stress in the pre-compressed tensile zone does not exceed the following values:

Concrete grade	30	40	50
Permissible tensile stress N/mm^2	2.50	2.85	3.20

7.2.7 Additional requirements for prestressed concrete

(a) Compressive and tensile stresses at transfer will not exceed those given in CP110 Para 4.3.3.3.

(b) Maximum compressive stress under any load combination at service conditions in any design phase will not exceed the following values:

Nature of Loading	Allowable Compressive Stress
Combined bending and compression	$0.33\ f_{cu}$
Direct compression only	$0.25\ f_{cu}$

(c) Maximum initial force in prestressing tendons will not exceed 70% f_{pu}.

(d) The following factors will be taken into account when assessing prestress losses:
- (i) Relaxation of steel in the prestressing tendons;
- (ii) Elastic deformation of the concrete;
- (iii) Shrinkage and creep in the concrete;
- (iv) Slip and movement of the tendons during anchoring;
- (v) Frictional losses in post-tensioned tendons due to unintentional variation from the specified duct profile and to design curvature where appropriate.

7.3 Ultimate Limit States

7.3.1 Partial safety factors

(a) Loads

Load Category	Design Phase				
	1,2&3	4A	4B	4C	4D
G_k	1.2	1.2	1.1	0.9 or 1.1	1.05
Q_k	1.6	1.6	1.2	0.9	1.05
S_k	1.4	1.4	1.2	0.9 or 1.2	1.05
V_k	1.4	1.4	1.2	1.2	1.05
D_k	1.1	1.1	1.1	1.1	1.05
A_k					1.05

Notes:

(i) It will be assumed that the structure will be unmanned during phases 4B and 4C.

(ii) Partial factor for prestress D_k will be taken as 0.9 if this value leads to a more unfavourable design case.

(iii) Partial factors for dead loads G_k and system loads S_k will be taken so that the most unfavourable design case occurs.

(iv) In phases 1, 2 and 3 partial factors will be taken as 1.05 where the structure is subjected to short term hydrostatic or other similarly well defined loads which occur only once in the life of the structure.

(v) In phase 4D only those loads acting simultaneously will be considered.

(b) Materials

	Strength Assessment	Excessive loads or local damage
Concrete	1.5	1.3
Steel	1.15	1.0

7.3.2 Design and detailing

Generally the design of sections at the ultimate limit state in bending or compression will be carried out using the methods given in CP110, and in shear and torsion will be carried out using Appendix 1 of Part 4, Chapter 3 of Lloyds Register Rules and Regulations for the Classification of Offshore Installations.

APPENDIX II

Summary of Structural Loadings for
March 1980 Breakwater Reference Design

Design Phase 2A - Transportation to holding area

Hull girder and local hydrostatic and hydrodynamic stresses determined using 'Provisional Rules and Regulations for the Construction and Classification of Concrete Ships' given by Lloyds Register of Shipping.

Design Phase 4A - Normal Environmental and System Conditions

Lowest still water level - depth taken as 16m
Highest still water level - depth taken as 20m

Overall sliding and overturning loads determined from an analysis of the hydrodynamic pressure distribution in a standing wave using second order theory.

Return period 1 month - wave height 5.5m

Depth m	Horizontal force t/m		Overturning moment t-m/m	
	Crest at structure	Trough at structure	Crest at structure	Trough at structure
16	139	-64	1484	-474
20	145	-80	1828	-759

NB - ve sign indicates change of direction of loading.

Coefficient of friction under base 0.45.

Hydrodynamic local loadings

(a) Nose and end walls

Pressure distribution taken from standing wave analysis

Crest at structure

Depth m	Pressure above hydrostatic kN/m^2			Maximum water level above SWL m
	Point 1	Point 2	Point 3	
16	56	74	0	8.3
20	49	67	0	7.4

NB (i) Point 1 is sea bed
Point 2 is mean sea level
Point 3 is point of maximum freeboard

(ii) At low still water level freeboard is 15m
At high still water level freeboard is 11m

Water Column pressure taken as $-20kN/m^2$
Water Column amplitude taken as -2m

Trough at structure

Depth m	Pressure below hdyrostatic kN/m^2		Maximum water level below SWL m
	Point 1	Point 4	
16	39	49	-3.8
20	37	49	-4.1

NB Point 1 is sea bed
Point 4 is water surface level

Water Column pressure taken as $+20kN/m^2$.
Water Column amplitude taken as +2m.

(b) Rear Wall

Parameters for waves approaching from rear

Maximum fetch 8 km
Wind speed taken as 10 m/s steady over 36 hour duration
Maximum wave height 1m
Maximum wave period 3 secs

Crest at structure

Depth m	Pressure above hydrostatic kN/m^2		Maximum water level above SWL m
	Point 1	Point 2	
20	-2.2	11.5	1.2

NB Freeboard taken as 6.5m

Trough at structure

Depth m	Pressure below hydrostatic kN/m^2		Maximum water level below SWL m
	Point 1	Point 4	
20	2.2	9.3	0.8

Design Phases 4B & 4C - Extreme Environmental and System Conditions

Overall sliding and overturning loads determined as for Phase 4A

Maximum Values - return period 1 to 50 years

Depth m	Horizontal force t/m		Overturning moment t-m/m	
	Crest at structure	Trough at structure	Crest at structure	Trough at structure
16	310	-98	4150	-699
20	374	-151	5666	-1347

NB - ve sign indicates load in opposite direction.

Coefficient of friction under base - 0.45

Hydrodynamic local loadings

(a) Nose and end Walls

Pressure distribution taken from standing wave analysis.

Crest at structure

Depth m	Pressure above hydrostatic kN/m^2		
	Point 1	Point 2	Point 3
16	101	136	18
20	101	149	64

Water Column pressure taken as $-80kN/m^2$
Water Column amplitude taken as -5m

Trough at structure

Depth m	Pressure below hydrostatic kN/m^2		Maximum water level below SWL m
	Point 1	Point 4	
16	66	70	4.0
20	79	89	5.4

Water Column pressure taken as $+80kN/m^2$
Water Column amplitude taken as +5m

(b) Rear Wall

Parameters for waves approaching from rear

Maximum fetch 8 km
Wind speed taken as 26 m/s steady over 12 hour duration
Maximum wave height 3m
Maximum wave period 4.5 secs

Crest at structure

Depth m	Pressure above hydrostatic kN/m^2		Maximum water level above SWL m
	Point 1	Point 2	
20	-7.8	31.5	3.4

Trough at structure

Depth m	Pressure below hydrostatic kN/m^2		Maximum water level below SWL m
	Point 1	Point 4	
20	10.1	28.8	2.1

Design Phase 4D

The following accidental situation considered:-

Extreme environmental conditions, Water Column closed off and empty (i.e. in maintenance mode)

APPENDIX III

Summary of Calculated Stability Parameters for March 1980 Reference Design

N.B. Calculations carried out for 68m long unit

Phase 2A Transportation from construction site to holding area

	per 4 cell unit
Displacement	31600t
Ballast	2300t
Still water draft	22.1m
Minimum freeboard	4.5m
Water plane dimensions	67.5x22.5m
Static metacentric height	1.0m

Phase 3A Combined structure and emplacement barge - just prior to emplacement

Displacement of combined structure	56800t
Extra displacement provided by barge	44000t
Still water draft	10m
Outside dimensions of water plane	72.0x82.5m

Phase 3B Immediately post emplacement prior to installation of rock anchors

Horizontal wave load	25200t
Overturning moment	382000t-m
Net weight of structure	22400t
Net weight of ballasted emplacement barge	45000t
Overall net weight	67400t
Coefficient of friction between structure and bed rock	0.45
Net resistance against sliding	30300t
Net resistance against overturning	759000t-m
Factor of safety against sliding	1.2
Factor of safety against overturning	2.0

Phase 4 Operational mode

Water depth	20m
Net weight of structure	22400t
Extra vertical load from rock anchors	51800t
Horizontal reaction from rock anchors	9100t
Coefficient of friction	0.45

	Phase 4A Normal environment		Phase 4B Extreme environment	
	1	2	1	2
Wave Loads -horizontal -overturning	 9790t 123420t-m	 5400t 51250t-m	 25250t 382460t-m	 10190t 91330t-m
Net resistance -against sliding -against over-turning	 42470t 699550t-m	 24340t 1067280t-m	 as for Phase 4A	 as for Phase 4A
Factor of safety -against sliding -against over-turning	 4.3 >5.0	 4.5 >5.0	 1.7 1.8	 2.4 >5.0

N.B. (1) = Crest at structure
(2) = Trough at structure (direction of wave load reversed)