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GEOMECHANICAL INVESTIGATION OF CORAL REEF STRUCTURE

Report submitted to

GREAT BARRIER REEF MARINE PARK AUTHORITY,
TOWNSVILLE

by

Dr. Helmut Bock
Senior Lecturer in Civil Engineering

DEPARTMENT OF CIVIL AND SYSTEMS
ENGINEERING

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AUTHOR: Dr H. Bock.

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The Authority will not publish the report.

Name of Project Officer: Richard Kenchington.

With its letters of 22nd and 23rd March, 1979 the Great Barrier Reef Marine Park Authority, Townsville granted funds in support of the following two research projects:

> "A Review of the Design and Construction Principles of Structures on Coral Reefs" and
> "Geomechanical Investigation of Coral Reef Structure".

These research projects have been continuously carried out throughout the year 1979 with what is believed to be quite considerable success. The results of this research are presented in three independent papers which are enclosed in the appendix.

Appendix 1: Summarising the activities, referring to the principal problems when building on coral reefs and presenting the main results so far gained:
BOCK, H. & E.T. BROWN: "Foundation Properties of Coral Reefs - Site Investigation Techniques and Preliminary Results".

Appendix 2: Referring to the site investigations on coral reefs undertaken in the year 1979:
BILLINGHAM, S.L.: "The Study of Site Investigation Techniques for determining the Engineering Properties of Coral Reefs".

Appendix 3: Referring to a Literature Study which made use of hardly accessible documents in different governmental institutions throughout Australia:
BULGARELLI, G.: "A Review of the Design and Construction Principles of Structures on Coral Reefs".

Townsville, 20 December 1979

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Appendix 1

BOCK, H. & E.T. BROWN:

"Foundation Properties of Coral Reefs - Site Investigation Techniques and Preliminary Results".

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FOUNDATION PROPERTIES OF CORAL REEFS - SITE INVESTIGATION TECHNIQUES AND PRELIMINARY RESULTS

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SUMMARY Foundations on coral reefs are becoming prolific in different parts of the world. After discussing some general foundation problems which are encountered on reefs and referring to a number of actual cases, the paper outlines activities which have been undertaken within the Department of Civil and Systems Engineering of James Cook University, Townsville since 1972. The objective of these activities is the determination of the near-surface geotechnical properties of coral reefs, down to a depth of about 25 m. The work was and will continue to be carried out in pursuit of the following two aims.

Its first objective is to study the feasibility of different site investigation techniques on coral reefs. A total of three techniques have been applied: diamond drilling, dynamic penetration and seismic refraction. It has been found that dynamic penetration is by far the best method of exploring the near-surface structure of coral reefs in detail. It is not only capable of penetrating through hard coral limestone but it also enables the determination of weak zones such as cavities or pockets of loose sand and it is these zones which will be critical for any foundation design. The second objective of the project is to gain more knowledge on the principal distribution of the dominant geotechnical units (coral limestone, sand and cavities) over the area of an individual coral reef. For this, Keeper Reef (approximately 60 km northeast of Townsville) was systematically investigated alongside a section running from the outer reef into the lagoon. First results will be presented indicating that in both horizontal and vertical directions there are characteristic changes in the near-surface reef structure. The engineer should be familiar with these changes when planning a foundation on coral reefs.

1 INTRODUCTION

Coral reefs of one sort or another are found in all warmer oceans (Stoddart 1969; Ladd 1977). Major reef provinces are found in the Indian and Pacific Oceans, the Red Sea, the Caribbean and the Arabian Gulf. The world's largest coral province is Australia's Great Barrier Reef, so named by Captain James Cook in 1770.

Coral reefs and, in particular, The Great Barrier Reef have long been a source of great fascination and scientific interest. Cook himself wrote in his journal of 17 April, 1777, "There are different opinions amongst ingenious theorists, concerning the formation of such low islands". In recent years, however, coral reefs have become of increasing engineering interest. Because of the proximity of reefs to continental shore lines in certain parts of the world and because large numbers of coral islands exist in the Indian and Pacific Oceans and in the Caribbean, engineered structures are being increasingly founded on corals. A majority of these structures may be classed as marine structures (docks, jetties off-shore oil terminals, navigational aids). Some, such as a platform constructed as a scientific base in the Red Sea, are more exotic, while others, such as the multistorey buildings and airports constructed on islands such as the Seychelles, are of a more conventional nature. In all cases, the unusual geotechnical properties of the coral deposits have caused construction difficulties.

This paper describes the research into the near-surface geotechnical properties of coral reefs that has been undertaken at James Cook University of North Queensland in recent years. The authors' initial interest in the subject was stimulated by a request that they erect a radar beacon on The Great Barrier Reef off Townsville (Cameron 1975), but this experience soon led to the wider study of site investigation techniques for use on coral reefs that is reported herein.

2 THE NATURE AND GENESIS OF CORAL REEFS

Coral reefs are built in clear, warm, shallow water primarily by anthozoan corals of the class Scleractinia, of which the hermatypic of reef-building members are remarkable for their ability to construct massive skeletal structures of calcium carbonate (Stoddart 1969). Minor contributions to the accumulation of calcium carbonate are made by other corals and by other reef-dwelling organisms such as molluscs, foraminifera and algae. The calcium carbonate framework is capable of withstanding wave action and, as a result, may form topographic features which rise from the sea floor to sea level.

Following Darwin (1842), coral reefs were classically considered to be of three types - fringing reefs, barrier reefs and atolls. Darwin's concept was that fringing reefs are formed at shallow depths around landforms such as islands formed by the tips of submerged volcanoes. With continued subsidence of the landmass (or rise in the sea level) and continued reef growth, a barrier reef is formed. Such reefs occur some distances from coasts, being separated from them by lagoons that are generally too deep to permit coral growth. As a final stage, the island becomes completely submerged, and a ring of coral or atoll, enclosing a salt water lagoon, is formed.

Recent workers in this field such as Maxwell (1968) and Stoddart (1969) have drawn a sharp distinction between *oceanic reefs* (the Darwinian fringing and barrier reefs and atolls), and *continental or shelf reefs* found, as the name implies, in the shallow waters of continental shelves. The Great Barrier Reef is of this latter type. The Great Barrier Reef has been most closely studied by Maxwell (1968; 1973), who distinguished some 13 morphological reef types, in addition to fringing reefs bordering the coast and continental islands. The 13 types each belong to one of two major groupings - platform reefs and wall reefs. *Platform reefs* are formed when the factors controlling their development (water temperature, current and wind directions, topography) exert equal influence in all directions producing a roughly circular reef platform. A typical cross-section showing the various zones of such a reef is shown in Figure 1a. An important feature of many of these reefs is the hard algal rim that grows up to a metre above low tide level.

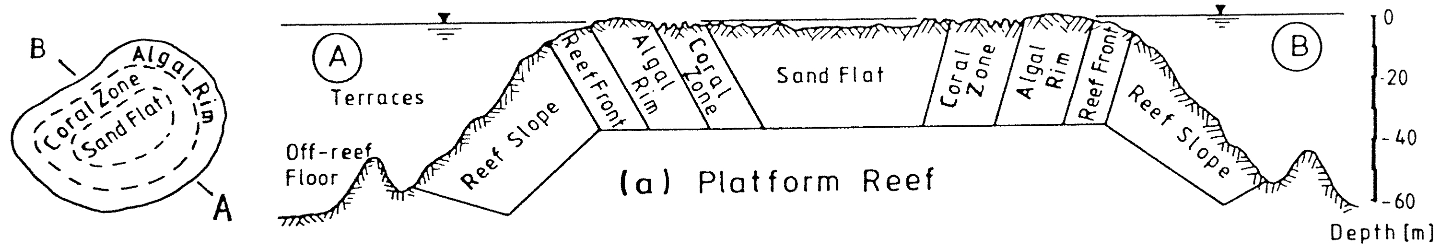
If, on the other hand, hydrologic and topographic influences cause the reef to develop in a preferred direction, a wall reef is produced. Keeper Reef, the subject of the site investigation studies to be reported below, is the "open-ring" variety of this type as indicated in Figure 1b.

In addition to coral reefs, The Great Barrier Reef province also contains coral islands of a number of types.

Deep borings in reefs (Ladd et al. 1953; Traves 1960; Goodell et al. 1969) have conclusively confirmed the general argument of Darwin that oceanic reefs developed by progressive subsidence of their foundations. Such reefs may be hundreds, or even thousands, of metres thick. The deepest borehole in The Great Barrier Reef province was drilled to a depth of 578 m at Wreck Island in 1959 (Goodell & Garman 1969). This bore went through coral rock into quartz sandstone at 121 m and then through a series of sandstones and calcarenites reaching basement rocks at 547 m.

Darwin's theory was developed before the general recognition of continental glaciation and the attendant shifts of sea level in the Pleistocene, and so omitted an important control on the development of reefs. In a series of papers published between 1910 and 1920, R.A. Daly drew attention to the consequences of sea-level shifts on reef morphology and proposed a glacial control theory of coral reefs. Daly argued that marine abrasion during glacial low sea levels prepared platforms on which reefs grew in post-glacial time. Reefs were generally unable to grow during low sea levels because of lowered ocean temperatures and the presence of sediment in the water.

Although the details of Daly's theory have been shown to be deficient in many important respects (Stoddart 1969), its essential concept is accepted. This has



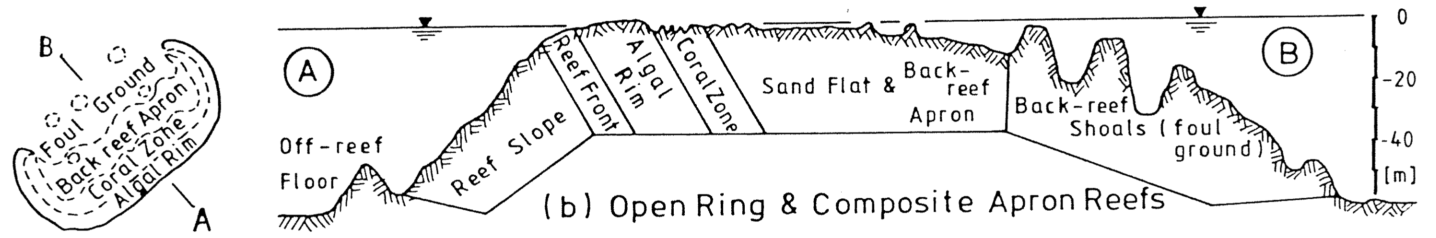


Figure 1 Plans and cross section of two types of shelf reefs (a) Platform reef, (b) Open ring and composite apron reef. (After Maxwell 1968: 106)

important geotechnical implications in that disconformities in the reef sequence might be expected to be present at depths corresponding to previous sea-or erosional levels. There are conflicting interpretations of the geological and geomorphological evidence (Stoddart 1969; Maxwell 1973) but, according to most recent investigations, it appears that in The Great Barrier Reef such disconformity, formed during the Holocene transgression, might be expected at depths of approximately 20 m below present sea level (Davies et al. 1977; Thom et al. 1978).

The calcareous skeleton produced by the corals and other organisms is a structure high in pore space and often containing large cavities left between sections of the skeleton. As the reef grows upwards, these cavities, which generally have communicating passages with the sea, become filled with sediment, including reef fragments and coralline sand. Cementation of the mass takes place by a number of processes including chemical precipitation by circulating sea water and marine organisms. It has also been found that deeper in the reef, solution, particularly of residual aragonite, may take place. The reefs are also subject to attack by marine borers, and cycles of solution and reprecipitation may also accompany changes in sea level. The net result is that, although the coral in the upper 100 m or so becomes lithified into what Maxwell (1962) describes as shallow reef rock or algal-coral limestone, it may remain almost as porous as when it was first formed. The wide-spread presence of cavities, generally filled with sand, is a major geotechnical characteristic of coral reefs (Field & Hess 1933; Broadhead 1970; Ridgway 1977; Dennis 1978).

3 ENGINEERED STRUCTURES FOUNDED ON CORAL

A wide variety of engineered structures have been founded on modern coral reefs or islands and on fossil coral reefs. These structures have included drilling platforms, scientific platforms, navigational aids, off-shore oil terminals and tanker berths, a variety of harbour and port structures including breakwaters, jetties and docks, airstrips, and multi-storey buildings such as hotels. These structures are generally, though not always, supported on piled foundations. Excavation of corals and coral-derived reef and beach-rock to form shipping channels and berths, for example, presents another class of engineering problem.

Drilling in coral and coralline sediments, for whatever purpose, is often fraught with difficulty. Core recoveries are low, running sands and hole collapses cause jamming and/or loss of the drill string, and drilling fluid circulation may be lost (Ladd et al. 1953; Field & Hess 1933; Broadhead 1970; Ridgway 1977). Pile-driving and grouting are also attended by difficulties associated with the porous and cavernous nature and vertical variability of the materials. As a result, contractors' claims for compensation for extra costs incurred by "unforeseen conditions" are not uncommon. For example the second author was recently involved in a major arbitration case arising from a large off-shore piling operation in corals in the Caribbean. In this case, some 75 cylindrical steel piles having diameters ranging from 1.068 to 2.502 m and with nominal lengths of 42.7 to 55.0 m were required to be driven to refusal (defined as 20 blows or more for the last inch of penetration) and provided with a grouted anchor system to resist uplift. Difficulties were experienced in obtaining the specified refusal, with some piles being driven several meters below design depth at very low resistance levels. Where piles were driven through intact coral limestone, the plugs of rock left inside the piles proved difficult to remove. The presence of cavities distributed irregularly throughout the rock caused great difficulties in drilling the 180 mm diameter anchor holes to an average depth of 22 m below pile tip elevation. Cave-ins caused the drilling tools to become jammed and sometimes lost, and the wide-spread presence of cavities resulted in the consumption of several times the quantity of grout anticipated at the tender stage.

Experiences such as this suggest that improved site investigation and construction techniques are required if foundations are to be more effectively constructed on coral reefs. Because of the unique environment and geotechnical characteristics of corals, special site investigation techniques may be required. A major need is for the development of techniques that can be used to locate the cavities and pockets of sand that provide the major source of difficulty in foundation construction on coral. It is with the development of such techniques that the rest of this paper is concerned.

4 SITE INVESTIGATION ON CORAL REEFS

4.1 General

After what has been outlined in Section 2 it becomes evident that the best basis for an evaluation of foundation properties on coral reefs will be a comprehensive study of the regional geology and geomorphology. This will provide a framework within which the detailed information, subsequently gathered by local site investigations, can most adequately be interpreted. Maps and air photographs will be the principal sources of information at this early stage of site investigation. The importance of air photographs, in particular, can hardly be over-emphasized. They provide the basis not only for regional studies but also for the identification of more local features which may be of concern for a particular foundation site. A good example in this respect is sinkholes, which are quite common on coral reefs (Purdy 1974); sinkholes often can clearly be recognized in air photographs, however hardly identified from the reef surface, as they are usually filled with reef detritus or overgrown by patchy corals. In addition, air photographs are often the only tool for defining actual positions on coral reefs, which is a delicate question when, for example, moving across the reef or selecting a location for a site investigation test.

It will rarely happen that the information from these general sources can be considered as sufficient. Special investigations such as drilling or the application of geophysical methods have to be carried out. When trying to do this in the environment of coral reefs, two main questions arise:

1. What technique can be applied in order to shift the equipment to a particular reef location at which a test should be carried out? (Access problem)
2. What methods and equipment should be selected in order to gain appropriate information on the foundation properties of coral reefs? (Data acquisition problem)

These two questions will be discussed in turn.

4.2 Access to the Site

The following technique has been developed and found most useful: After sailing aboard a motor vessel into a sheltered position at the back of a reef, the equipment such as drilling rig or heavy penetrometer (ref. Section 4.3) is loaded onto a raft (details: see Appendix 1). Preferably during the outgoing high tide, the raft is manoeuvred to the desired reef location using an outboard motor (Fig. 2). Here the raft is anchored.

If the reef surface falls dry during the low tide then the test is commenced as soon as the raft is settled onto the reef surface (Fig. 3). Depending on factors such as reef morphology, weather and tide conditions a time span of up to three hours is available for performing the test, operating from a completely *fixed position*.

If the reef surface is still submerged even at low tide then the raft is anchored at four points. Each anchor is operated via a 5 mm steel cable by a hand winch, mounted on one of the corners of the raft. By this technique, which allows for adjustment to changing weather and tide conditions, the raft can be held almost stationary. Furthermore, the particular wave pattern inside a reef, usually consisting of waves of small amplitudes and wave lengths (choppy conditions), imposes only a minimal degree of lurching of the raft. It was found that, due to these favourable circumstances, a site investigation test could be performed even from a floating position, but only if the wind velocity is not exceeding 20 knots. When combining these two techniques, almost any point of a coral reef surface is accessible and can be thoroughly investigated for its foundation properties.

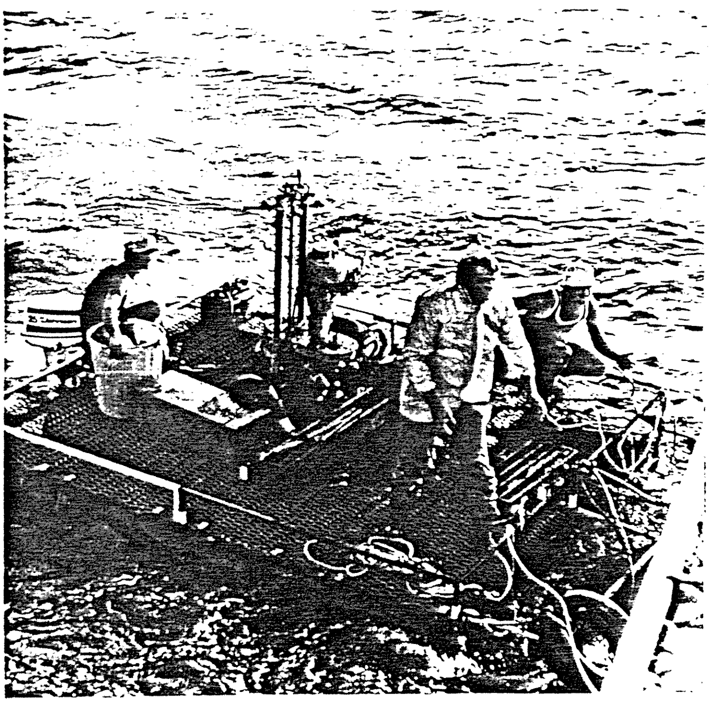


Figure 2 Light drilling rig on self-constructed raft

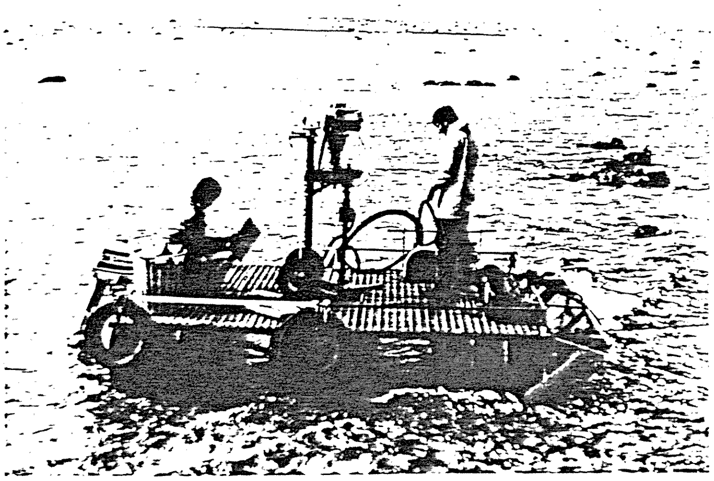


Figure 3 Raft stranded on reef surface at low tide in order to enable diamond drilling from fixed position

4.3 Site Investigation Techniques

Three special site investigation techniques - diamond drilling, dynamic penetration and seismic refraction - have been applied.

Diamond Drilling

The first step in the site investigation programme undertaken was by diamond drilling. Using Atlas Copco's Minuteman Mobile Drilling Machine with a BX-series single- or double-tube core barrel (see appendix) a total of five tests were performed. All drilling tests showed very disappointing results. The core recovery was rather poor (from 0% up to a maximum of 35%), although all drilling tests were carried out on the hard algal rim (ref. Section 2). Furthermore, running sand caused an early jamming of the core barrel between 2.8 m and 6.1 m penetration, which is well below the machine's usual capacity.

In all, only six suitably sized cores were recovered and thoroughly tested in the laboratory. The main results are (Foster 1974; Moss 1976):

Unconfined compressive strength: 17.5 ± 2.6 MPa with peak values of 26.5 MPa maximum and 10.3 MPa minimum;

Young's modulus: 2540 ± 340 MPa;

Porosity: 39-49%.

Measures to improve the core-recovery such as casing the borehole or the application of Rocha's (1971) integral sampling method were discarded, because these methods would have slowed down the drilling operation to a degree which was judged to be unacceptable for a coral reef environment.

During drilling operations it was observed that pockets of sand and substantially sized cavities constituted most of the near-surface parts of the reef. Realizing that diamond drilling is not capable of giving quantitative information on these weak zones and, on the other hand, bearing in mind that it is just these weak zones and not so much the more solid parts of the reef which will normally be critical for a foundation design on coral reefs (ref. Section 3), the scope of this site investigation study was substantially reorientated. The question now was to find a particular technique capable of gaining quantitative information on these weak spots, if necessary accepting a certain loss of information on the more solid coral limestone. As outlined in the following section, a heavy penetrometer is a very suitable instrument for this purpose.

Dynamic Penetration

In planning the dynamic penetration tests, there were two principal questions to be answered:

1. Would it be possible to penetrate through hard layers such as the surface crust of the algal rim?
2. Would the dynamic penetration technique be as sensitive to jamming as diamond drilling?

In order to be better equipped to answer the first question, a number of differently shaped tips were tested on big samples of hard coral limestone before performing in-situ tests on the reef itself. It was found that a 90^{o} cone-shaped tip was suitable for penetrating through even the hardest coral limestone encountered (which was subsequently confirmed by the in-situ tests), whereas the hollow cylinder of the Standard Penetration Test (SPT) proved to be less capable in this respect. Therefore, a 90^{o} cone-shaped driving point of 15 cm^2 cross-sectional area was chosen, which is in agreement with the specifications of the German Standard DIN 4094 (Sheet 1). Referring to the second question raised it was decided, again in line with DIN 4094, to use a slightly bigger driving point than the sounding rods (43.7 mm versus 32 mm diameter). This measure proved successful in avoiding any jamming even at considerable penetration depths.

The machine used was a Heavy Penetrometer, manufactured by Nordmeyer, West Germany (for more details see Appendix 1). Figure 4 presents typical results from one of the 16 dynamic penetration tests performed. (A complete documentation of these tests is given in Billingham 1979.) The following aspects are particularly noteworthy:

- *Considerable penetration depth* of 22.0 m; (the maximum depth reached within the test series was 26.0 m, which still was not at the upper limit of the penetrometer's capability);
- Identification of zones with different penetration resistances, particularly spotting *cavities* (e.g. between -4.4 m and -6.1 m), *pockets of loose sand* (e.g. between -1.6 m and -3.3 m; several layers between -10.5 m and -17.2 m) and hard coral limestone

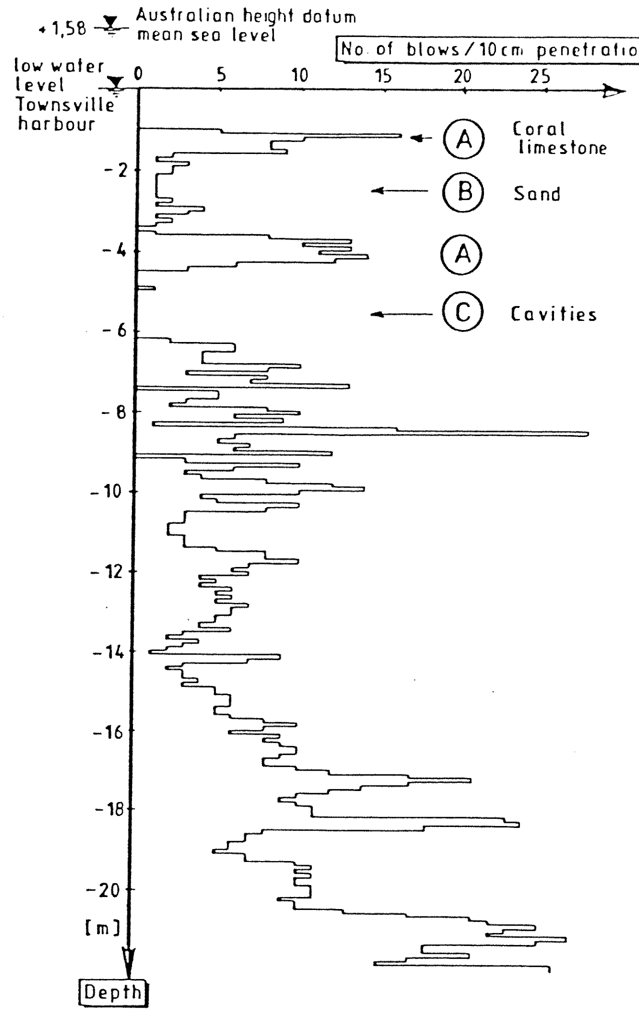


Figure 4 Result of a dynamic penetration test (Penetration Test No. 5). Note that this test disclosed substantially sized cavities and pockets of loose sand

(e.g. at -8.5 m and with increasing frequency from -17.0 m onwards);
- Penetration test from a floating position (1 m water coverage still left at low water datum).

After becoming familiarized with the dynamic penetration technique and the special environmental conditions on coral reefs, one penetration test down to a depth of about 22 m was performed within approximately two hours. Allowing for the time needed to set up and relocate the raft before and after a test, a total of up to three tests could be performed per day. This makes dynamic penetration a relatively economic site investigation technique. As it also gives appropriate quantitative information particularly on weak spots in the underground - which, as outlined before, are the controlling factors of the foundation properties - it becomes obvious that dynamic penetration is a site investigation technique which is capable of coping with the special geo-

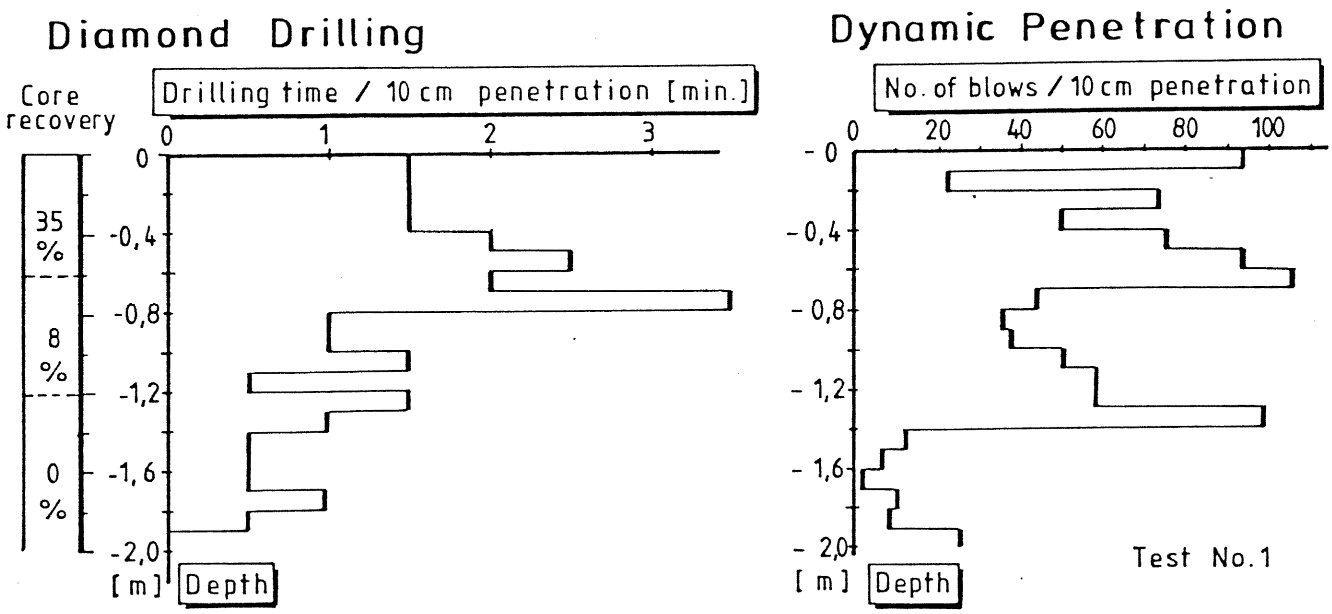


Figure 5 Comparison of the results from diamond drilling and dynamic penetration tests performed at the same location

technical conditions of coral reefs. From a foundation engineering point of view, it appears that on reefs this method is superior to diamond drilling.

Strictly speaking, this statement applies only to the particular drilling machine used; however, it might be extended to any other machine which is not equipped to monitor continuously pertinent drilling parameters such as penetration rate, thrust and torque, and this will be the case for almost any machine presently on the market. Figure 5 suggests that, despite the extremely poor core recovery, a graph of the drilling rate versus the depth at constant thrust gives information which is almost equivalent to that from the dynamic penetration test.

Seismic Refraction

Having found a suitable technique, it was of interest to cross-check the results by use of alternative methods. The most obvious one in this respect is seismic refraction, particularly as this method has recently been applied to The Great Barrier Reef region by Harvey (1977a and b) and Harvey et al. (1978, 1979).

Normally the refraction method is applicable when the stratum to be investigated possesses higher seismic velocity than that of the overlying strata; furthermore, each consecutively deeper layer must possess a certain finite thickness, related to velocity contrasts and depths. Usually, to be detectable, each succeeding layer must be thicker than the one above it. It can therefore be expected that seismic refraction should be able to disclose both geometry and depth of at least the Holocene/Pleistocene interface, as this discontinuity defines the boundary between predominantly unconsolidated Holocene top layers and more consolidated Pleistocene sediments in the deeper underground.

On the other hand, as velocity *reversals* cannot be recorded by seismic refraction (Baule and Dresen 1973), *weak spots* such as cavities and pockets of loose sand underlying harder coral limestone will *not be detectable*. This means a substantial decrease in the general usefulness of the refraction method to the foundation engineer when building on coral reefs. The scope within this study was therefore confined to checking the capability and reliability of the refraction method in detecting major disconformities and comparing its results with those from the dynamic penetration tests.

The seismograph used was a portable, single channel time-distance plotting instrument (HUNTEC, FS-3; details in Appendix 1). In the field a particular methodology as outlined in detail in Harvey (1977b) was applied, which takes into account the special intertidal environment of coral reefs.

It was found that a successful use of this seismograph on coral reefs depends very sensitively on the actual sea conditions. Even a minimal surf at the reef edge constitutes a considerable ambient noise level so that seismic signals, triggered either by hammer blow or small detonation, became obscured in about 50 m distance from the seismic source. This limited the investigation depth substantially to about 10 m. As indicated in Figure 8, refraction horizons were spotted in about 6 m depth, which correlate well with some layers of increased resistance as detected in the dynamic penetration tests, particularly in the region of the reef front (penetration tests 1 and 15). The seismic velocities for most of the top layers is about 1.6 km/sec, which is in accordance with Harvey's (1977a and b) experience, however substantially increased (2.1 km/sec) in the algal rim. The subsurface velocity averages 2.2 km/sec.

5 APPLICATION

The in-situ tests have been performed on Keeper Reef, approximately 60 km northeast of Townsville (Figs. 6 and 7). This is in Maxwell's (1968:106) terminology an "open ring and composite apron reef" belonging to the family of "shelf reefs".

As indicated in Figure 6(c) Keeper Reef has been tested approximately alongside a section running from the outer reef front into the lagoon. Most tests were done by dynamic penetration. Typical results of these tests are synoptically presented in Figure 8. It can be realized that in both vertical *and* horizontal directions there are some significant changes in the near-surface structure of the reef. The most remarkable features are as follows.

1. In the lagoon (left in Fig. 8) a systematic increase in the penetration resistance is evident at a depth of about -16 m. It is of interest that investigations in other parts of The Great Barrier Reef (Davies 1974; Davies et al. 1977; Hopley et al. 1978) have all disclosed a major discontinuity at similar depths. Most recently, radiocarbon tests on samples from Bewick, Hayman and Heron Islands (Thom et al. 1978; Hopley et al. 1978; Davies and Marshall 1979) have verified the idea that this discontinuity represents the Holocene/Pleistocene interface. This should provide ample evidence to justify interpreting the subsurface structure on Keeper Reef in the same way.

2. There are some indications (ref. penetration tests 8 and 2) that the pre-Holocene unconformity may rise from the central part of the reef towards the algal rim and reef margin. Similar tendencies have been observed by Harvey et al. (1978) in the Capricorn and Bunker Groups of The Great Barrier Reef. However, with respect to the situation on Keeper Reef the authors would like to have some more test results available before commenting on this question in more detail.

3. Enough data, on the other hand, are available to state that the lowest 2 to 3 m of the Holocene sediment are characterized by a particularly low penetration resistance, as this has been disclosed in almost every penetration test. This indicates that the Holocene base consists of unconsolidated sand of low relative density.

4. The upper Holocene layers are subject to quite significant variations in their geotechnical properties. In the lagoon weak layers, consisting of coral sand, extremely porous coral limestone and cavities, are dominant. Towards the reef front higher strength limestone varieties become more prominent and cavities, if any, are less numerous and smaller in size. This holds true particularly for the algal rim, where some layers of extreme penetration resistance and increased seismic velocity were encountered (penetration test No. 1, ref. Figs. 5 and 8).

6 CONCLUSIONS

Comparison between the three different site investigation methods applied indicates that, from a foundation engineer's point of view, dynamic penetration is a method which on coral reefs is superior to both diamond drilling and seismic refraction, as it accurately indicates the presence of problem areas such as cavities and loose sand. Penetration tests on reefs should adopt some specifications of the German Standard DIN 4094, as cone-shaped driving points are more capable of penetrating through hard coral limestone than hollow cylinder ends as used in the SPT. Furthermore, different diameters of sounding rods (32 mm) and driving points (43.7 mm) contribute significantly in avoiding jamming. Diamond drilling could only be considered an alternative if important drilling parameters such as penetration rate, thrust and torque could be controlled and continuously monitored in order to obtain truly quantitative information on weak spots. Seismic refraction may be a useful tool for the disclosure of some more general subsurface structures; however, it is not possible to investigate thoroughly small areas for their pertinent foundation properties by exclusive use of this method.

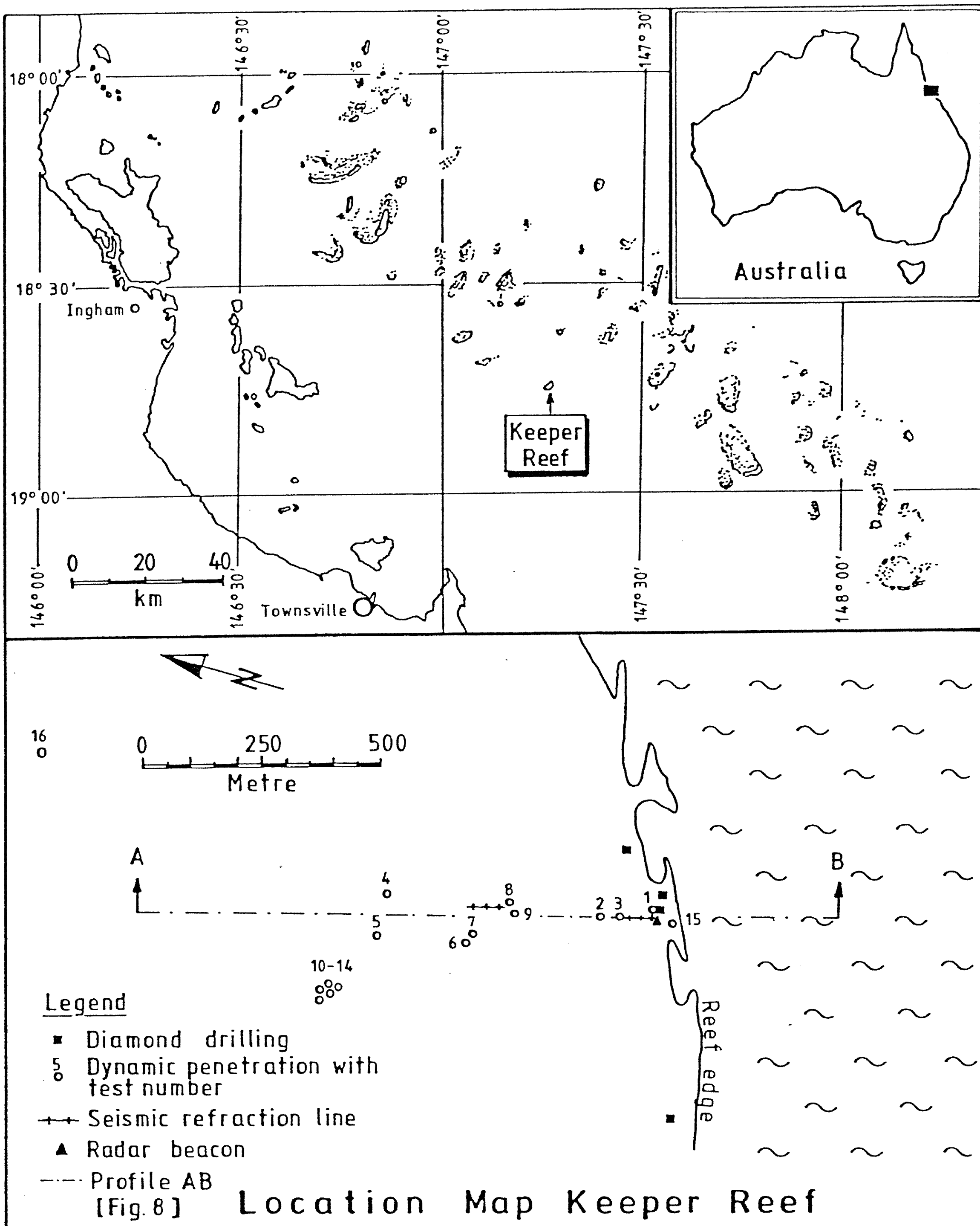


Figure 6 (above) Map of Keeper Reef with location of the different site investigation tests performed.

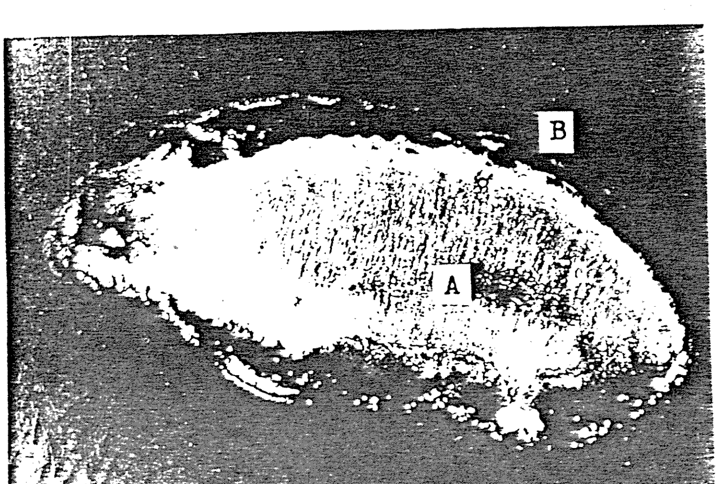


Figure 7 (left) Aerial view of Keeper Reef with cross section AB as indicated in Figure 6.

Depending on the position within a coral reef, significantly different foundation properties can be expected. The outer reef area, particularly the algal rim, is characterized by relatively sound foundation properties, although even here cavities and loose sand still may occur. Without doubt this area would be most suitable for a foundation on coral reefs. However, it is often associated with some other problems as this site is particularly exposed to waves and not easily accessible by boat. Therefore, most major engineered structures will be erected in the more inner, sheltered parts of the reef (as in the case of the large research platform on Britomart Reef north of Townsville, which is presently planned by the Australian Institute of Marine Science), although these lagoonal areas tend to have very poor foundation properties with coral sand, porous limeatone and substantially sized cavities. Where it is proposed to utilize piles for a foundation in this area, and this will be done in the majority of cases, tests should be carried out at *each individual pile* position, since in the lagoon the variability in extent, thickness and competence of coral strata within the overall mass can be extreme.

ACKNOWLEDGEMENTS

The authors wish to thank Mr. Rod Matheson, Technical Officer, for his notable and dedicated technical work, without which this project would not have been launched. Dr. D. Hopley (Geography Department, James Cook University, Townsville) supported the project by discussions and by generously making the seismograph available to us. Funding was from The Great Barrier Reef Marine Park Authority, Townsville (Grants No. 9.5.3.1 and No. 9.5.3) and from various University grants, all of which are gratefully acknowledged.

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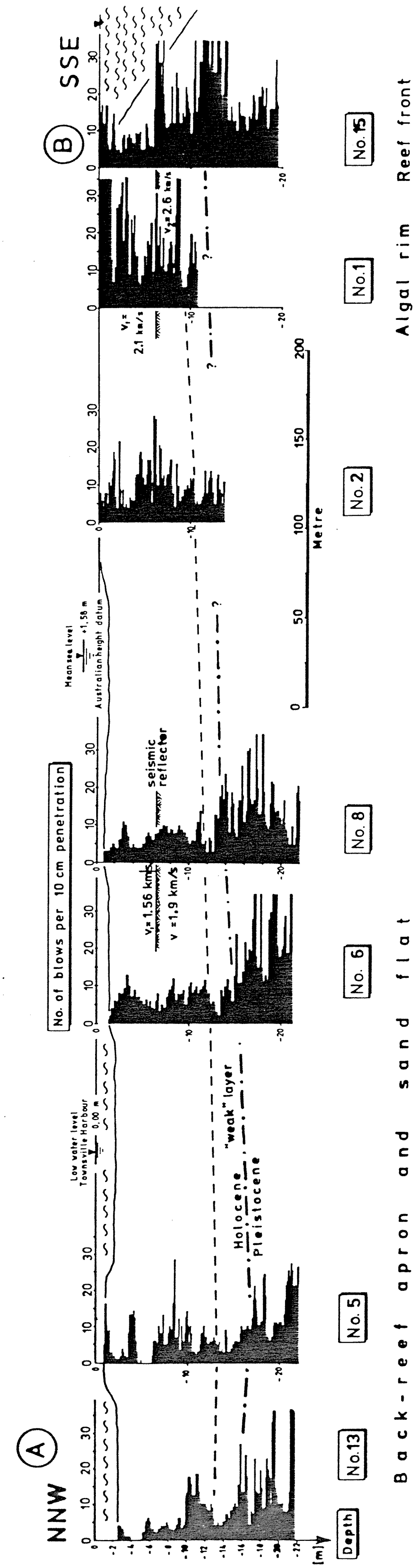

Figure 8 Near-surface structure of Keeper Reef from the reef front (right) to the sandy flats of the lagoon (left) with interpretation. (Notice = Higher than 35 blows per 10 cm penetration are not graphed)

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APPENDIX 1: TECHNICAL INFORMATION ON EQUIPMENT USED

Vessel:
James Cook University's Research Vessel "James Kirby". Overall length: 17.2 m
Engine: Mercedes Diesel 132 kW at 2000 RPM
Max.speed: 9.5 knots

Raft:
Self constructed, consisting of a square-steel framework (50 x 50 R.H.S.) with 10 to 14 200-litre fuel drums supplying flotation.
Working area on the raft's platform: 3.00 x 3.70 m
Draught, when fully loaded: 48 cm
Outboard motor: 18.6 kW
Anchor: Reef picks
Anchor cables: 5 mm ϕ galvanized steel cable
Remarks: A sheetmetal cowling on the front of the raft facilitates towing behind research vessel.

Drilling Machine:
Atlas Copco 'Minuteman' Mobile Drilling Machine, model M/M, manufactured under licence by Fox Manufacturing Company.
Max.power: 3.7 KW at 3600 RPM
Max.torque: 10.4 N.m at 3000 RPM

Heavy Penetrometer:
Manufactured by W. Nordmeyer D3150 Peine/ W.-Germany
Total weight: 880 kg
Drop weight: 50 kg
Height of fall: 50 cm
Sounding rods: 32 mm diameter, 1 m long, engraved with 10 cm marks
Driving points: 90° conical-shaped tips of hardened steel, 43.7 mm diameter (=15 cm^2 cross sectional area), pinned (not screwed) to the sounding rods in order to lose the tip when recovering the rods ("lost-point" approach)
Motor: air-cooled Diesel engine, 3 kW at 2500 RPM

Refraction Seismograph:
FS-3 portable Facsimile Seismograph, manufactured by Huntec Ltd., Scarborough/ Canada. This is a single channel time-distance plotting instrument which permanently records an entire seismic event produced either by a hammer blow to the surface of the ground, or by an electrically detonated explosive charge. The record, produced on electro-sensitive paper, is in the form of short dashes which signify the positive zero crossings of each cycle of the shock waves (wavelength) initiated by the hammer or explosive charge.
Geophones: Hall Sears HS-J, Model L1, Velocity Sensitive, Coil Resistance - 280 ohms, Natural Frequency - 14 Hz.
Amplifiers Dual Channel: Input Impedance - 700 ohms, Frequency Response.
Gain Control: Adjustable attenuator; 6 db steps from 0 - -66 db.
Printing Sensitivity: 2 micro-volts peak to peak with attenuator control at 0 db.
Time Base: 3 to 180 milliseconds (Normal), 163 to 340 milli-seconds (Delayed), Accuracy - ±1%, Size: 45 x 35 x 15 cm.

Appendix 2

BILLINGHAM, S.L.:

"The Study of Site Investigation Techniques for determining the Engineering Properties of Coral Reefs".

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CHAPTER 1 - INTRODUCTION

The majority of the substantial civil engineering structures (tourist resorts, research and weather stations) on coral reefs along the Queensland coastline, have been constructed on reefs and cays, which are exposed at high tide. As the number of these available islands are gradually becoming exhausted, there is a growing interest from community sectors into the possibility of erecting structures on bare coral reefs. Enterprises which may utilise these reefs, are the tourist and fishing industries, as well as research organizations. Although a very political issue, bare coral reefs may also be used by different mining organizations (e.g. oil companies etc.) in connection with future exploration.

The framework of bare coral reefs, range from well developed reef tidelands to fully submerged coral outcrops.

Previously designed structures for bare coral reefs (that is lighthouses and weather stations) have not been structurally disadvantaged from an economical point of view. Thus, little has been learnt of the geomechanic properties of coral reefs from their construction. This is due to the loss against cost benefit that would arise, in case of a failure.

As no commercial undertakings have been developed on bare coral reefs, it has not been warranted to produce a more rational approach to foundation designs. To develop feasible foundation design criteria, the engineering properties of coral reef limestone would first have to be investigated.

From previous research (Smith, 1972, Moss 1976) a substantial amount of material has been gathered on the geomechanical properties of coral limestone. However, sand, holes and very weak coral are more critical to the behaviour of foundations than the overall

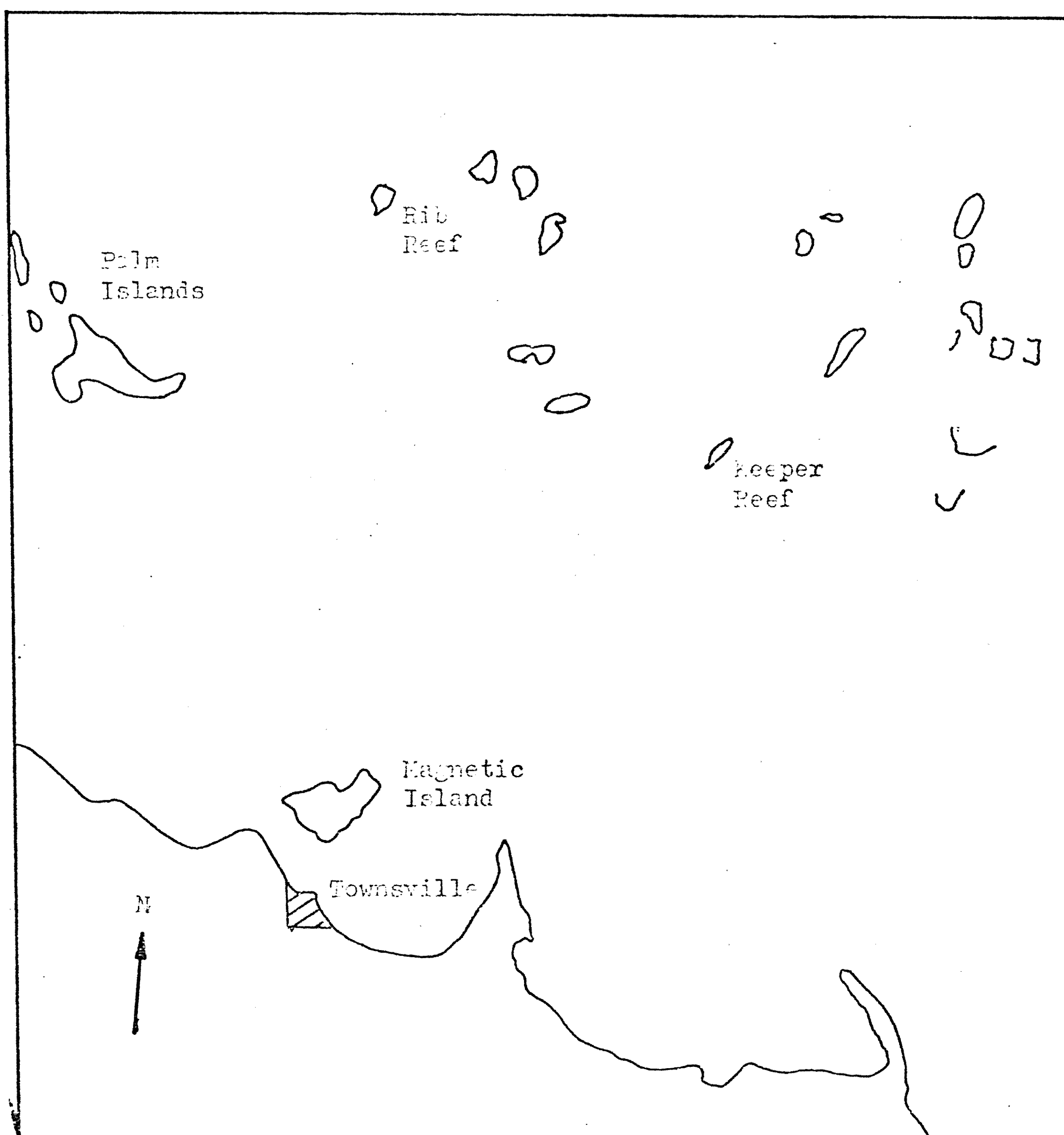


Figure 1.

Location Map

Scale

geomechanical properties of coral limestone. Thus, it is endeavoured to increase the available knowledge of the engineering properties of coral reefs to depths of 25 metres. As most foundations would not have any effect on the coral at a depth of 25 metres, investigations have been limited to depths shallower than 25m. To determine the basic geomechanic units at such depths, drilling, sounding and geophysical techniques are a few methods that can be used.

It is the aim of this thesis, to study the feasibility of different site investigation techniques, in determining the engineering properties of coral reefs, to a depth of 25 metres. As a secondary objective, testing will be performed along a transect, so collected data may be used to determine the geology of the reef along this section. This is to confirm the genesis of the reef, so favourable construction sites on the reefs may be ascertained in the future.

The study was undertaken on Keeper Reef, which is a bare coral reef, located 60 kilometres north-east of Townsville. The position of the reef is shown in Figure 1.

CHAPTER 2 - LITERATURE REVIEW

Within the last decade, various research projects have studied the characteristics of coral reefs, throughout the Great Barrier Reef province. As little was known of the origin and structure of the reefs, research into the geology and engineering properties evolved.

Research carried out by the Engineering, Geography and Geology departments of J.C.U.N.Q., have collected data on the structure and development of reefs, as well as the mechanical properties of coral materials. A brief outline of some of the various research programs undertaken by these departments during the previous seven years, are listed below.

P.T. SMITH (1972: Engineering)

Conducted standard tests (Uniaxial compression and Indirect tension tests) on samples of coral rock, to enable the classification of the rock under existing engineering classification systems. Results allowed only an approximate classification, as large variations in the strength of samples occurred. These were not accounted for, as little was known of the basic structure of different types of coral.

D.F. FOSTER (1974: Engineering)

Classified coral rock according to existing rock classification schemes and found the scheme proposed by Stapleton (1968) best suited for coral limestone. This scheme is based on the unconfined compressive strength of dry samples. All samples tested, were obtained from depths of only 2-3m.

P.A. CAMERON (1975: Engineering)

Constructed a radar beacon (approximately 7m high) for navigational purposes and designed a marine platform for use on reefs. Very little information was obtained of the engineering properties of the coral limestone from the construction. However, some experience was gained of the difficulties involved in working in a marine environment while erecting the beacon.

K.H. MOSS (1976: Engineering)

Performed research into ways of understanding the behaviour of the coral rock mass and associated materials with reference to engineering problems. Several in-situ bearing capacity tests were carried out and the behaviour of marine carbonate sands when used in concrete was investigated. Core drilling operations were performed to obtain core samples as well as evaluating core drilling operation as a site investigation technique.

N. HARVEY (1977: Geography)

Carried out seismic refraction surveys using a Huntec FS-3 portable seismograph, to investigate the presence of subsurface solution disconformities beneath reefs. The studies revealed a marked seismic discontinuity, which was interpreted as representing a solution disconformity ranging in depth from 6.3m to 19.3m, for seven reefs between $14^{o}S$ and $17^{o}S$.

A.I.M.S. (SEPT. 1979: Geology)

A drilling test was performed on Britomart Reef to a depth of 70m, to determine the geology of a proposed site for the erection of a research platform. The test returned a wide range of core recovery per metre (3-80%), of which very little was suitable for laboratory tests. As a high percentage of fine material (carbonate sand) was not collected with the core recovery, no estimations of the insitu strength of the coral can be obtained.

From the research performed, a substantial amount of information has been collected on the geomechanical properties of coral materials and the geology of coral reefs. However, very little information has been gathered on the engineering properties of coral reefs for any great depth. Although drilling tests have been performed, the amount of data that may be obtained of the engineering properties of the subsurface material, is negligible.

Thus a study of various site investigation techniques for determining the engineering properties of coral reefs, has been carried out. The basic objective of the research project, is to widen the knowledge of the engineering properties of coral reefs to a depth of approximately 20m. It is concerned with the distribution and mechanical properties of the different geomechanic units such as coral limestone, sand or holes which make up the underground of reefs.

An outline of the research project is given in the following chapters of the thesis and the basic steps undertaken to achieve the research objectives are listed below.

1) Onshore tests were performed to become familiar with the mechanics of the testing equipment.

2) Results from the onshore tests were used to calibrate the various equipment, allowing future comparison with on-site test results.

3)a. Extensive testing was performed on site (Keeper Reef) with each technique.

3)b. Test results and testing procedures were studied to ascertain the feasibility of each technique.

4) As a secondary objective, the test results were used to determine the geology of a section of reef, that had been tested.

CHAPTER 3 - EXPERIMENTAL EQUIPMENT AND PRINCIPLES

3.1 INTRODUCTION

Although there are many site investigation techniques that may be used to determine the basic geomechanical units of the coral, only three were examined in any detail on the reef.

The research work involved the use of a small drilling machine, a dynamic heavy penetrometer and a portable facsimile seismograph. Although all three methods were studied, the penetrometer was used extensively throughout the program, as drilling and seismic tests were limited by technical problems and time. The type and basic principles of each machine has been outlined below.

3.2 DRILLING MACHINE

The rotary drilling machine and associated equipment used for core recovery operations on Keeper Reef are shown in Figure 3.1 in an operational state. The drilling and core recovery operation, incorporated the use of a Minuteman Mobile Drilling Machine, Model M/M manufactured by Fox Manufacturing Company.

Rotary drilling machines are generally used for recovering core samples of highly resistant materials. A rotating drill bit cuts and grinds the material at the base of the hole into very fine particles. Water which is continuously circulated through the hollow drill rods, washes the ground material from the cutting face to the surface of the bore hole. This water is pumped into the drill rods via a swivel coupling situated at the base of the drilling machine.

As drilling progresses, a cylindrical sample is cut out of the underground. The greater part of the operational time is consumed with the recovery of the core barrel, after sufficient penetration. The core sample is then removed and the procedure repeated.

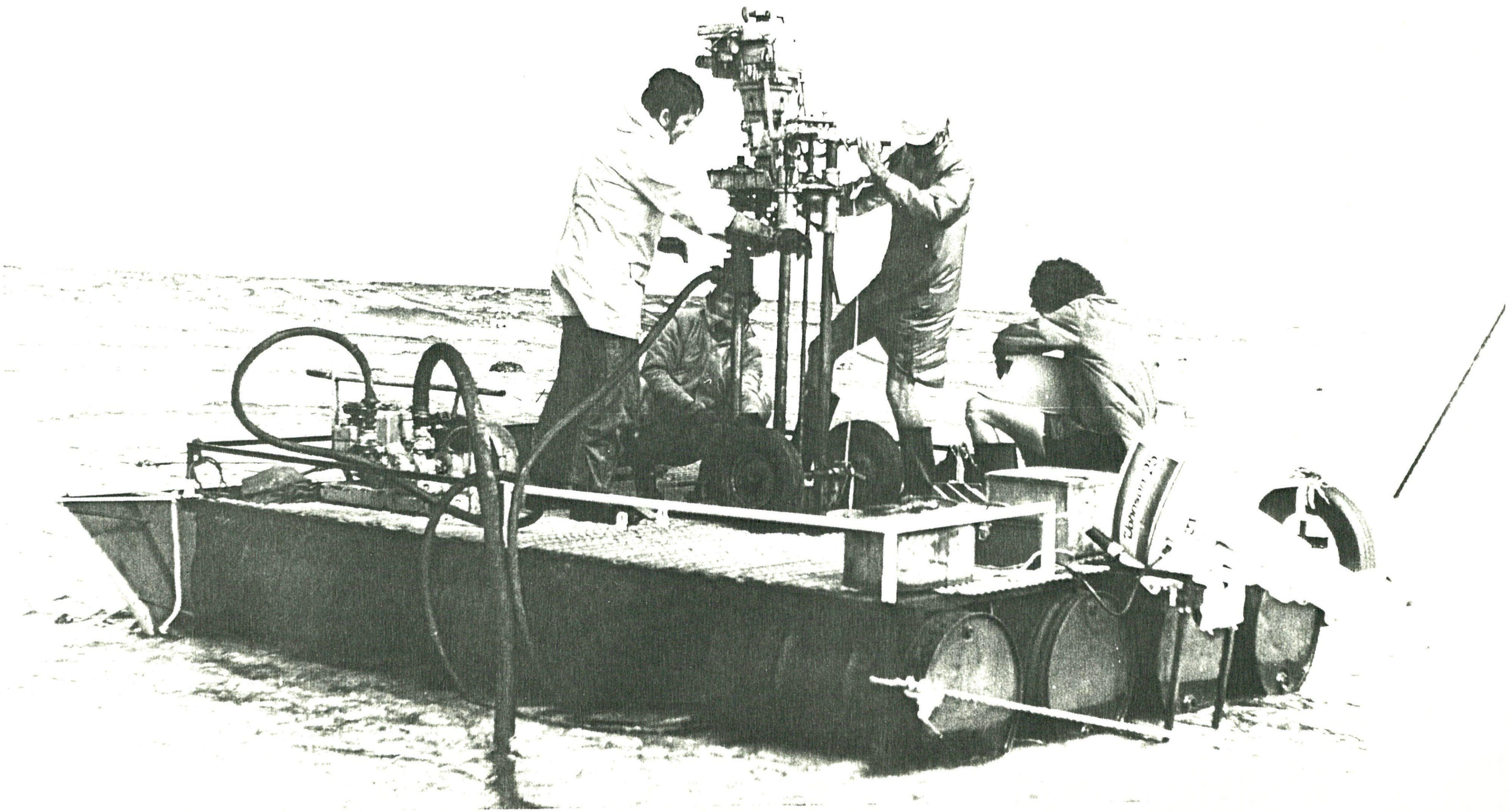


FIGURE 3.1

Single tube core barrels can be used, but a smaller percentage of the finer material is collected, as the circulating water is passed through the core samples. The percentage of recovery may be improved if a triple tube core barrel is used, as this type of barrel prevents the circulating water from being washed through the core recovery.

Laboratory tests can then be performed, (uniaxial compression test, direct shear test etc.) on the core samples recovered, to determine the geomechanical properties of the material. By determining the properties at different depths, some indication of the coral structure can be obtained.

3.3 PENETROMETER

A Nordmeyer dynamic heavy penetrometer was used for all penetration tests performed. The 880 kg self-contained instrument is powered by a 4 HP Farremann single cylinder air cooled diesel engine. With the mast in a lowered position, the unit has the approximate overall dimensions of 3600, 1600 and 1400 mm.

A percussion mechanism is mounted on the mast, and is free to travel downwards as penetration progresses. The unit is rested on the anvil screwed to the top of the 32 mm diameter sounding rods. The mechanism is activated by a series of catches on an endless chain. Each catch lifts the 50 kg drop weight to the correct height of 50cm before releasing it.

Thus, as testing proceeds the sounding rods are hammered into the subsurface with a constant energy per blow. Therefore, the relative hardness of subsurface formations, are directly proportional to the number of blows required to penetrate a given distance.

Measuring grooves machined into the rods at 10cm intervals, enables the number of blows required to penetrate 10cm to be recorded. The blows produced are counted by a simple blow counter tripped by the catches on the endless chain.

Depending upon the hardness or depth of a deposit to be penetrated, a lost point or recoverable tip may be used. The difference between the tips, is that one is secured by a shear pin and the other screwed onto the sounding rods respectively.

Non recoverable tips are used where difficulties may occur in recovering the sounding rods, due to the hardness of a medium being penetrated. By applying an upward force to the rods, the shear pin is severed leaving the tip behind. As the outside diameter of the tip (43.7 mm) is larger than that of the sounding rods (32 mm), no difficulties occur in recovering the rods.

To recover the sounding rods, the percussion mechanism is connected to the anvil, and winched to the top of the mast. By removing one rod, the anvil can be screwed to the next rod and the procedure repeated. The rods are prevented from falling by a special ball clamp, which only allows upward movement of the rods to occur.

In the United States, the most widespread test performed using a penetrometer, is known as the standard penetration test. A hammer weighing 62.5 kg is dropped from a height of 76.2 cm onto drill rods which have the same outside diameter as the penetration tip. For the German penetrometer, the drop weight and fall of height is different and the penetration tip has a larger diameter than the sounding rods. This gives a more accurate value of the penetration resistance at the tip, as less friction is developed along the sounding rods.

By determining the penetration resistance of the subsurface with depth, a complete profile of the materials relative hardness at the test site, can be ascertained. From a plot of the penetration resistance against depth, the engineering properties and geology of the site can be interpreted.

3.4 SEISMOGRAPH

A single channel portable Huntec FS-3 facsimile seismograph was used to accomplish refraction surveys on Keeper Reef. The instrument permanently records on a time-distance plot, the entire seismic event initiated by either a hammer blow or an electrically detonated explosive charge.

The seismic wave train received by the geophones, is plotted on electro-sensitive paper in the form of short two millesecond dashes. Each dash represents the positive zero crossing of the shock wave, as shown in Figure 3.2. Recorded events are plotted over a 3 to 180 millesecond time base, with a specified accuracy of $\pm$ 1%. The time distance curve is produced by manually advancing the paper for each seismic event. An example of a time-distance curve is shown in Figure 3.3.

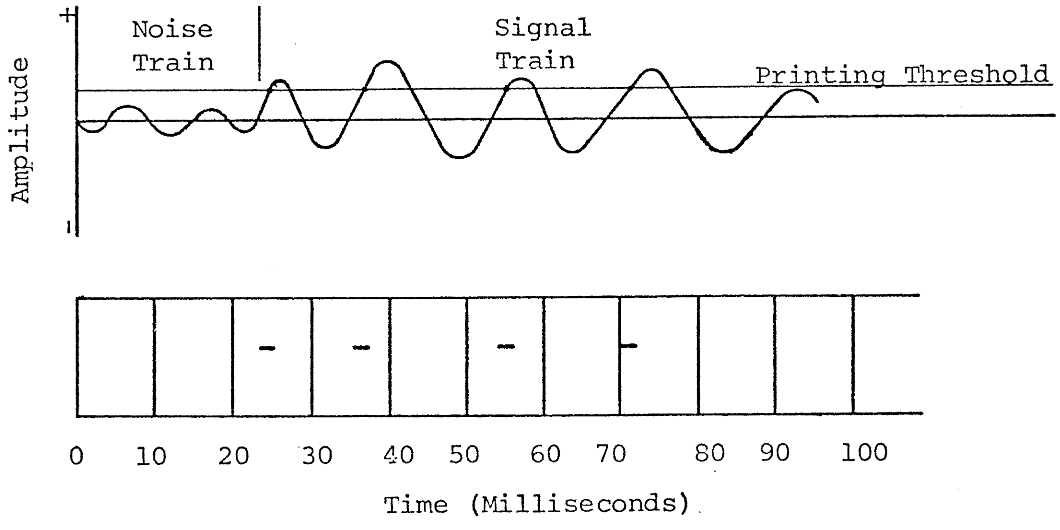
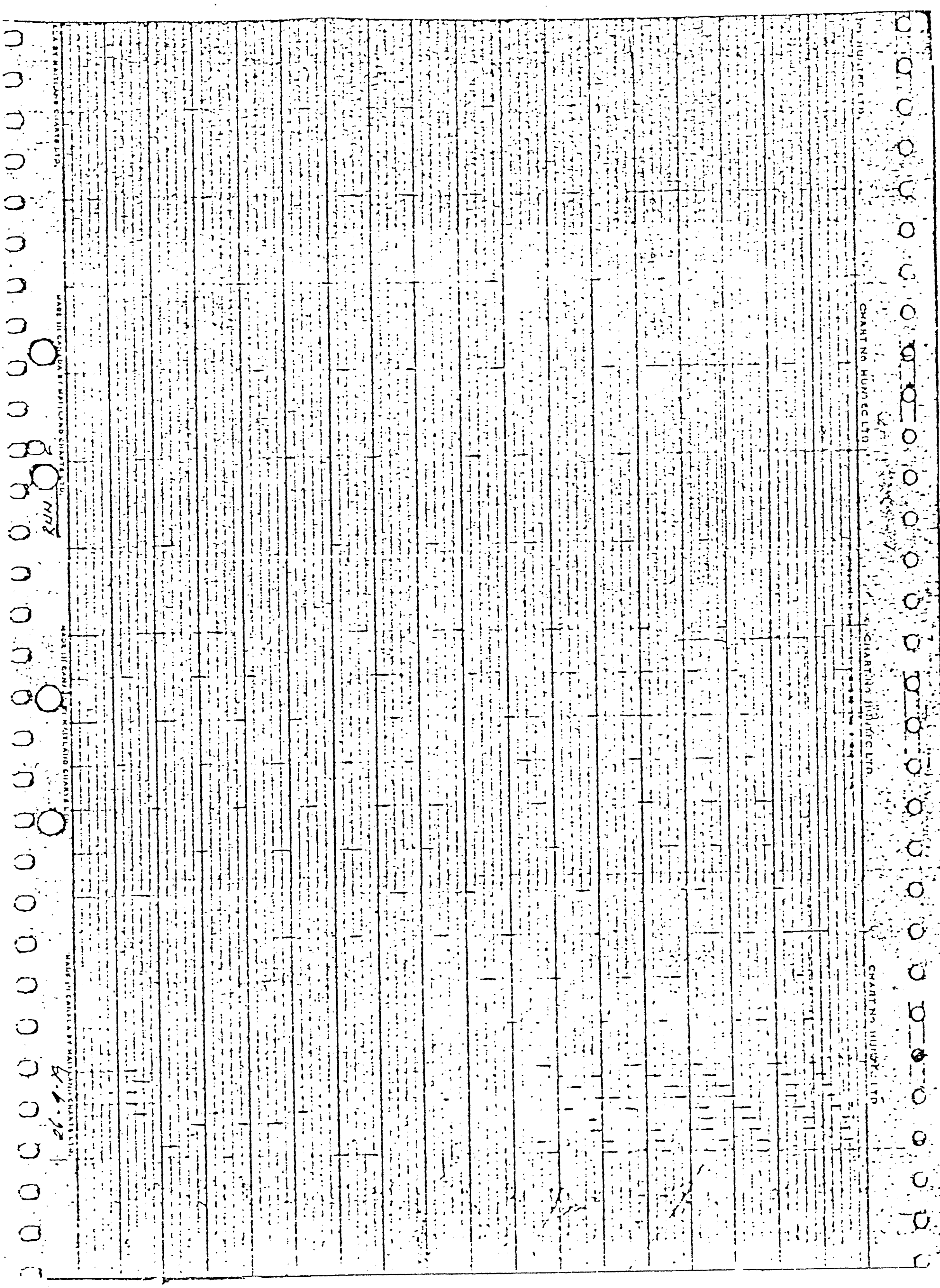


FIGURE 3.2

For a stratum to be detected by refraction, the distance between the shot point and the detector, has to be several times the depth of the layer. However, as the distance increases, the intensity of the signal detected by the geophones decreases. Over long distances, seismic detonators (manufactured by ICI Australia) can be used instead of a 4.5 kg hammer, due to the greater resolution

FIGURE 3.3



yielded. The use of a hammer is further limited by the high level of ambient noise.

Because of the instruments size (450 x 300 x 150 mm) and light weight (14.5 kg with NiCad batteries) it has an advantage of being easily handled around small boats. The Ni-Cad rechargeable batteries, also eliminated the need for replacement batteries as 240 V power was available on the mother ship.

As the seismograph had been designed for land based operations, a number of technical problems arose, when it was initially used in an aquatic environment. Such problems and how they were overcome are discussed in Appendix A.

3.4.1 Theory

The seismic method depends fundamentally on the propagation of seismic waves in an elastic media. The principles governing the propagation of seismic waves are similar to those that govern the propagation of light waves. Thus the laws of reflection and refraction are applicable to seismic studies.

Where no information is available on the subsurface geology, reflection surveys give little information on the compaction of the underlying media. Reflection surveys are directed towards determining the geometry of subsurface formations. However, with refraction, seismic velocities (degree of relative hardness) as well as the geometry can be obtained.

The mechanism of refracted waves in a two layered subsurface is shown in Figure 3.4. The upper layer has a velocity of V_o and the lower stratum that of V_1. For the lower layer to be detected the velocity of the near surface layer must be less than that of the lower stratum.

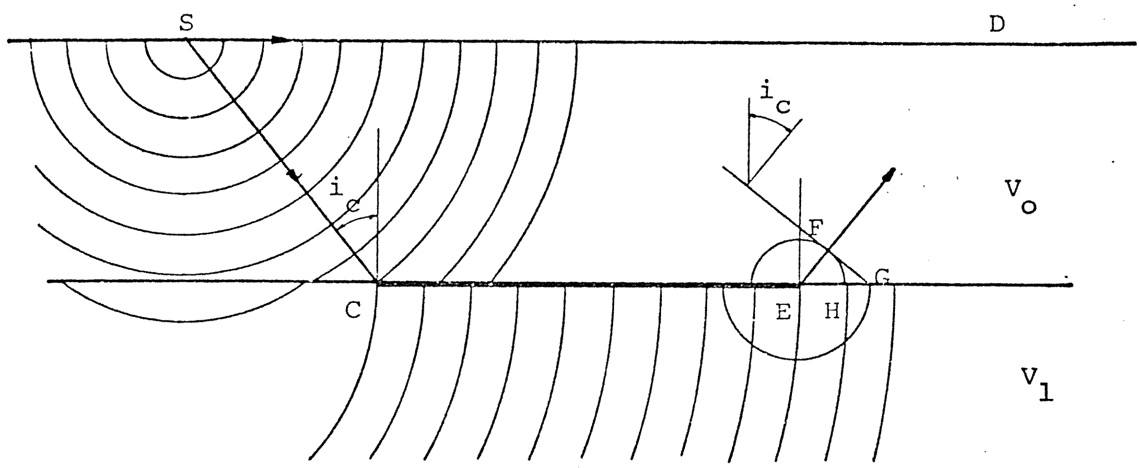
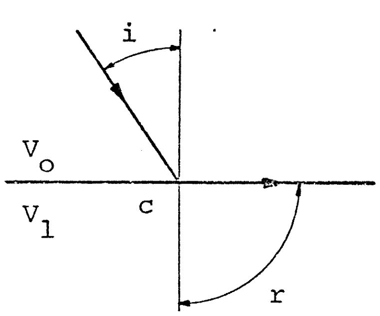


FIGURE 3.4

From an energy source S, generated seismic waves travel in a hemispherical wavefront centred around the source. On striking the interface, the rays are refracted into the lower layer, with a corresponding velocity change. The velocity and angle change of the rays, is governed by Snells Law. That is:-

$$\frac{\sin i}{\sin r} = \frac{V_o}{V_1}$$



At a particular point C on the wavefront, the refracted ray will travel along the interface, with the velocity of the lower layer. Thus the angle that ray SC makes with the boundary is the critical angle. As this wavefront travels along the interface, new disturbances are generated due to the oscillating stresses in the upper medium. These new disturbances spread out spherically in the upper and lower media with velocities V_o and V_1 respectively. This is shown at point E. In the lower medium the wave will travel a distance EG compared with EH in the upper medium for the same time.

It can be shown, the new wavefront FG, will return to the surface at a critical angle with the perpendicular to the interface.

$$\text{Sin } i_c = \frac{EF}{EG} = \frac{V_o \times t}{V_1 \times t} = \frac{V_o}{V_1}$$

The ray that first arrives at the detector D, will depend upon the distance D is away from S. If the distance SD is small, the surface ray will arrive first, due to the extra distance that the refracted ray has to travel. However, as SD increases, the refracted wave arrives first due to the time saved in travelling through the faster medium.

Therefore, by plotting the first arrival times against the short-detector distance, a useful curve of the refraction data can be obtained. The velocity of each layer, can be derived from the slope of the curve, corresponding to each layer. Either a Critical Distance or Intercept-Time Method can be used to calculate the thickness of each stratum. The accuracy of interpretation is in the order of 10%, which can be reduced if reverse spreads are performed.

Thus, as a direct relationship exists between the hardness and seismic velocity of a material, a variety of engineering problems, can be solved using refraction techniques.

CHAPTER 4 - PRELIMINARY TESTING

4.1 INTRODUCTION

Prior to using the penetrometer and seismograph on site (Keeper Reef), several onshore tests were performed. These tests allowed an insight into the mechanics of the equipment to be obtained and minor testing techniques and technical problems to be overcome before site testing. The results of the tests were used to calibrate the equipment, so as the site investigation results could be better appreciated.

By knowing the penetration resistance of different geological materials from onshore tests, the soundness of the sub-surface on site can be visualized by comparing results. Similarly, the hardness of the materials can be related to their seismic velocities and a comparative firmness of the formations on site can be determined.

4.2 ONSHORE TESTING

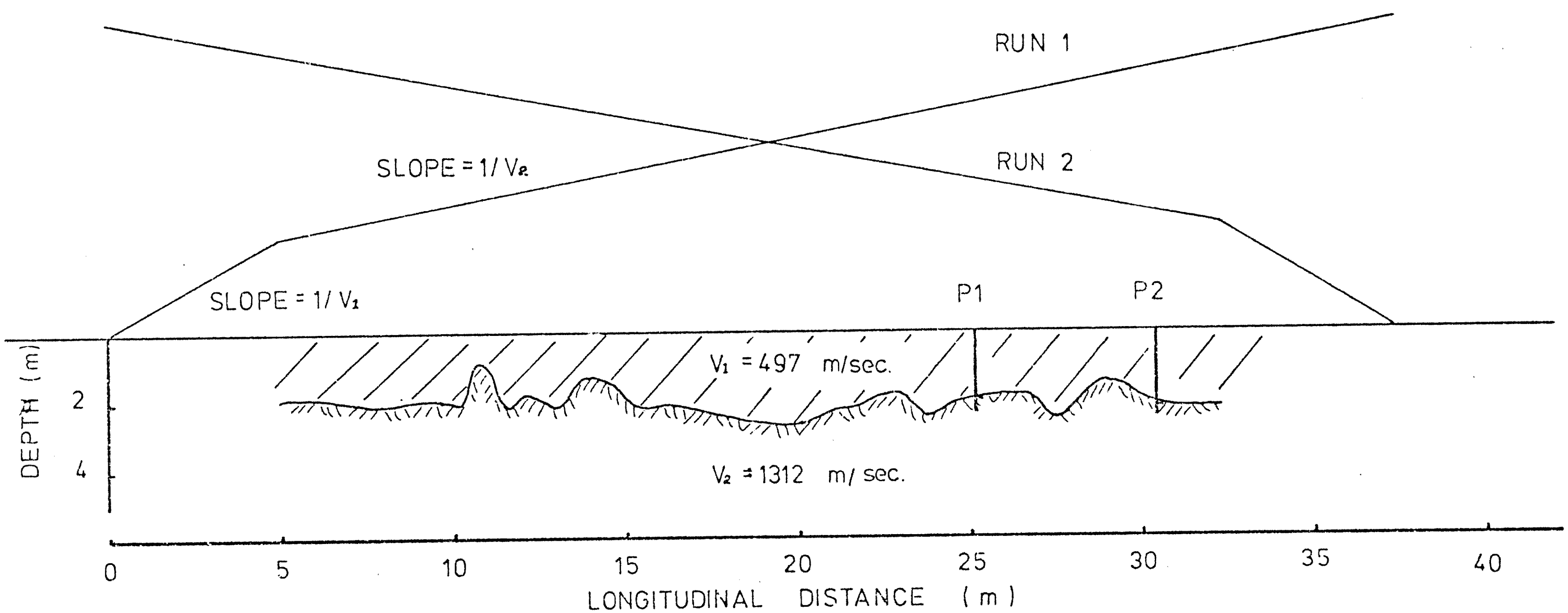
Several seismic spreads and penetration tests were performed at Douglas Campus and Pallarenda. The seismic tests were performed in April and May and two penetration tests also in May.

4.3 SEISMOGRAPH - DOUGLAS CAMPUS

The southern end of Douglas Campus was chosen for testing as the location is situated at the base of Mt. Stuart. It was anticipated that shallow subsurface formations would exist below the topsoil, due to the adjacent hill slopes. The topsoil consisted of a fine grained material with a small percentage of rocks. Deeper layers would most likely contain a much higher percentage of rocks or weathered granite.

From the initial seismic test (April), a shallow layer was detected at an approximate depth of 2.1m. From further tests

Profile of Seismic Penetration Tests at Douglas Campus. FIGURE 4.1



performed (May), the interface between the two top layers, was profiled for a distance of 30m. To produce the profile, reverse spreads were done, and Hawkins (1961) Method was used to plot the profile from the back and forth shots.

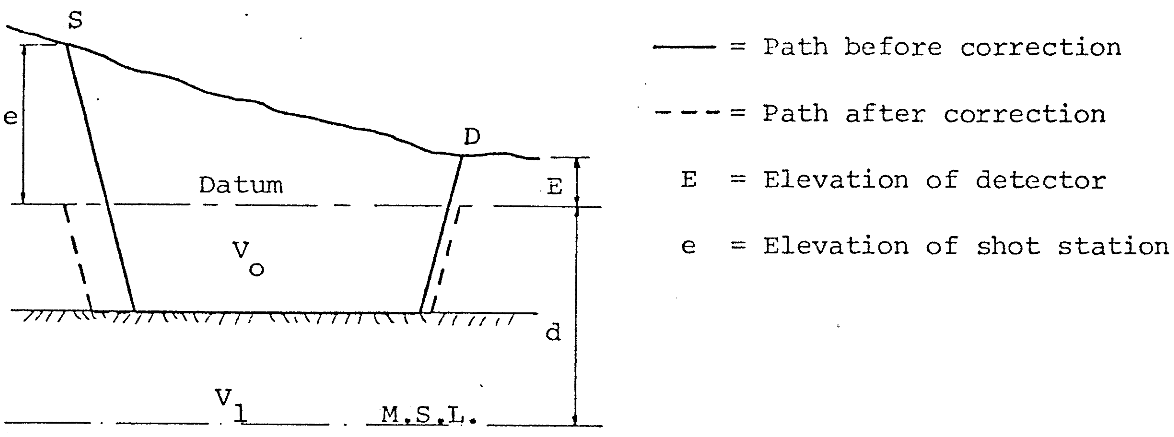
The results of the survey are shown in Figure 4.1, from which the subsurface formation is situated at an average depth of 1.96m within a 0.8 to 2.68m range. The average seismic velocity of the near surface material is V_1 = 500m/sec and the lower stratum V_2 = 1312m/sec.

However, results from the (April) tests, determine the seismic velocities V_1 and V_2 as 544 and 1290m/sec respectively, which vary from the values determined in May. These differences, may be accounted for by the error that occurs in interpreting the results and the variation in moisture content of the test site between tests. Both could account for the differences that occurred in the seismic velocities. The raw data of both series of tests, April and May, are shown in Appendix B.

4.4 SEISMOGRAPH - PALLARENDA

The tests at Pallarenda, were performed at the base of a hill, in an attempt to locate the bedrock beneath the topsoil of loose sand. Due to the uneven topography of the area, a profile of the lower strata has not been plotted. The effect on the profile, from the variation of elevation, would probably be greater than the actual irregularities of the interface.

The curve can be corrected for elevation, as set out in Dobrin, but such an exercise was unwarranted as only the average seismic velocities were required for calibration purposes. However, the method of correction has been detailed below.



$$\text{CORRECTION} = \frac{(e + E - 2d)\sqrt{V_1^2 - V_o^2}}{V_1 V_o} \text{ sec.}$$

The new correction is subtracted from the arrival time of each new shot point.

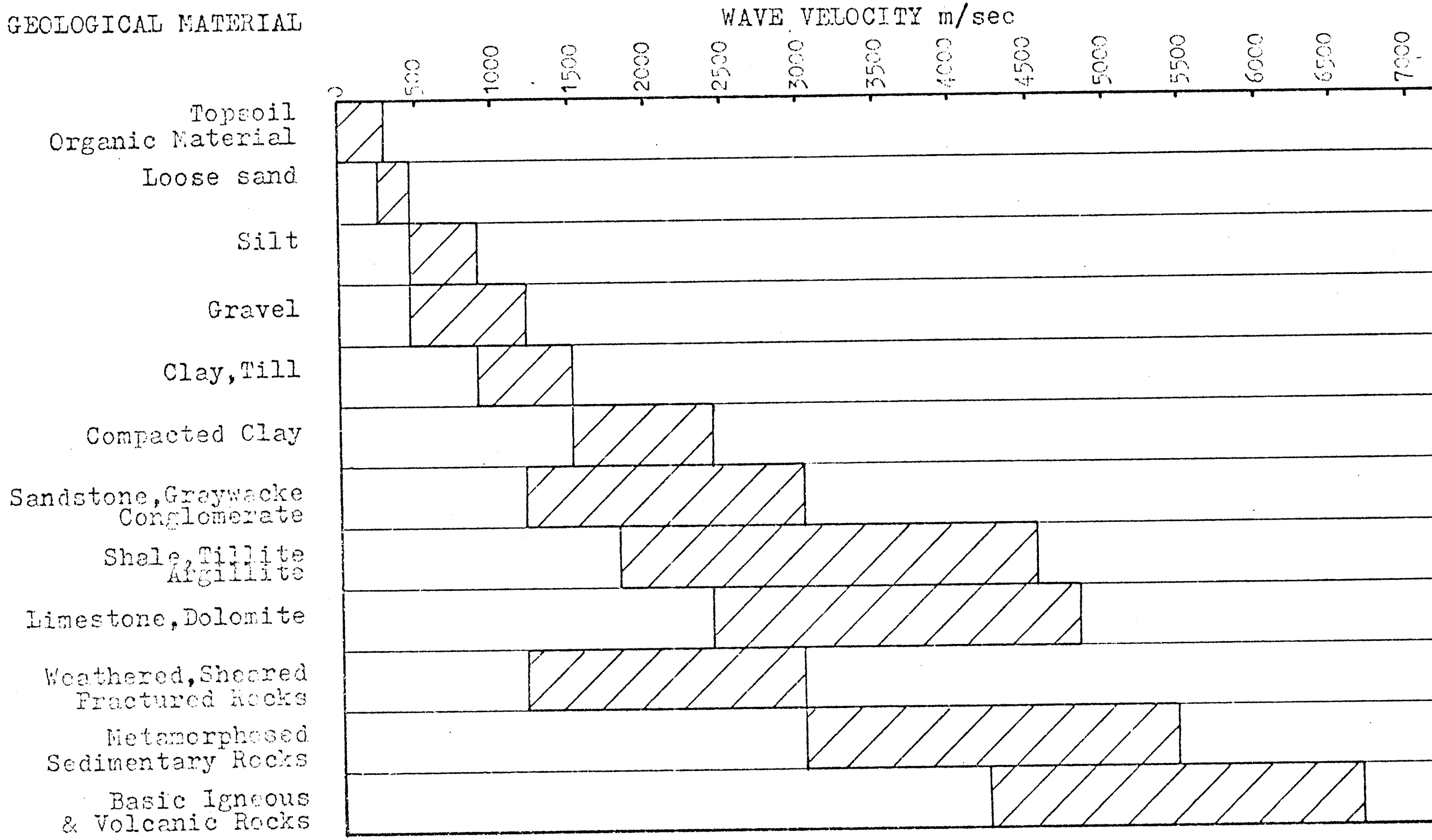
The curves obtained from the surveys, indicate that three layers of different materials exist. The seismic velocities and approximate thickness of each layer, have been tabulated for both runs in Table 4.2.

TABLE 4.2

LAYER	RUN 1 VELOCITY m/s	THICKNESS m	RUN 2 VELOCITY m/s	THICKNESS m
1	250	3.65	250	3.39
2	2780	5.83	2500	1.89
3	5280	-	5000	-

As the seismic velocities and layer thickness for layer two is different for both tests, the interface between layers two and three is dipping. Thus layer three is most likely to be bedrock, from the magnitude of its seismic velocity and the fact that it is dipping.

As the velocity of sound in water is approximately 1500m/sec, fully saturated soils have a seismic velocity equal to or exceeding



Classification of Geological Materials from Seismic Velocities

FIGURE 4.3

this value. From the test results, there is no indication of the ground being saturated for depths above 3.5m.

For comparison with values given in Figure 4.3, Classification Chart of Geological Materials from Seismic Velocities, all the geological materials tested, have been tabulated in Table 4.4. By comparing materials for a given seismic velocity it is quite clear that test results correlate closely with those given in Figure 4.3.

TABLE 4.4

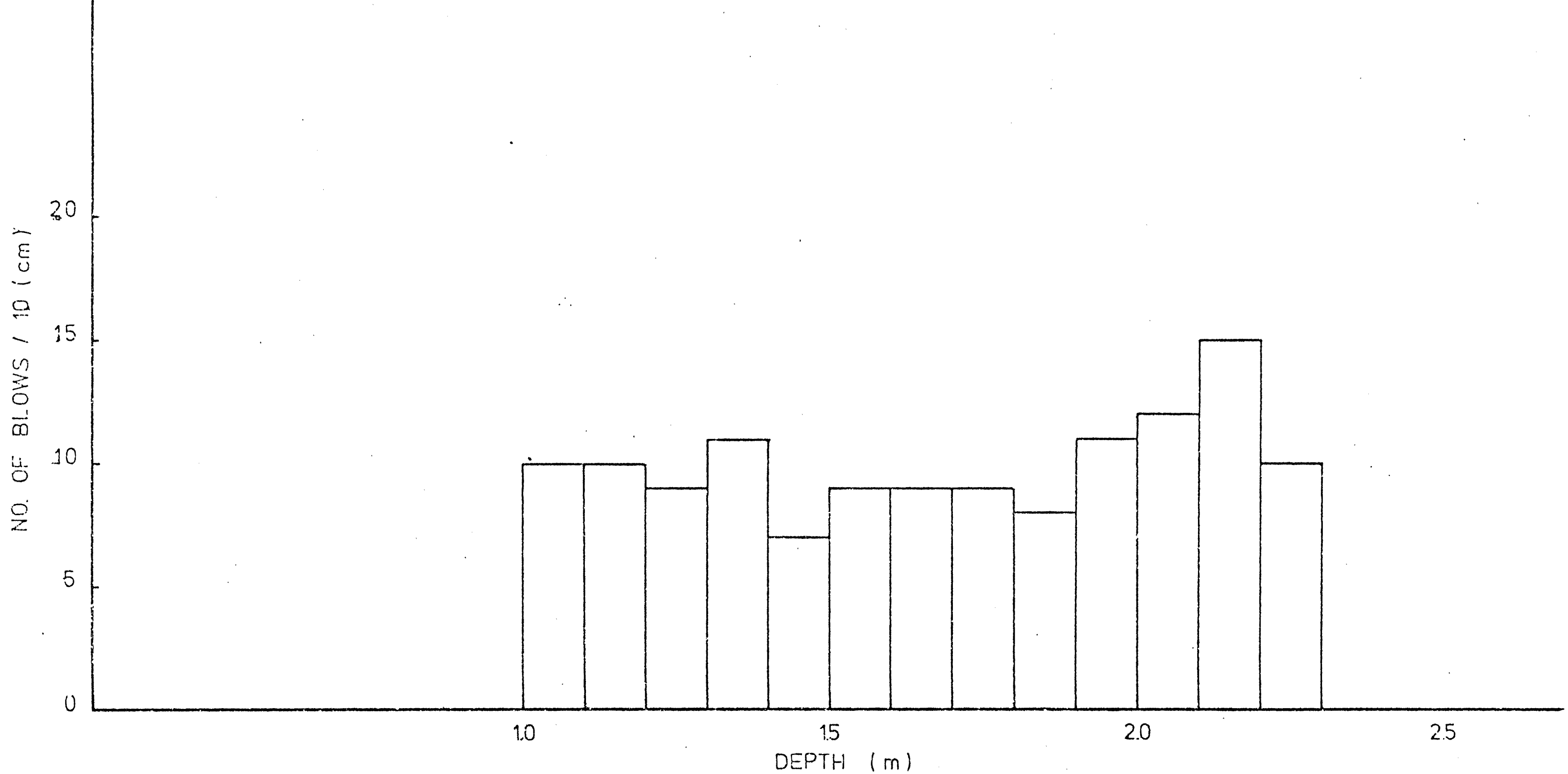
SEISMIC VELOCITY m/sec	GEOLOGICAL MATERIAL
250	Loose sand
500	Fine grain topsoil
1300	Conglomeration of rock and sand
2500-2800	Dense sand and/or weathered rock
5000-5280	Igneous rock (granite)

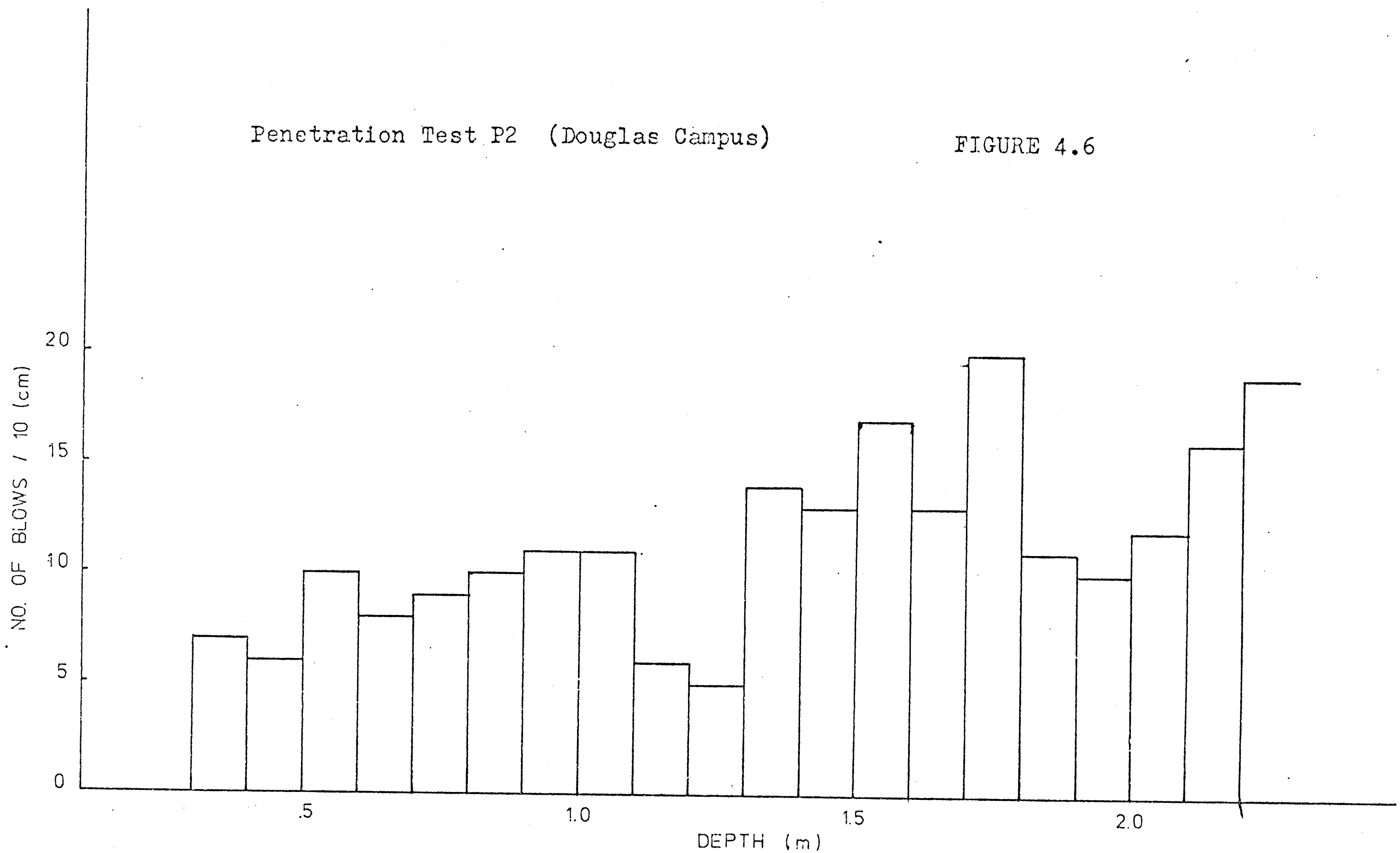
Thus, to determine the relative soundness of a material from its seismic velocity, Table 4.4 and Figure 4.3 may be used to obtain a similar geological material for that velocity. From this material, the relative firmness of the unknown material may then be ascertained.

4.5 PENETROMETER - DOUGLAS CAMPUS

The two shallow penetration tests performed in May 1979, were on a section which had previously been surveyed by several seismic spreads. Only 2.3m of sounding was done for each test, as the depth of the lower stratum was known to be approximately 2m. The main aim of the tests, was to compare the results obtained with those of the siesmograph.

Penetration Test P1 (Douglas Campus). FIGURE 4.5




Penetration Test P2 (Douglas Campus)
FIGURE 4.6
NO. OF BLOWS / 10 (cm)
20
15
10
5
0
.5
1.0
1.5
2.0
DEPTH (m)

The results of both tests have been plotted as a graph of the number of blows required to penetrate 10cm against depth. The graphic form of the results are shown in Figures 4.5 and 4.6. The tests were performed at a distance of 25.3 and 30.5m from the zero position of the seismic survey. Their position are indicated in Figure 4.1 by P_1 and P_2 respectively.

From test P_1, Figure 4.5, an increase in penetration resistance occurs at a depth of 1.9m, with a corresponding increase in the average N values of 9 to 12 blows per 10cm.

In test P_2, Figure 4.6, two increases in penetration resistance occurs at depths of 1.3 and 2.1m. The soil above a depth of 1.3m has an average N value of 8, while below 1.3m, the N values increase to an average of 15 blows per 10cm.

By comparing the results of tests P_1 and P_2, with those of the seismograph, the increases in penetration resistances align very well with the profile of the lower stratum obtained by the seismograph. Thus it can be seen that both techniques are capable of detecting the same formations, even if only for shallow depths.

Due to the simplicity of the penetrometer, a limited amount of testing was required to become conversant with the machines mechanics. Therefore, little information has been collected, to allow the calibration of the machine.

However, by considering the average penetration resistance of the soil of Douglas campus, 10 - 15, the relative soundness of a material can be obtained by comparing N values.

4.6 TECHNICAL PROBLEMS

From these initial tests, the method of recording raw data was found to be inadequate. As collected data from tests was soon disorganized or forgotten, field sheets for recording the penetrometer and seismograph results were fabricated. Examples of the field sheets can be found in Appendix B and C.

CHAPTER 5 - SITE OPERATIONS

5.1 CORE DRILLING OPERATION

5.1.1 Introduction

Core sampling is one of the oldest methods of determining properties of subsurface formations, for foundation problems. From the core recovery, the variation in engineering characteristics of the lower strata, can be determined by laboratory testing.

Drilling operations were performed over a two day period, on the low tides of May, 1979. The tests were carried out on Keeper Reef and their respective locations are shown in Figure 5.1.

5.1.2 Drilling Operations

On passage to Keeper Reef, the raft was towed behind the research vessel 'James Kirby' completely unloaded, due to the rough sea conditions often encountered. On arrival, the vessel was anchored on the sheltered side of the reef. The drilling machine and other auxiliary equipment were loaded upon the raft and prepared for drilling operations the next day.

As with all testing involving the use of the raft, the raft was motored to, or as close as possible to the test site. On several occasions, the test sites could not be reached under power and the raft had to be manhandled the remaining distance. As the tide was sometimes misjudged, or the test site was covered with small coral outcrops, the manoeuvreability of the craft was severely limited.

On reaching the test site, the raft was anchored and allowed to settle onto the reef, with the receding tide. Prior to being stranded, the drilling machine and other equipment were prepared to commence drilling.

A position close to the radar beacon was chosen for drilling, as it is on the algal rim of the reef, where the coral growth is

SITE PLAN

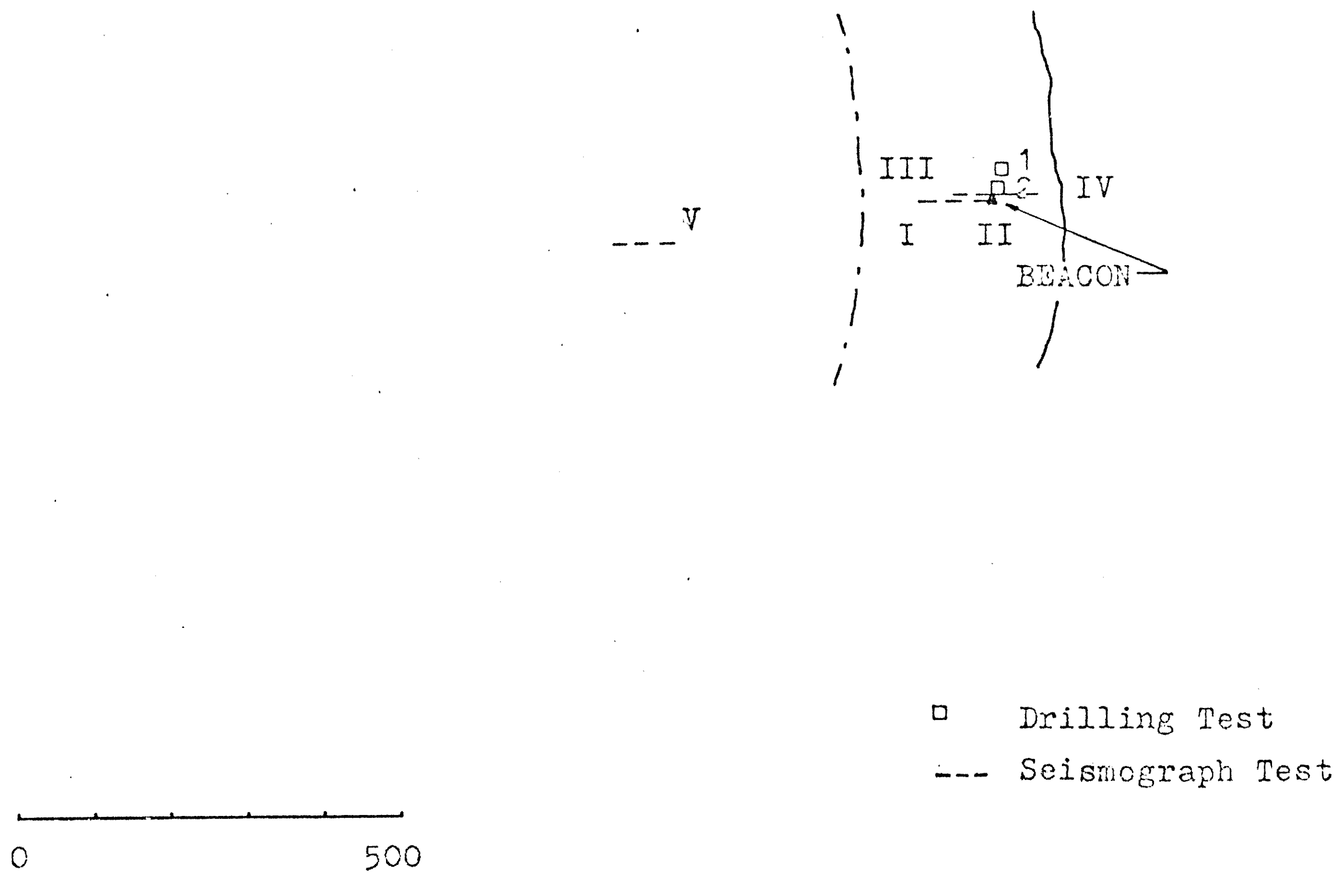


FIGURE 5.1

developed the most. As this position it was assumed that core recovery would be satisfactory.

The first hole was drilled to a depth of 2.1m, which returned a total core recovery of 17% and a maximum of 30% for a 600mm section. Of this recovery, there were no samples of suitable length to allow laboratory testing. An 8.2cm intact section of core sample is required for laboratory testing, if a 4.1cm diameter core sample is recovered. The largest intact sample recovered was approximately 3cm long.

Drilling was stopped after 51 minutes of operation time due to the inflow of sand into the bore hole, on removal of the core barrel. This meant drilling back through the sand periodically, with the possibility of the drill bit jamming in the process. A qualitative description of the core recovered with depth is shown in Table 5.2.

TABLE 5.2

Depth m	Description	Core Recovered Length	Percentage/600mm
0	poor recovery	18cm	30%
.6			
.6	no recovery	-	-
1.2			
1.2	poor recovery	15cm	25%
1.8			
1.8	no recovery	-	-
2.1			

A photograph of the total core recovered from the test is shown in Figure 5.3 labeled test 1.

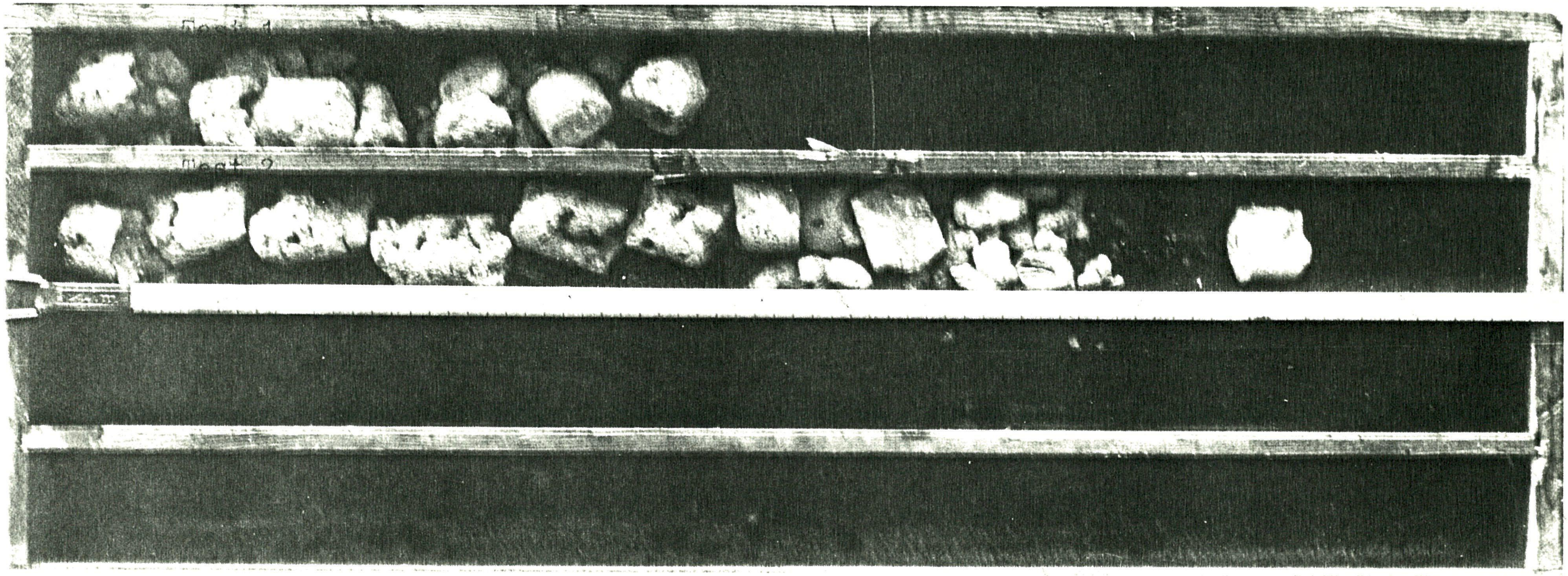
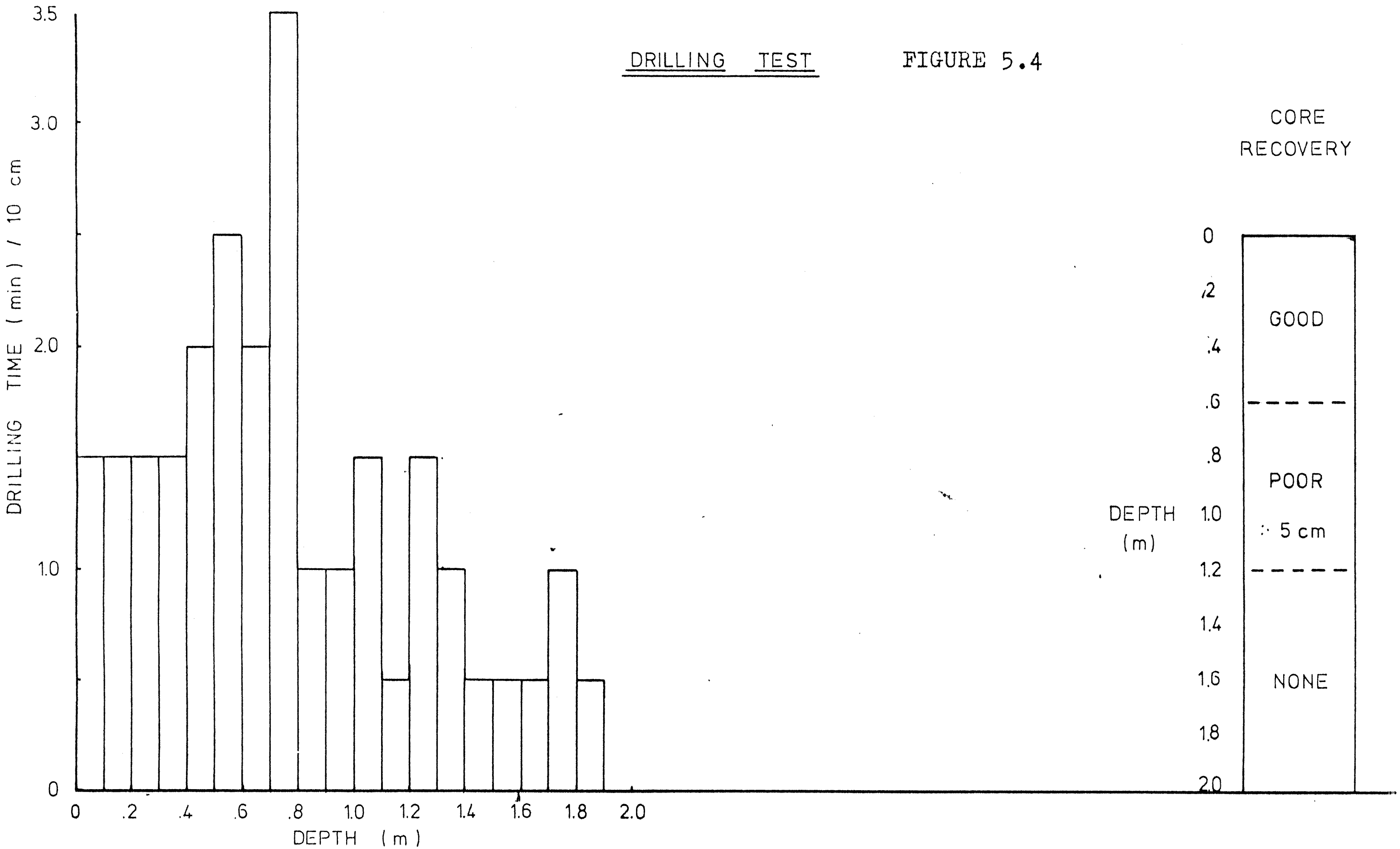


FIGURE 5.3



DRILLING TEST FIGURE 5.4

As the raft was still stranded on completion of the test, the drilling machine could only be moved approximately .6m from the original hole. As the subsurface geology was unlikely to be different at this distance, drilling was ceased for the day.

For the second test, the rate of drilling was monitored, in an attempt to increase the amount of obtainable data from the drilling operations. That is, a tape measure was mounted on the drilling machine so as the time required to drill 10cm could be recorded. Thus by plotting the time to drill 10cm against the depth, an idea of the relative hardness with depth can be ascertained. The results for the second test is shown in Figure 5.4.

Drilling was discontinued after 43 minutes of operation, at a depth 2.4m due to the inflow of sand into the bore hole. The operation returned a total core recovery of 19%, and a maximum of 68% for a 600mm section. Of the core recovered, no samples were suitable for laboratory tests. A qualitative description of the test is given in Table 5.5.

TABLE 5.5

Depth m	Description	Core Recovered Length	Percentage/600mm
0	Good recovery	41cm	68%
.6			
.6	Poor recovery	5cm	8%
1.2			
1.2	No recovery	-	-
1.8			
1.8	No recovery	-	-
2.4			

Of the time taken throughout the test, 58% of the operational time was used drilling and the remaining 42% in retrieving the core barrel and inserting or removing drill rods. A portion of the drilling time also consisted of penetrating the sand in the bore hole.

No further testing was performed with the drilling machine throughout the year, as the technique had been thoroughly tested before by (Moss, 1976). The drilling tests were carried out so that the performance and feasibility of the technique, could be compared and commented against that of other methods.

5.1.3 Comments and Recommendations

From the operations, a definite need for a casing procedure to be introduced into the technique was noted, if core recovery is to be expected from any great depth. This has also been determined earlier by (Moss, 1976) who estimated the cost of adapting the drilling machine including casings, to be around $3000. However, this would stop the inflow of sand into the bore hole, but for the percentage of useful core recovered, the adaption would not be financially viable.

As seen in Figure 5.4 information of the relative hardness can be obtained directly if the drilling rate is monitored. To successfully achieve this, a constant pressure is required on the cutting face, so as a relatively constant drilling rate may be maintained. A hydraulic feed would be required on the machine, to obtain this aim. The extra information collected, would supplement the poor core recovery obtained.

For site investigations (coral reefs) the manoeuverability of the drilling machine or mode of transport would have to be improved to allow some flexibility into the technique. With the present method, the success of the operation is constrained by the tide and prevailing weather conditions.

FIGURE 5.6

5.2 SEISMIC OPERATIONS

5.2.1 Introduction

Before seismic methods were introduced for geophysical explorations the same principles has been applied by earthquake seismologists, to determine the properties of the earth's interior. By the same physical laws, seismic prospecting can be used to ascertain the subsurface geology on a smaller scale.

Several short seismic spreads (90m) were performed on Keeper Reef during June and August of 1979. The main objective was to detect the engineering properties of the coral limestone.

5.2.2 Seismic Tests

Due to the tidal conditions that exist on bare coral reefs, two different techniques were improvised to perform the seismic surveys. Normal land based operations were used on a small section of the reef and a fully afloat technique involving two small boats, was used on the submerged areas.

Land based operations involved the use of a 4.5kg hammer and a metal plate to generate a seismic source. A hydrophone was used instead of normal geophones, because of the wet surface conditions. By using a hydrophone, the hammerman moves along the traverse, as in normal land based operations (Figure 5.6).

The variations that occur in the surface hardness affects the amount of energy transferred into the lower strata by a hammer blow. With this and the ambient noise level, the resolution yielded with the hammer decreases rapidly with distance. Seismic detonators were used, but are uneconomical and slower to use than the hammer.

Although Keeper Reef is approximately 3km long and 1km wide, only a 200m strip along the seaward edge becomes fully exposed at low tide. Of this area, about 10% consists of a hard flat surface, while the majority is covered with numerous coral outcrops and deposits of loose coral. When the surface becomes covered with more than 2-3cm of water, it is inconvenient to work with a steel plate and hammer. Furthermore, the surface conditions deteriorates in a leeward direction due to the degree of coral development decreasing towards the lagoon.

Although land based operations are practical and financially viable, the method is limited by the length of spreads and the resolution yielded. The main factors limiting this type of operation are the tide, weather and general reef development.

To overcome such limitations, a method involving the operation of equipment from two dinghies, working along a float line was improvised.

To perform the operation, a float line is first anchored over the test site, with buoys and reef picks. A hydrophone is placed at the end of the line where the seismograph is stationed in an anchored dinghy. A second dinghy is then rowed along the float line, laying charges at the required distances marked by the floats.

Although the method sounds simple, care was required to keep the electrical firing cable from being fouled on the coral. This was to avoid unnecessary abrasion to the insulation, when moving along the traverse and retrieving the cable.

In case of injury to personnel, extreme care was also required when firing the seismic detonators, as the shot box was situated at the hydrophone end. As the distance between stations increased, a reliable method of signalling had to be developed for firing the detonators.

This method requires more back up equipment and man power (at least four) than land based operations, but allows a greater degree of operational flexibility. The length of electrical cable on each seismic detonator is the only physical limitation to the depth of water in which the operation can be performed. As a 6.2m cable can be ordered, there is no area within the confines of Keeper Reef, that could not have been surveyed.

Although the method is flexible, it is far more time consuming and uneconomical than land based operations, because of the necessary man-power and cost of seismic detonators ($1.70 each). However, this method of surveying allows the use of a single channel seismograph into a marine environment, where other geophysical methods are limited by the reef's topography.

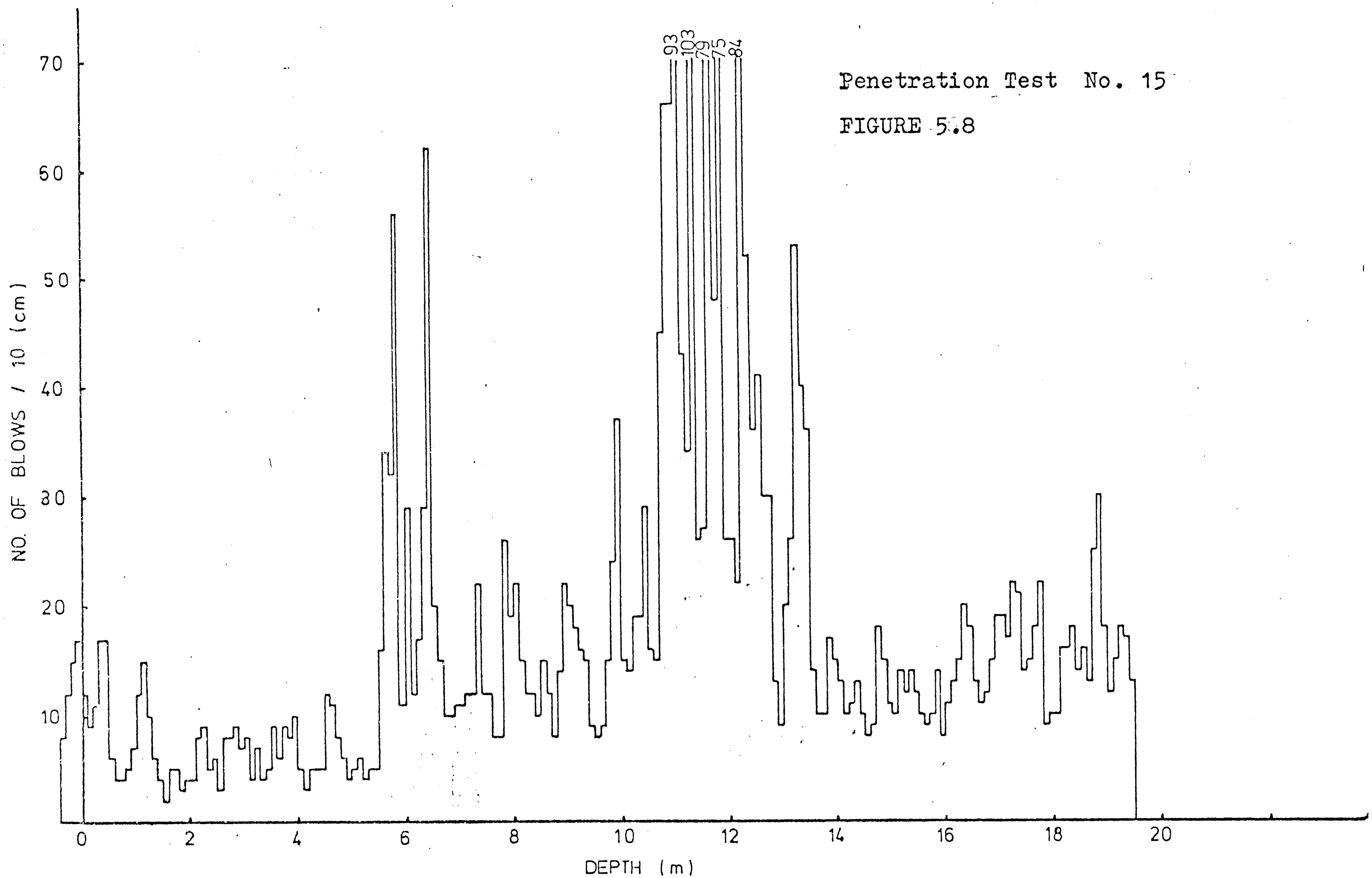
Both methods above, were used to obtain seismic data from a total of five 90 metre spreads. The location of the runs are shown on the site plan of Keeper Reef in Figure 5.1. Four runs were performed on the reef flats and the remaining one a distance of 300m in a northern direction from the beacon.

Runs I, II and V were carried out using the floating method, while III and IV were performed by land based operations. A hydrophone was used instead of the normal geophones for all operations.

5.2.3 Test Results

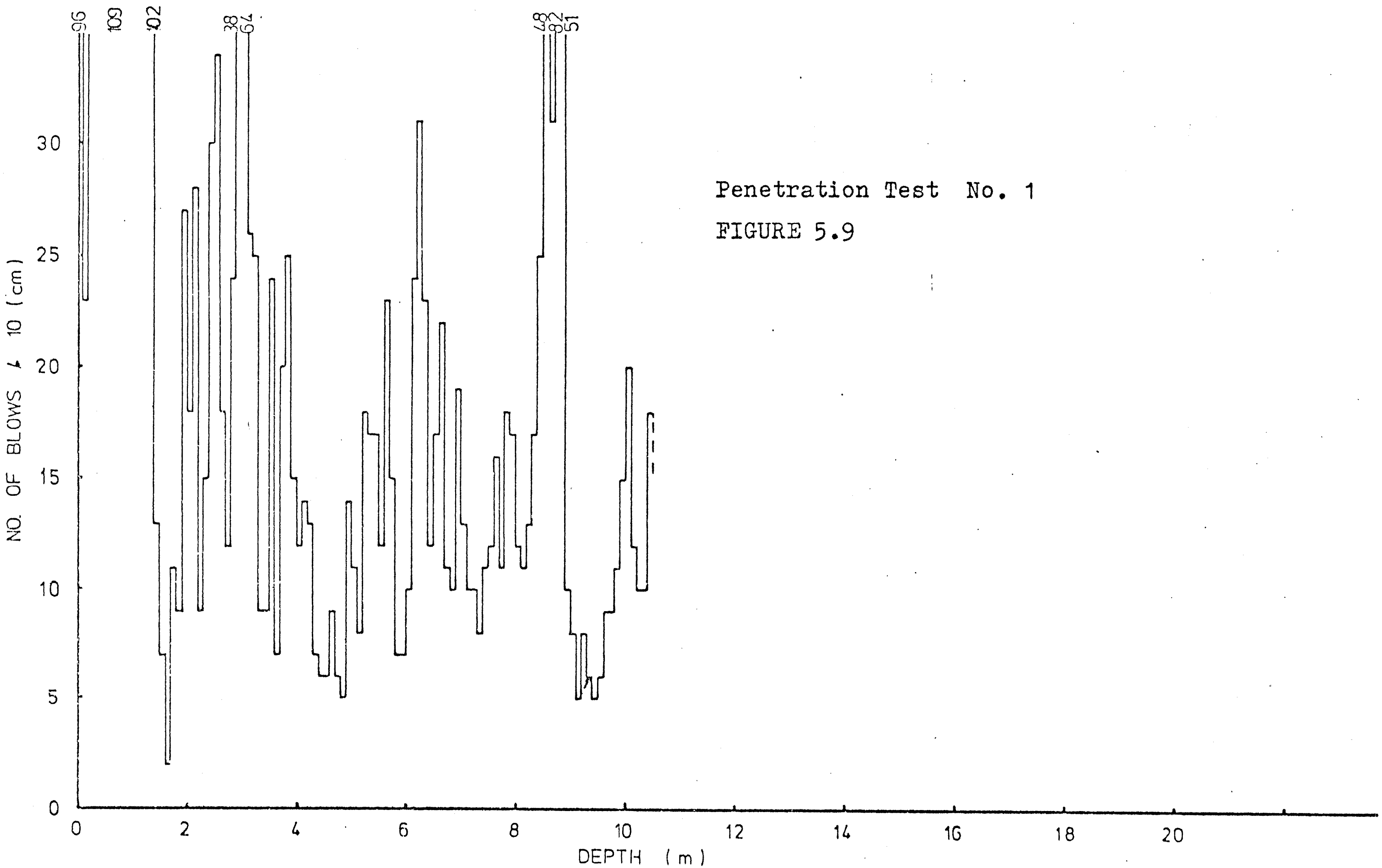
From the raw data of each seismic survey, the seismic velocity of the two shallowest formations, along with the depth of the interface between them, has been calculated. The values from the five runs are shown in Table 5.7. The raw data collected from the surveys is given in Appendix B.

From Table 5.7, the values of V_1 fall within a 7% range, except for run III, where the surface material has a seismic velocity of 2.14km/sec. It must be remembered, that the surface velocity is obtained from the material near the beginning of a spread. The



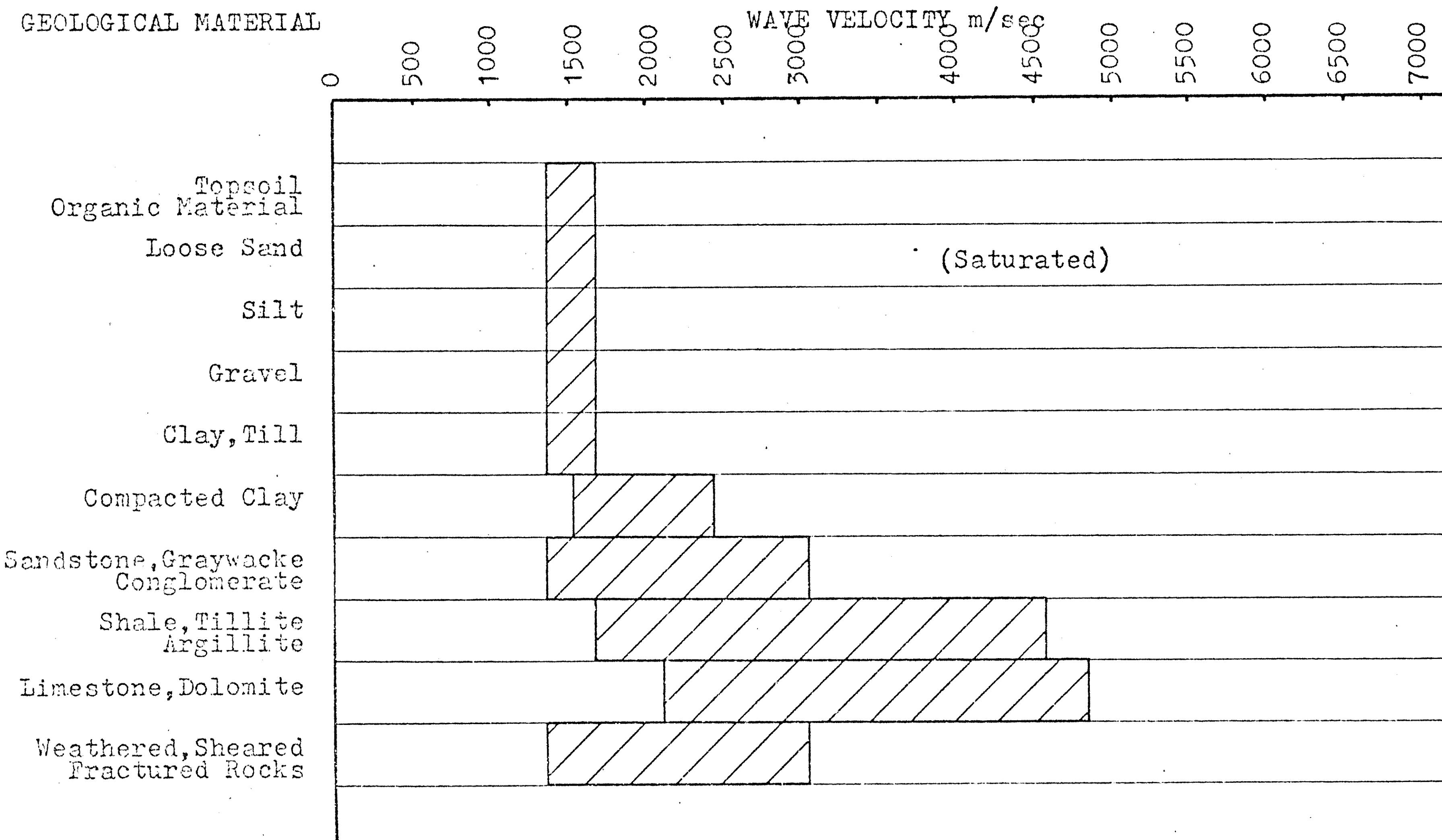
Penetration Test No. 15

FIGURE 5.8



Penetration Test No. 1

FIGURE 5.9



Classification of Geological Materials from Seismic Velocities

FIGURE 5.10

starting position of spreads III and IV also roughly coincide with penetration tests 15 and 1 respectively. From the graphs of these tests, Figure 5.8 and 5.9, the soundness of the near surface material around test 1 is observed to be extremely harder than that of test 15. The variation in V_1 for run III can therefore be accounted for. Thus the seismic velocity of the general reef surface may be taken as approximately 1.7km/sec and 2.1km/sec for harder sections on the reef flats.

TABLE 5.7

Run No.	Layer Velocities (km/s) V_1	V_2	Layer Thickness(m)
I	1.67	2.22	6m
II	1.67	2.09	5m
III	2.14	2.62	5.9m
IV	1.67	2.18 *	3.6m *
V	1.56	1.85	5.1m

* Due to the obscurity of the time-distance curve of the first arrivals, an accurate value of V_2 cannot be ascertained. The depth of the interface between layers, also contains gross errors.

The second layer detected on the reef, has an average seismic velocity of 2.2km/sec, and occurs at a depth of 5-6m. Layers deeper than 6m have not been detected, due to the shortness of the runs. Runs of 300m or more, would be required to determine the depths of deeper deposits.

By comparing the materials given in the Classification Chart of Geological Materials from Seismic Velocities, Figure 5.10, for the seismic velocities of the reef material, the relative soundness of the coral limestone can be obtained.

As the velocity of all saturated low speed materials fall within the same velocity range 1.3 to 1.7km/sec, no comparison can be made for the surface material with a velocity V_1 = 1.7km/sec. However, the coral limestone with V_1 = 2.14km/sec, has a similar velocity of compacted clay through to shale, which is firmer than the topsoil on Douglas Campus (V_1 = 500m/sec).

The seismic velocity of the second layer, corresponds to the velocity of compacted clay through to weathered, sheared or fractured rock. Thus the relative firmness is substantially higher for the coral limestone than the lower stratum detected at Douglas Campus, which had a seismic velocity of 1.31km/sec.

Therefore, from the seismic velocities obtained of the limestone it could be generally assumed that the coral limestone is firmer than the topsoil at Douglas Campus.

For a layer to be detected by refraction methods, its velocity must be higher than that of the preceding layer. This must occur for all successively deeper layers. When a stratum has a seismic velocity lower than the one above it, it will not occur on the time-distance plot of the first arrivals. Therefore, layers of sand or very porous coral which occur below firmer material, will not be detected. Errors will result in the calculation of depths and thickness for layers below these weaker layers.

As the presence of softer material cannot be detected, a hole full of water within the limestone will not be detected also. This is due to the seismic waves propagating across the hole, via the fluid medium.

Therefore, the results of any survey will only allow the characteristics of the firmer strata that exist on the test site to be detected. Thus, the seismic velocity, depth and thickness of each layer are the only characteristics that can be derived directly from a time-distance plot of a spread.

5.2.4 Comments and Recommendations

The seismograph is not capable of determining the engineering properties of coral reefs, with respect to holes and weak spots that exist within the coral limestone. However, the general subsurface formation of the reef can be determined, with an indication of the relative soundness of each stratum.

By using a floating survey technique, far more flexibility is obtained than by land based operations. Also the cost factor involved in the floating techniques, is outweighed by the amount of surveying that can be performed.

For future surveys, it would be more advantageous to use a 1x2 channel operation rather than a single channel, due to the increase in signal to noise ratio. Therefore, by using two hydrophones instead of one, the clarity of the time distance plot may be improved.

Due to the electrical cable fouling on coral outcrops, it would be helpful if attachable floats could be connected to the cable while being fed out. This would allow the cable to be kept off the reef floor, allowing further manoeuverability to the shot station.

5.3 PENETROMETER OPERATIONS

5.3.1 Introduction

The main use of heavy dynamic penetrometers in the past, was for onshore site investigation work. The determination of various strata thicknesses and their relative hardness, was one aspect of the machine's capabilities.

As the penetrometer has had only minor previous testing (three soundings in 1978) it is intended to perform further testing to assess its feasibility as a site investigation technique on bare

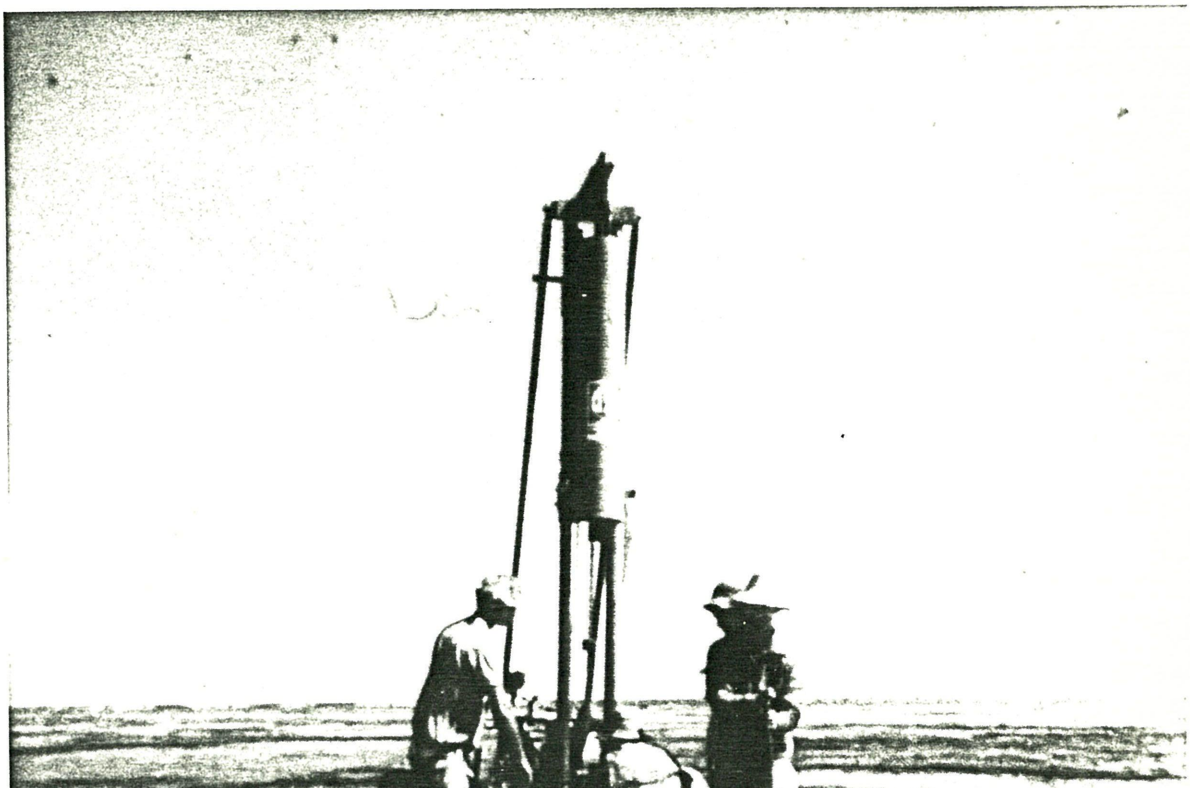


FIGURE 5.11

coral reefs. The objective is to determine the engineering properties of coral limestone to a depth of 25m.

5.3.2 Testing Operations

The heavy penetrometer used for the investigation, is shown in Figure 5.11, mounted on the raft in an operational state. Two series of tests were achieved in June and August of 1979, where six and ten soundings were achieved respectively.

For previously performed tests, the raft was anchored over the test site and allowed to become stranded before testing was started. However, from the May trip (drilling series 1979) it was observed that the testing procedure could be made more flexible, if tests were performed while the raft was afloat.

Prior to the June trip, manually operated winches were installed onto the raft at each corner. The purpose was to prevent any slack occurring in the anchor cables, thus stopping any appreciable movement of the raft while anchored. This was done in an attempt to increase the work potential of the raft, by allowing tests to be performed while afloat. Thus testing would not be restricted to a small percentage of the reef area, consisting of the seaward edge of the reef and isolated outcrops. Furthermore, the available testing time in a day would not be limited to a few hours either side of low tide.

5.3.3 First Series of Tests

The first series of penetration tests were carried out during four days of extremely bad weather in June. A total of six tests to an average depth of 22m were performed, of which five were achieved while afloat and the other test performed stranded on a large coral outcrop.

The initial test was executed in approximately 2m of water,

SITE PLAN

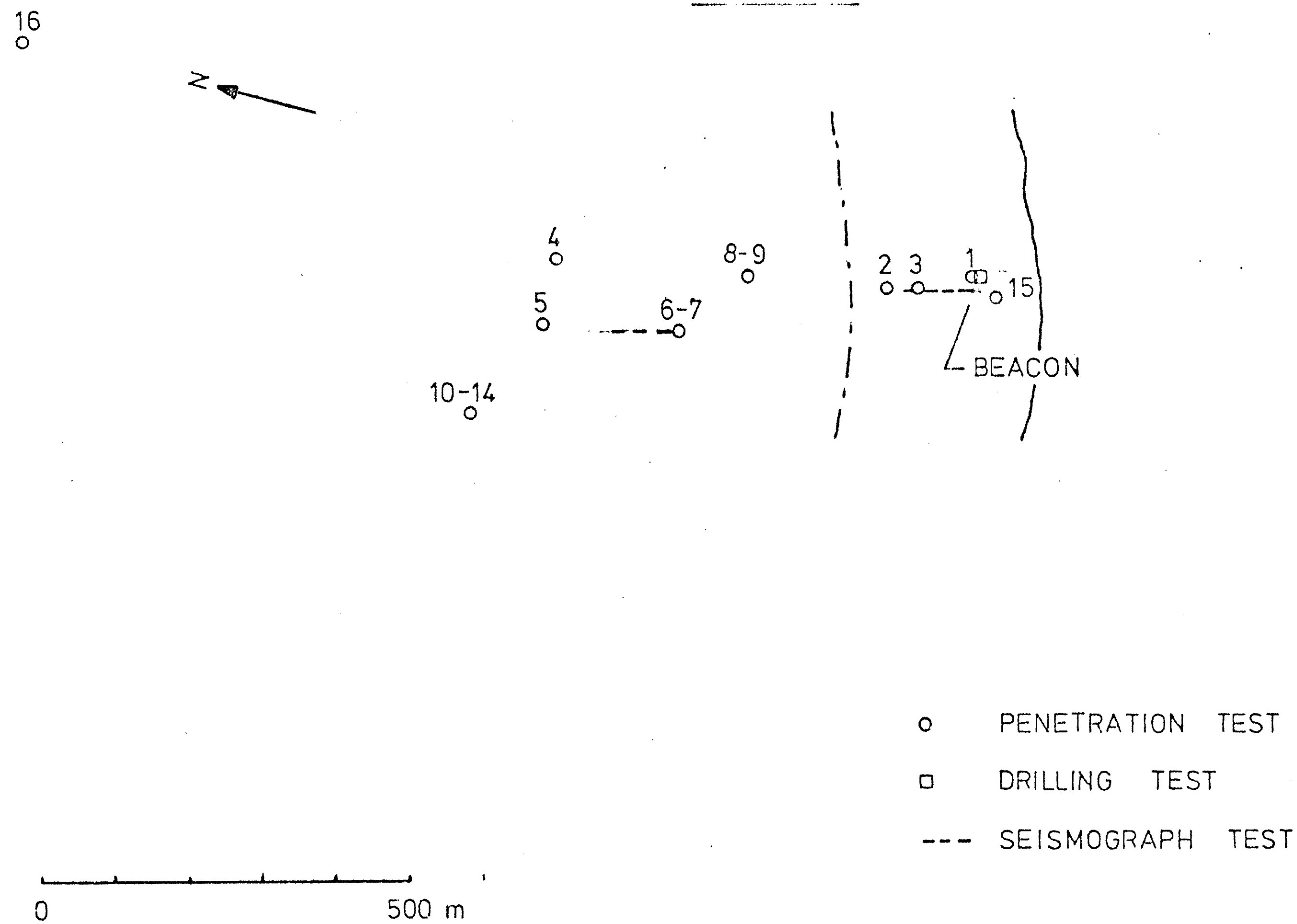
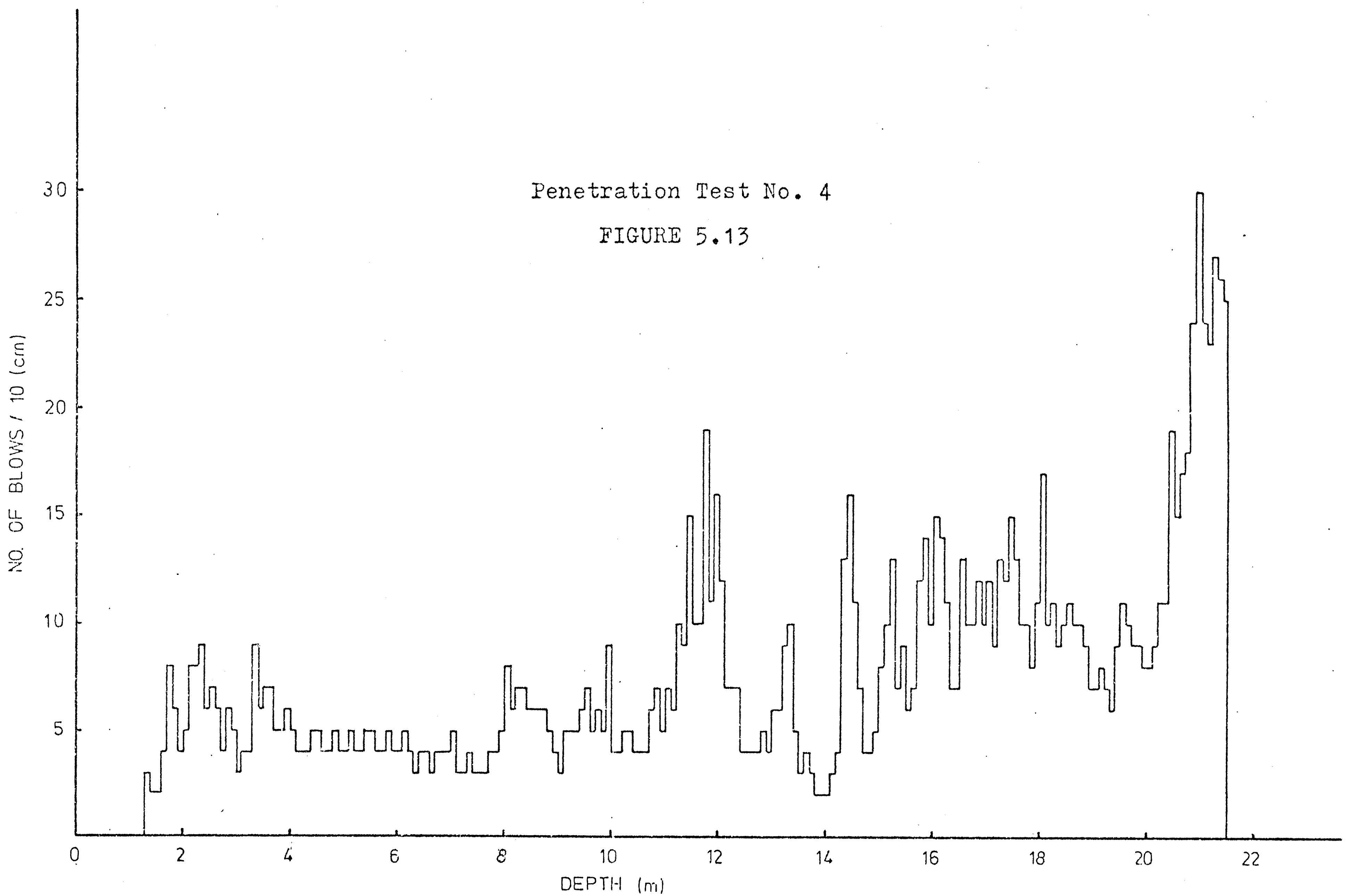
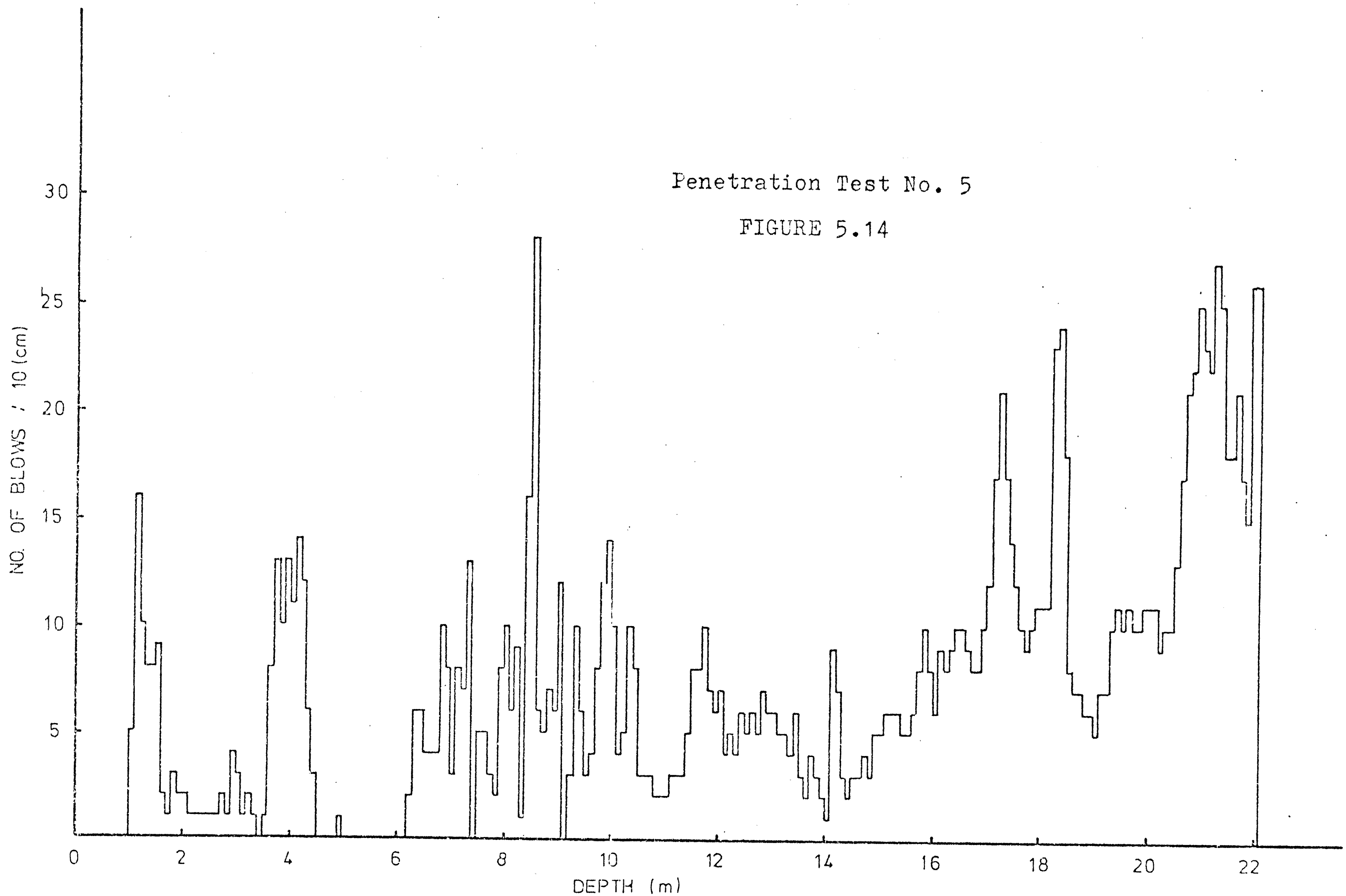


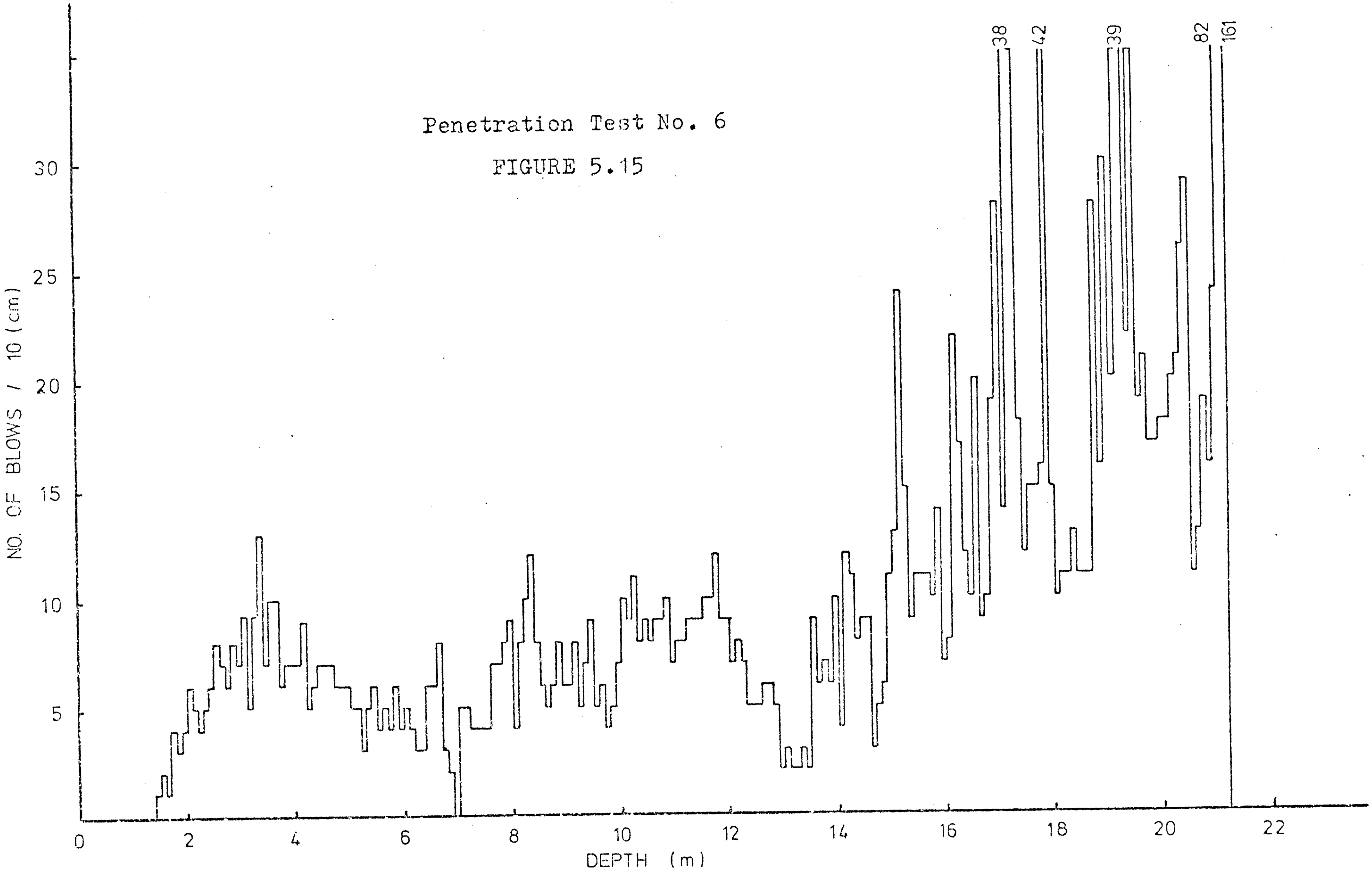
FIGURE. 5.12


Penetration Test No. 4
FIGURE 5.13
NO. OF BLOWS / 10 (cm)
30
25
20
15
10
5
0
2
4
6
8
10
12
14
16
18
20
22
DEPTH (m)


Penetration Test No. 5
FIGURE 5.14
NO. OF BLOWS / 10 (cm)
DEPTH (m)
30
25
20
15
10
5
0
2
4
6
8
10
12
14
16
18
20
22

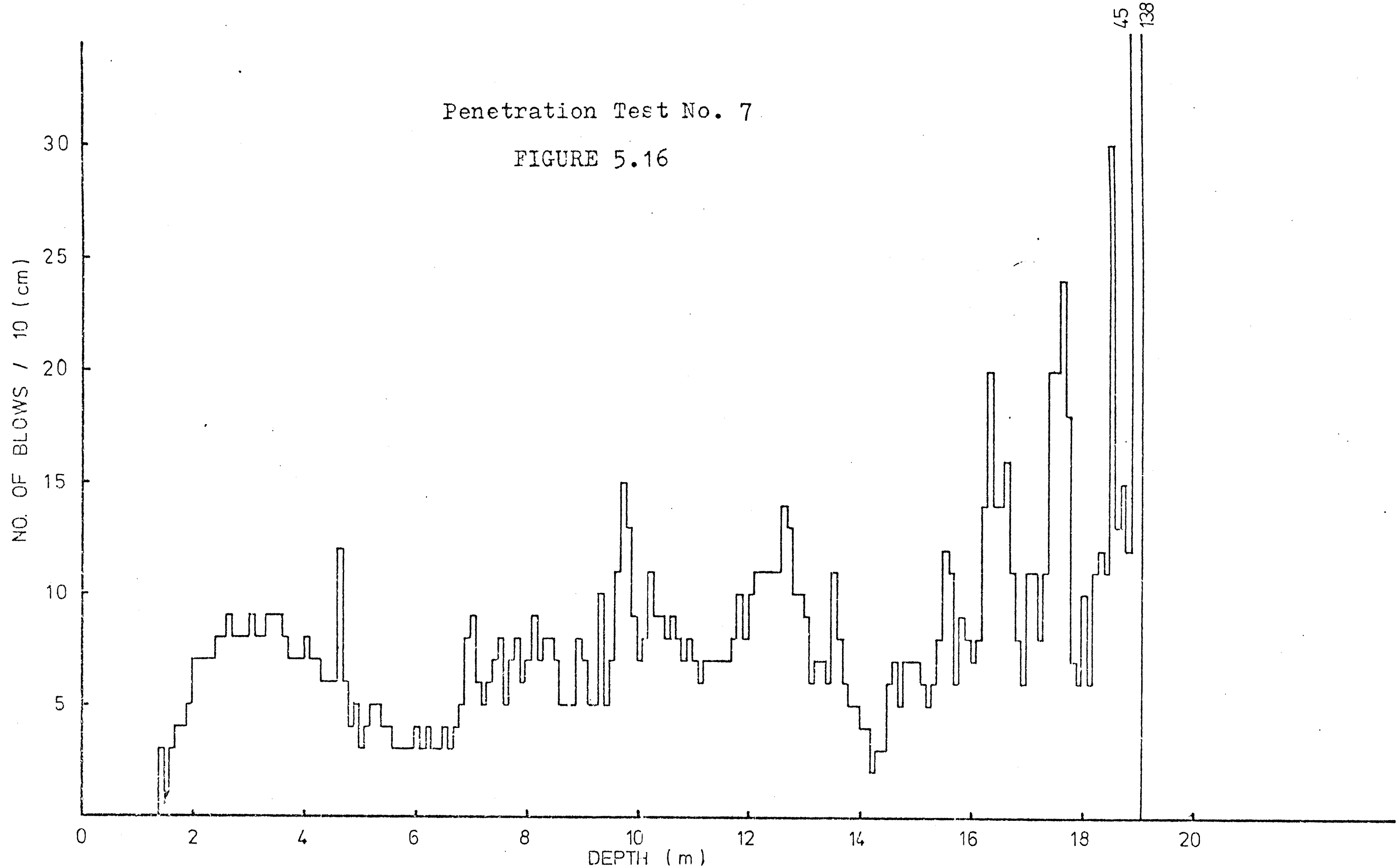
Penetration Test No. 6

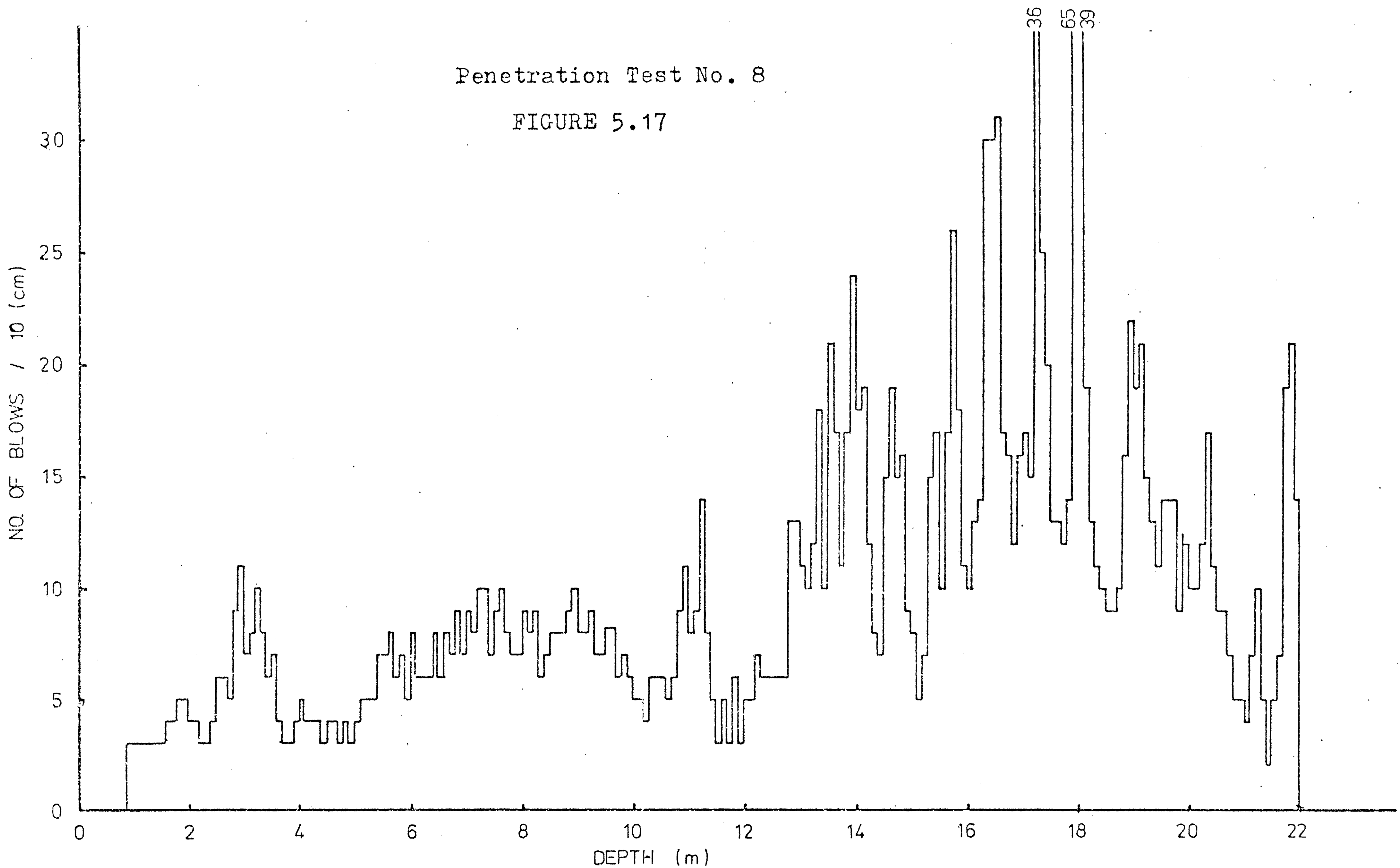
FIGURE 5.15



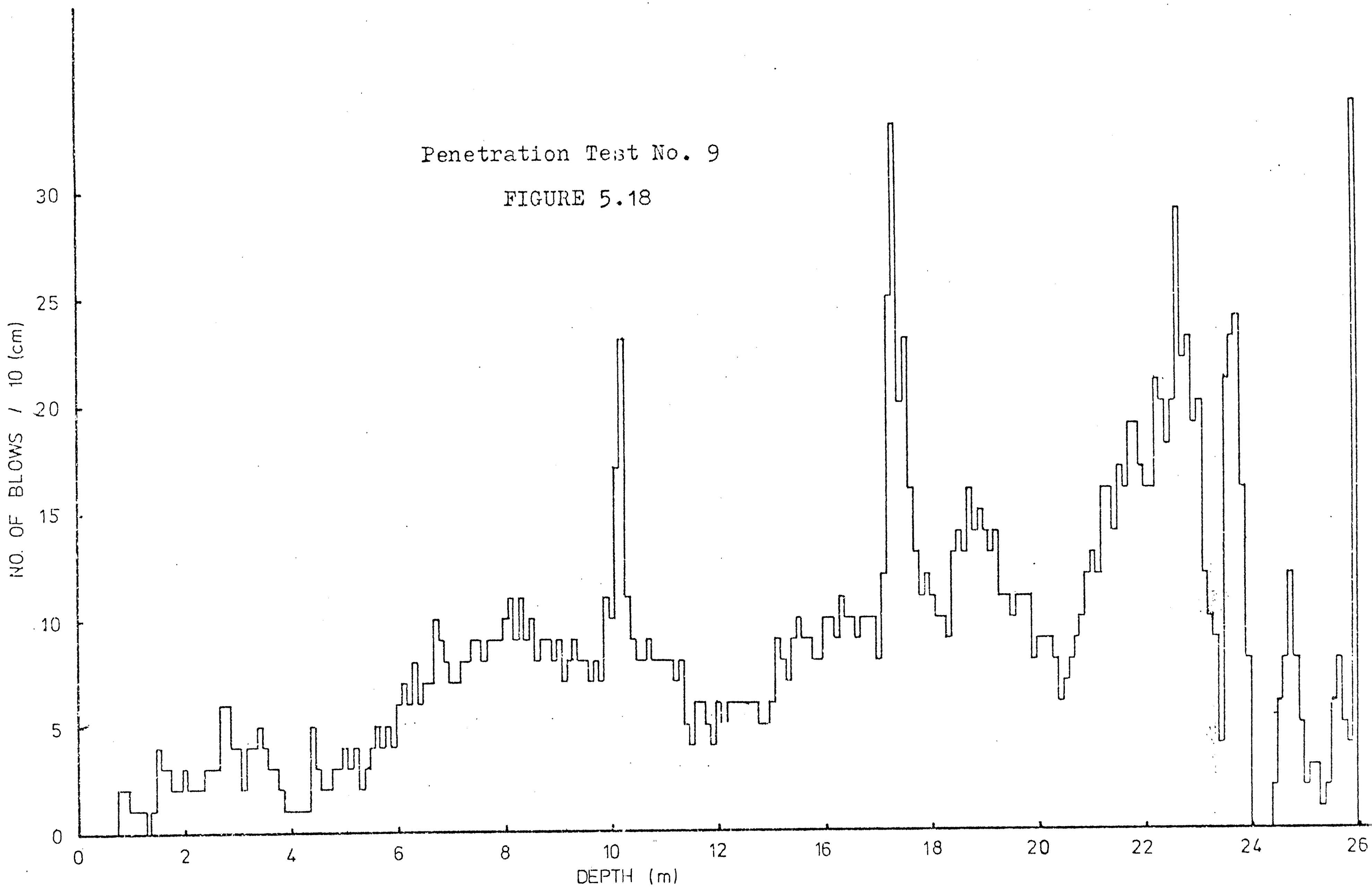
Penetration Test No. 7

FIGURE 5.16



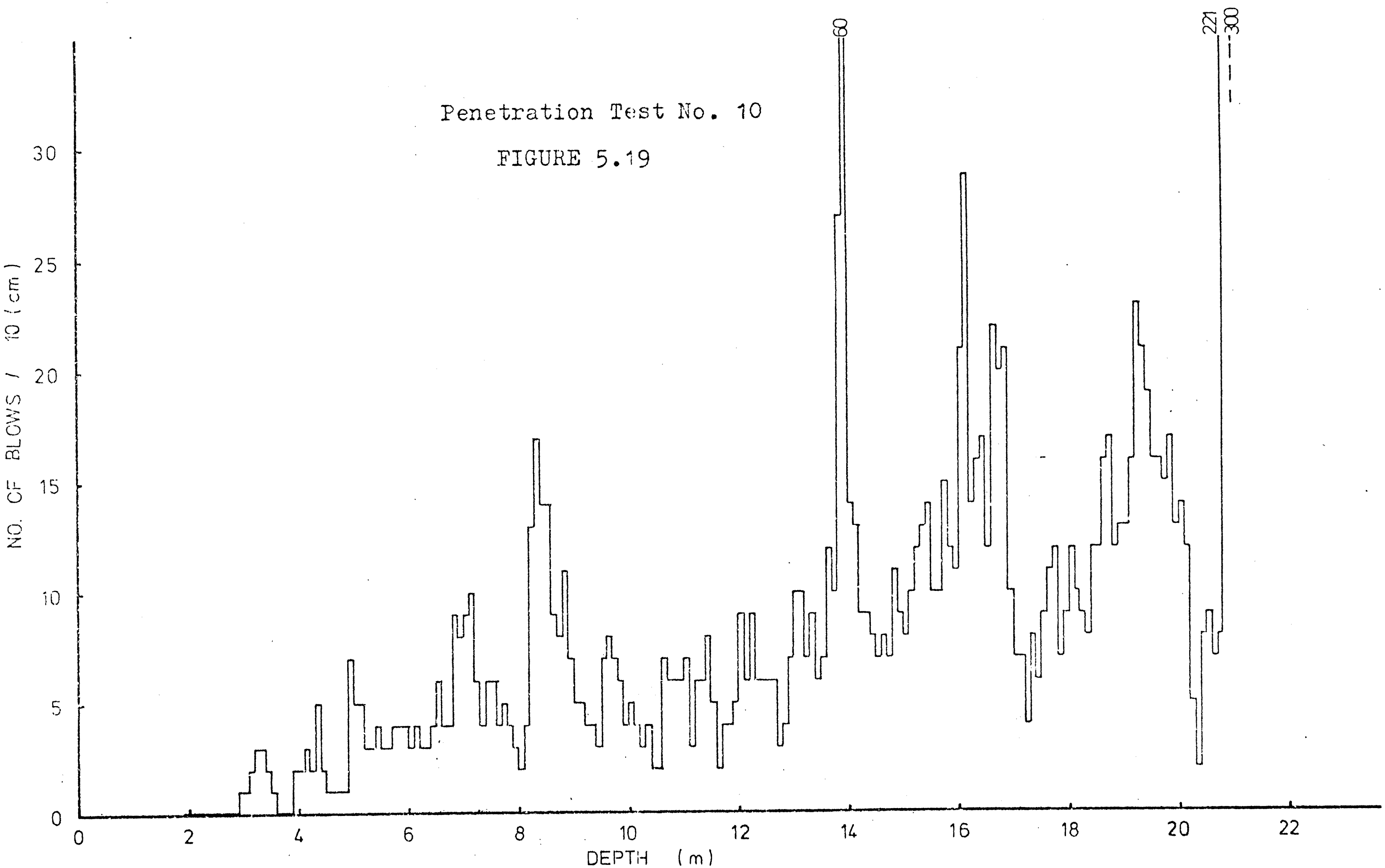


Penetration Test No. 8
FIGURE 5.17
NO OF BLOWS / 10 (cm)
DEPTH (m)
0
5
10
15
20
25
30
0
2
4
6
8
10
12
14
16
18
20
22
36
65
39

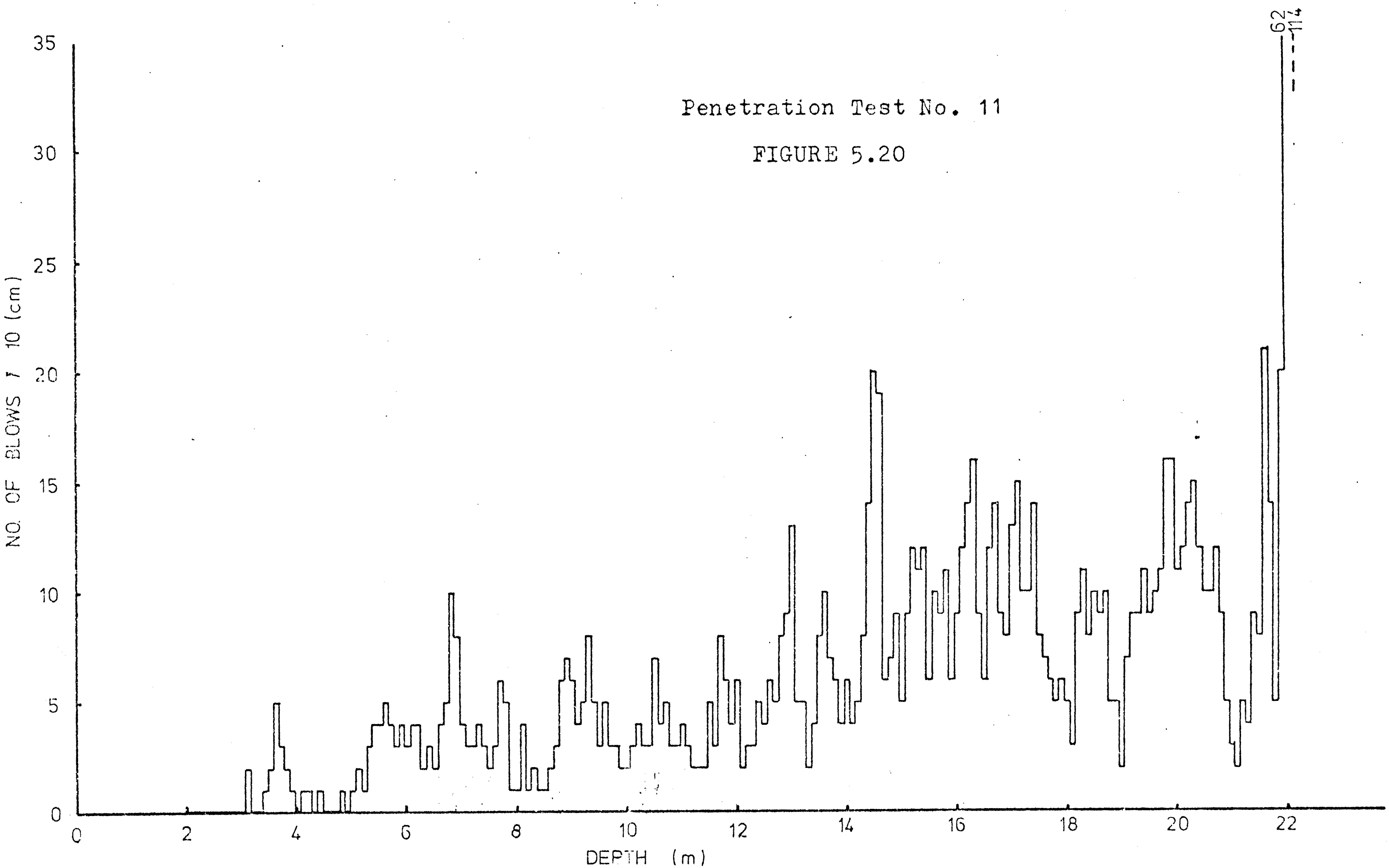


Penetration Test No. 9
NO. OF BLOWS / 10 (cm)
0
5
10
15
20
25
30
0
2
4
6
8
10
12
16
18
20
22
24
26
DEPTH (m)

FIGURE 5.18



Penetration Test No. 10
FIGURE 5.19
NO. OF BLOWS / 10 (cm)
DEPTH (m)
0
5
10
15
20
25
30
0
2
4
6
8
10
12
14
16
18
20
22
60
221
300

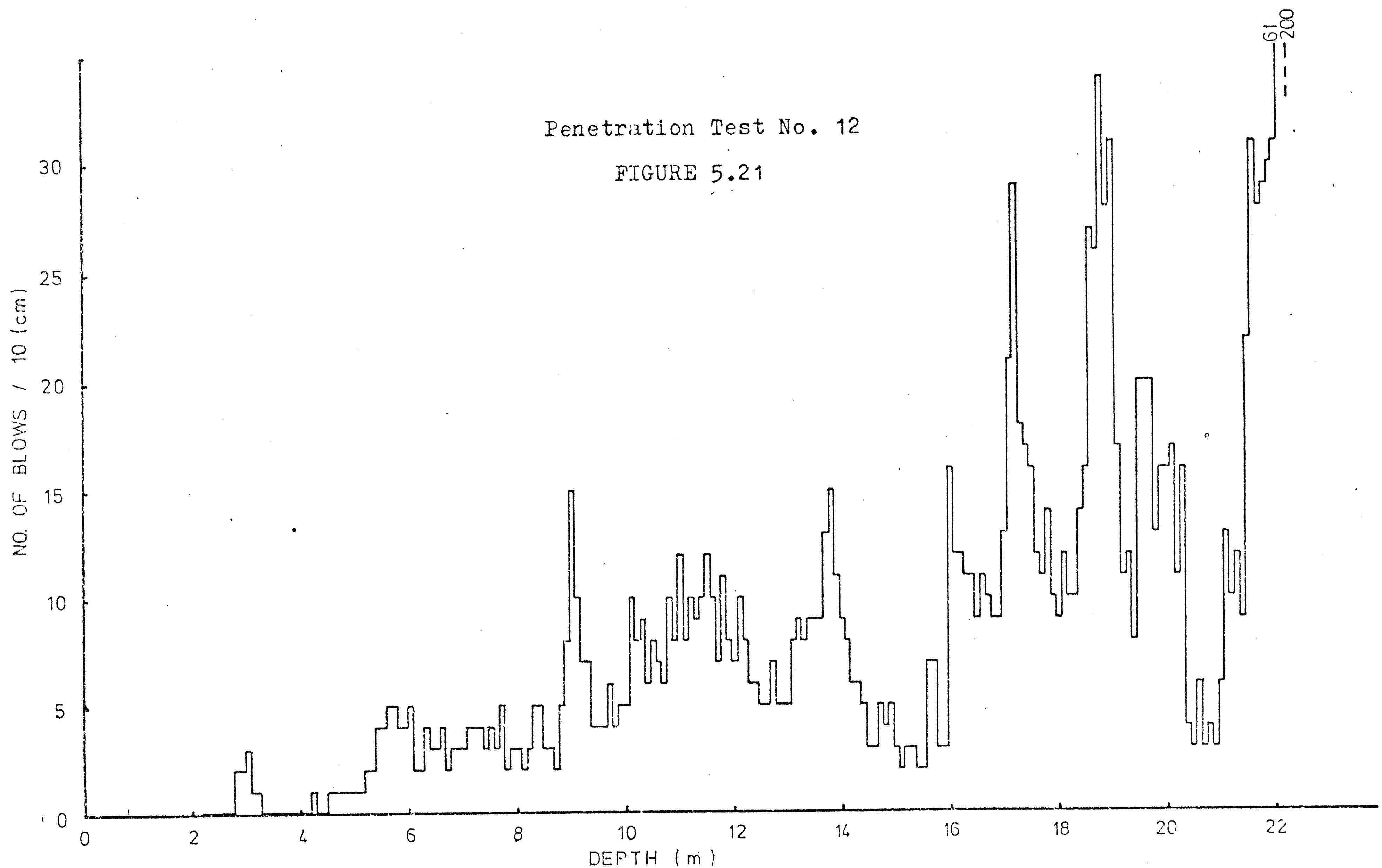


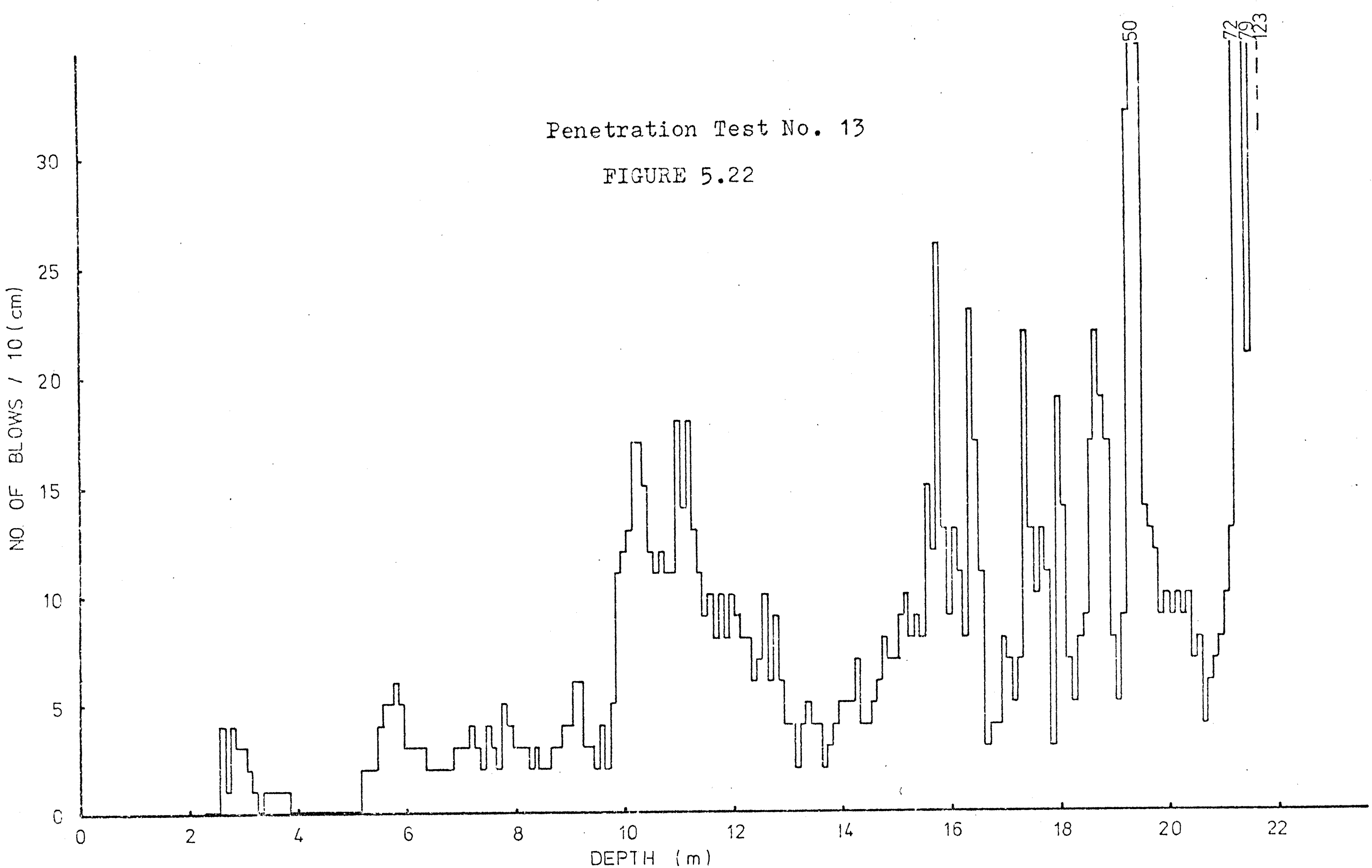
Penetration Test No. 11
NO. OF BLOWS / 10 (cm)
DEPTH (m)
62
114
0
5
10
15
20
25
30
35
0
2
4
6
8
10
12
14
16
18
20
22

FIGURE 5.20

Penetration Test No. 12

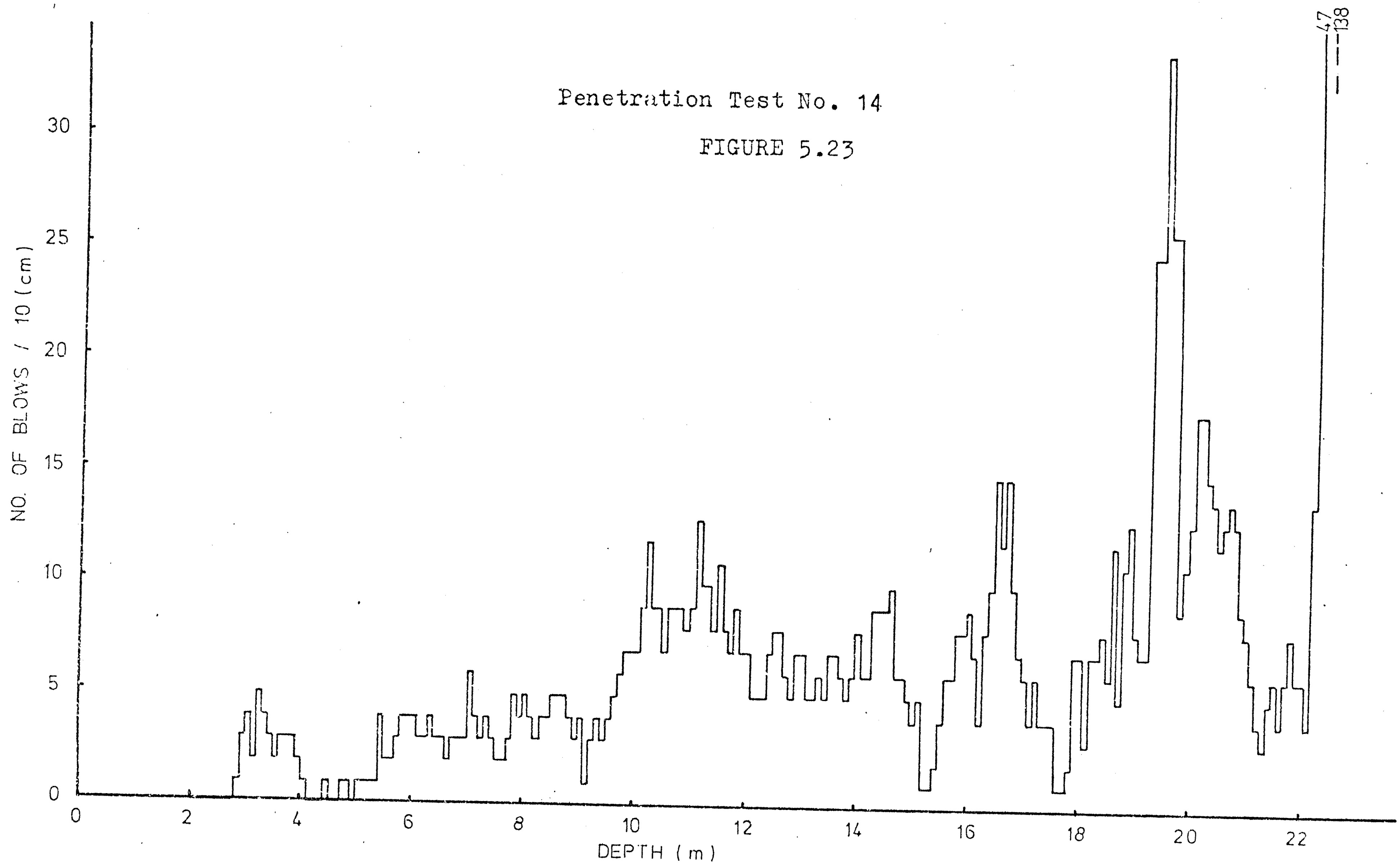
FIGURE 5.21

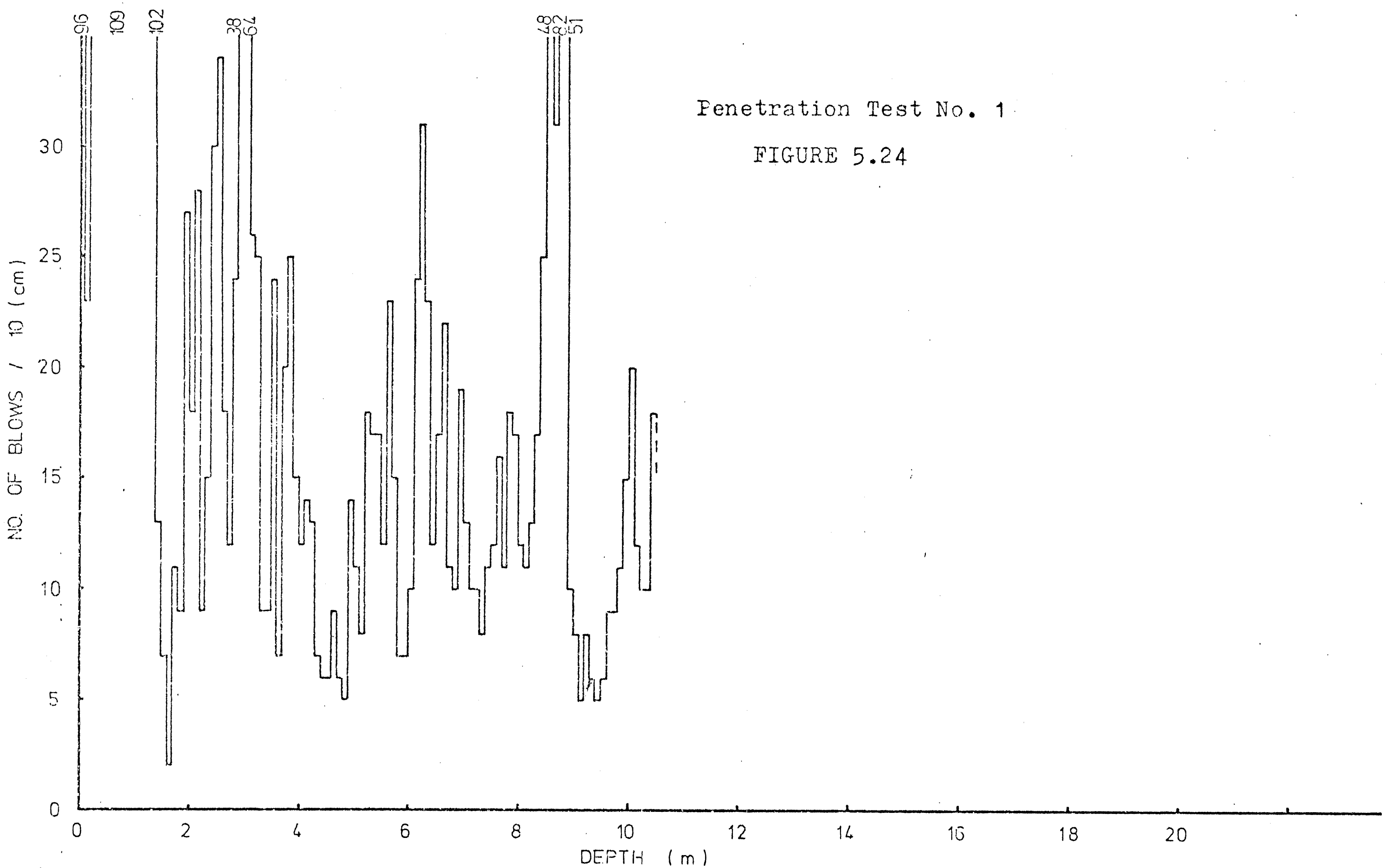



Penetration Test No. 13
FIGURE 5.22
NO. OF BLOWS / 10 (cm)
DEPTH (m)
0
5
10
15
20
25
30
2
4
6
8
12
14
16
18
22
50
72
79
123

Penetration Test No. 14

FIGURE 5.23



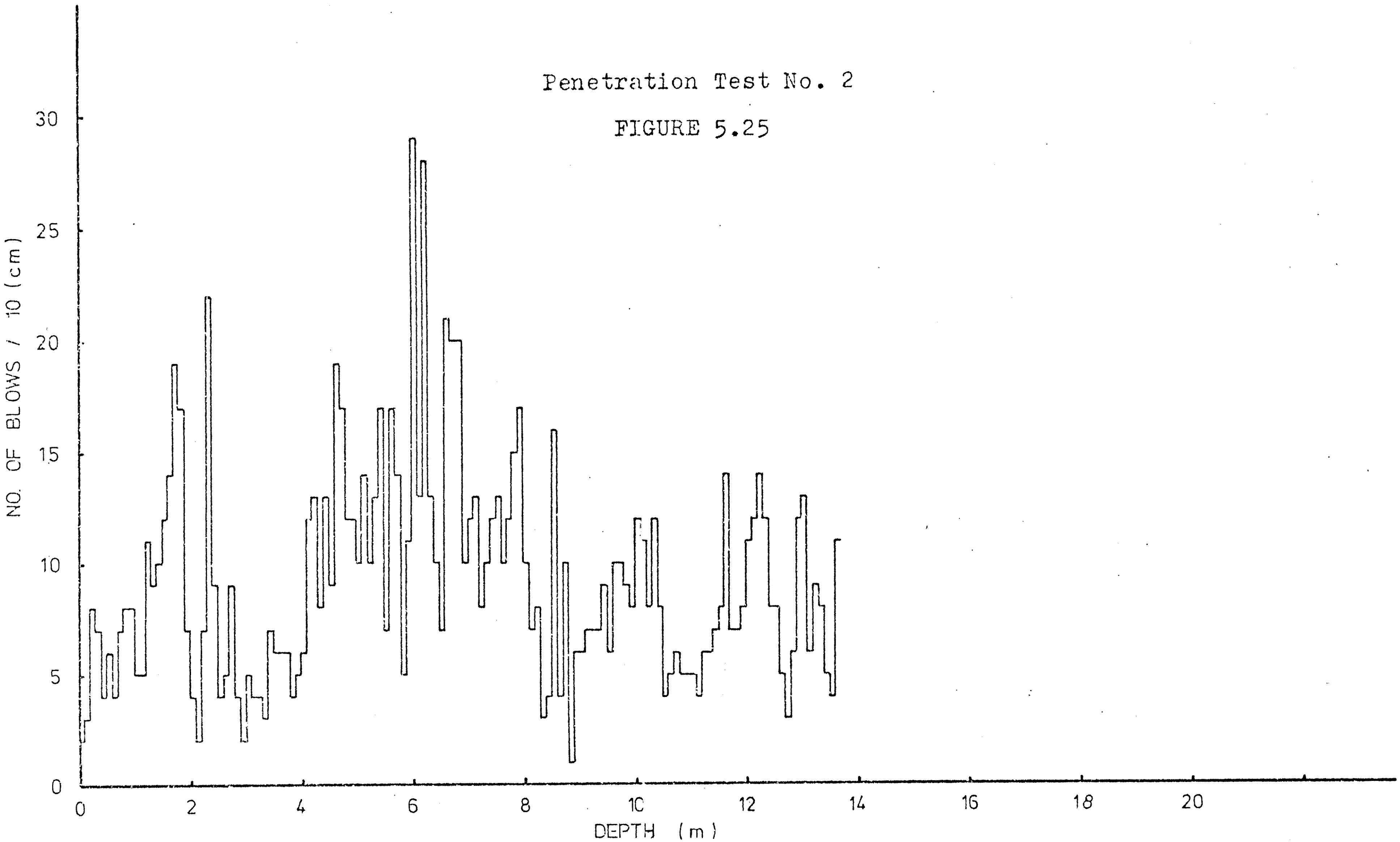


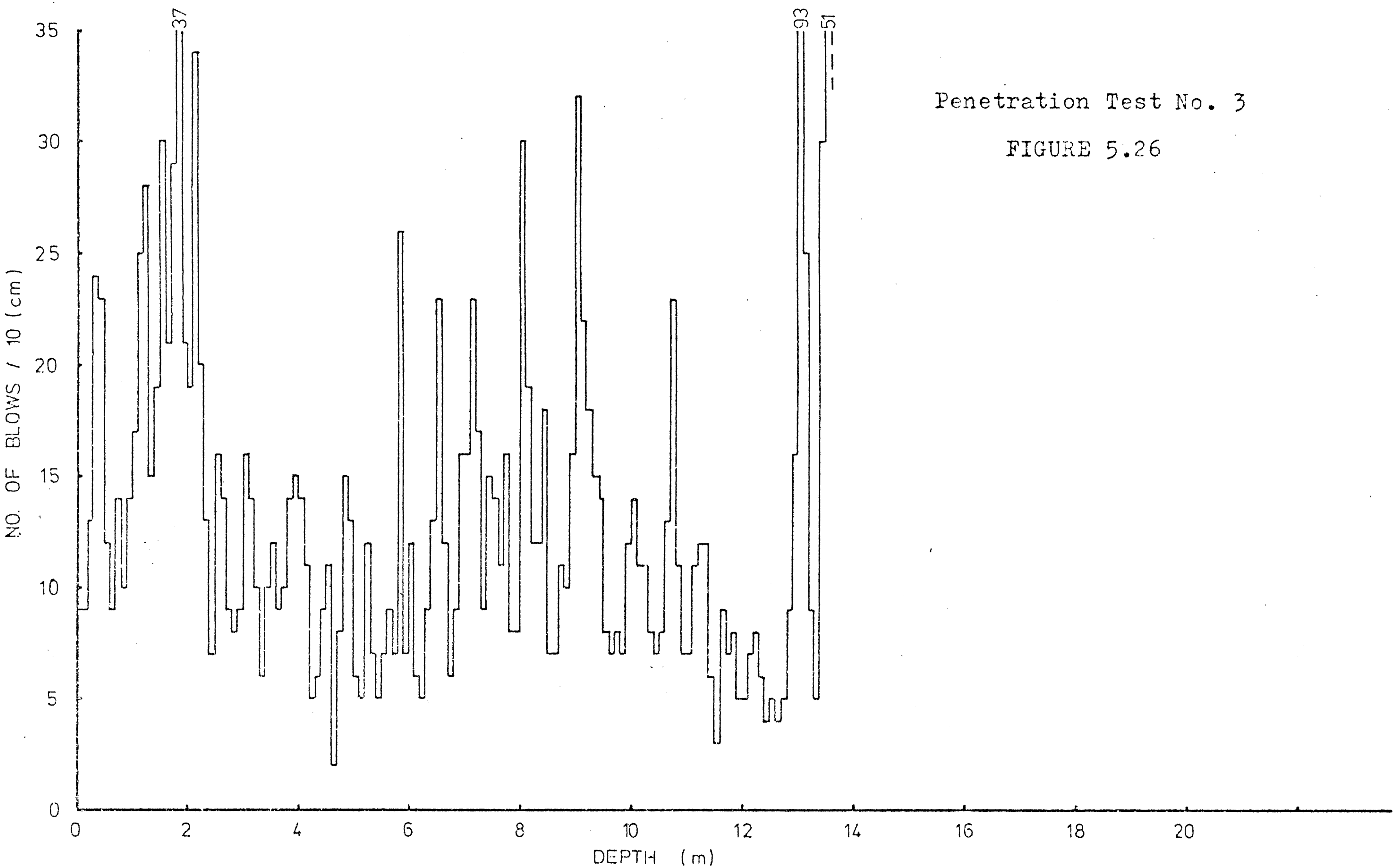
Penetration Test No. 1

FIGURE 5.24

Penetration Test No. 2

FIGURE 5.25



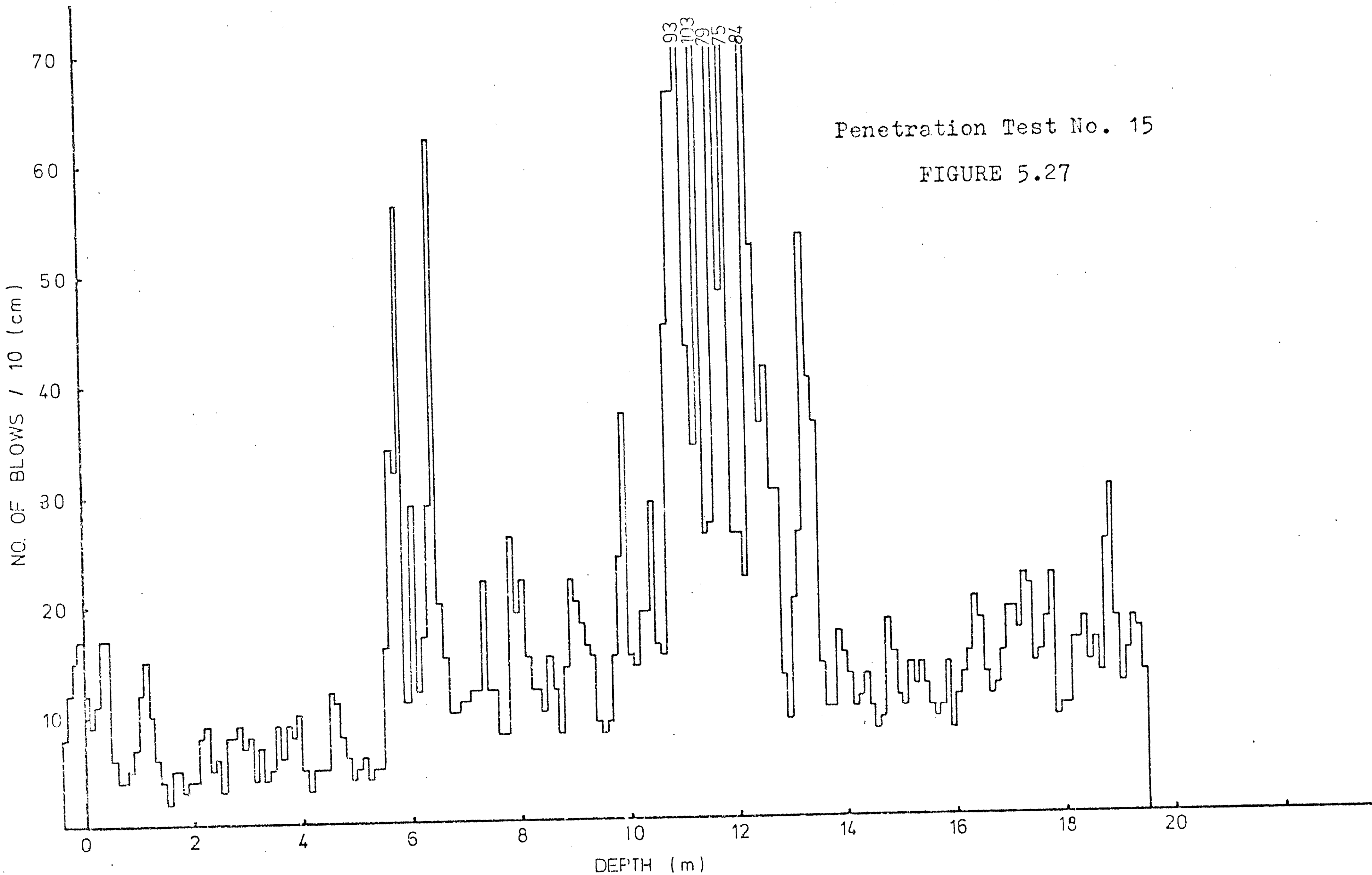


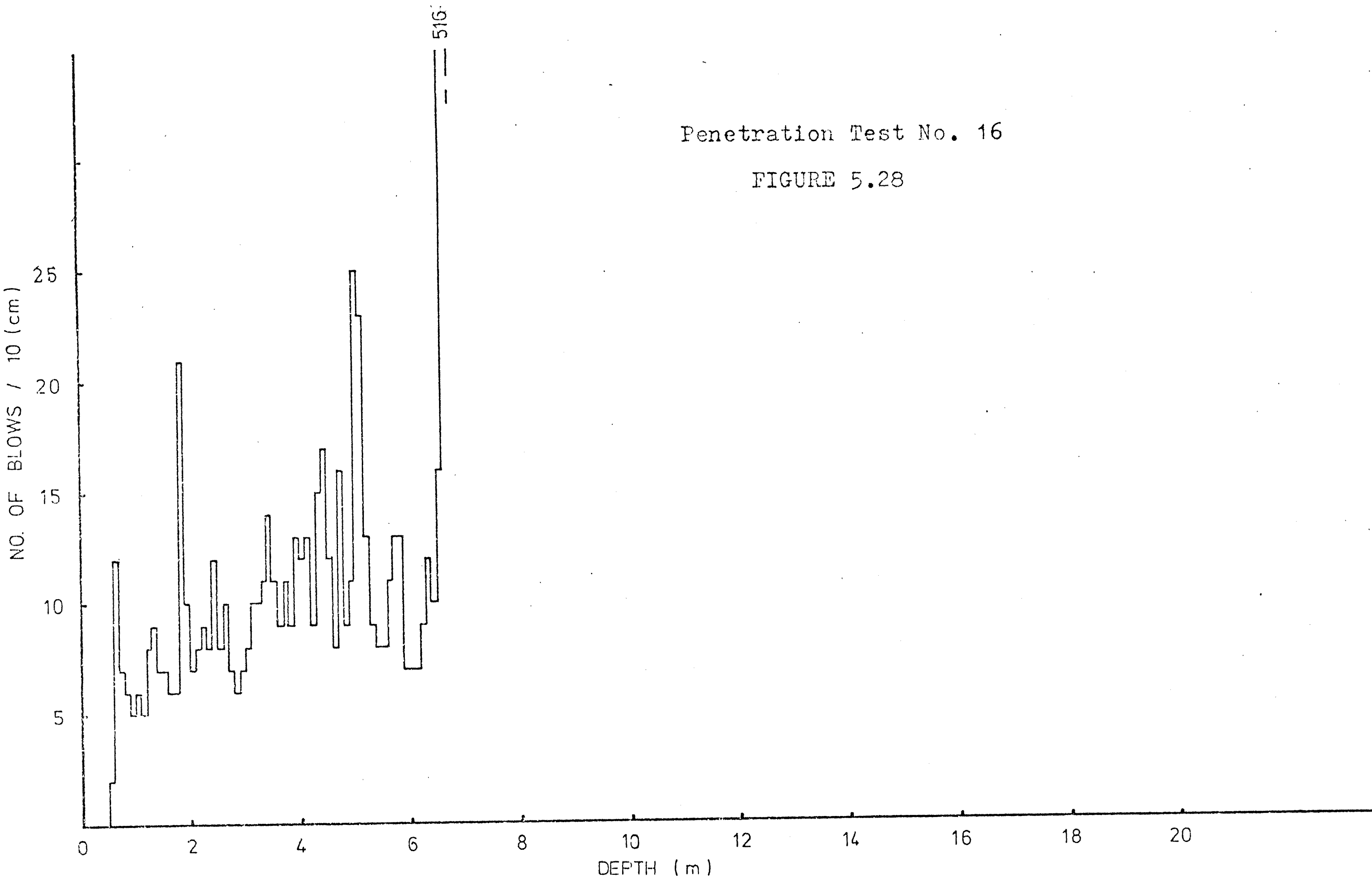
Penetration Test No. 3

FIGURE 5.26

Penetration Test No. 15

FIGURE 5.27





Penetration Test No. 16

FIGURE 5.28

to observe the effectiveness of the winches in keeping the raft stable. It was found that a penetration test could be performed while the raft was afloat, with no major problems occurring in the equipment. However, due to the rise and fall of the tide, the winches required periodic adjustment to keep the raft centred over the hole. The results of test 4 are shown in Figure 5.13 and its location is indicated on the site plan in Figure 5.12.

One test in this series was executed while the raft was stranded on a coral outcrop within the lagoon. This was to determine if any variation in soundness existed between the coral outcrops and the floor of the lagoon. The results of test 5 are shown in Figure 5.14.

By using a floating technique, it was possible to do two soundings in the same area, by moving the raft a few metres using the winches. This allowed more than one test to be achieved on any particular site, where previous procedures allowed only one test to be performed. The remaining four tests in this series were carried out in pairs, and the results of these tests (6-9) are shown in Figures 5.15-5.18.

5.3.4 Second Series of Test

This series was performed over five working days in August, 1979. A total of ten soundings were attempted of which seven were completed to an average depth of 20m, and three more were abandoned due to technical problems.

The first five tests were performed within an area of approximately nine square metres, to carry out a sensitivity test. The tests were performed in a small pocket of water 2-3m deep which permitted testing to continue throughout a whole day. Two and three tests were performed over two consecutive working days. The results of tests 10-14 are shown in Figures 5.19 - 5.23, while the location is shown on the site plan, Figure 5.12.

In September, 1978, three penetration tests were carried out by H. Bock, on the seaward edge of the reef. The results of tests 1-3 are shown in Figures 5.24 - 5.26. As the depth of penetration was only to 14m, a further test was planned to a depth of 20m.

Two tests were attempted, but both were abandoned due to malfunctioning of the penetrometer, which resulted in several metres of sounding rods being damaged. As the problems were caused by the prevailing sea conditions, it was futile to attempt further tests and the raft was motored to a large bommie on the leeward side of the reef.

The raft was allowed to settle onto the bommie and a test started. At a very shallow depth severe bends occurred in the sounding rods and the test was discontinued. This was caused by large cavities in the coral deflecting the rods off course, when the penetration tip encountered the side of a hole. The penetrometer was then rotated 180° and another test attempted approximately 1.5m from the first. However, the same problem occurred and no further tests were performed on bommies on the outer leeward edge of the reef.

Of the two tests completed at the end of this series, one was performed on the extreme seaward edge and the other approximately 500m from the leeward side of the reef. The results of test 15 and 16 are shown in Figures 5.27 and 5.28 respectively. Test 15 was performed while the raft was stranded during the extremely low tides (-.1m) that were experienced at the time.

The results of all the tests carried out on Keeper Reef, have been plotted as a graph of the number of blows required to penetrate 10cm against depth. The tests have been numbered in a chronological order and their respective positions plotted on a site plan given in Figure 5.12. The raw data collected from the penetration tests is given in Appendix C.

The actual location of each test was determined by means of the research vessels radar. Bearings and distances (accuracy of $\pm$.01 of a nautical mile) were obtained of the raft and radar beacon with respect to the vessels position. From these values, the test locations were plotted on a site plan relative to the radar beacon.

When plotting the test results, the datum used is a calculated value of the tidal datum. While performing a test, the starting and finishing times along with the corresponding depth of water were recorded. With these values and times and heights given in the Tide Tables, it is possible to approximate the tidal datum from calculations assuming a sinisoidal tidal variation. The depth of water below the datum can then be obtained for each test, allowing all the results to be plotted with respects to the same datum.

The locations of the thirteen tests performed, were as close as possible to a line with an approximate bearing of 165^{0} with respect to the radar beacon. The tests were performed along this section so as any changes in the reefs composition may be noted.

5.3.5 Testing Limitations

Because of a combination of human, technical and geological factors, the amount of testing that can be achieved for any given period, is far from predictable.

Due to the reef topography, it is not possible to manoeuvre the raft within the reef's confines, other than at high tide. Thus, once a test is completed another cannot be started at any great distance from the present position, until sufficient water returns to allow repositioning. However, this situation was aggravated by the fact that testing was carried out while the lowest tides for the year were being experienced. This could be improved if testing was performed during the neap tides, where the rise and fall is minimal.

The sea conditions over the test site can sometimes cause the malfunctioning of equipment. If the movement of the raft is excessive, the endless chain can catch on the percussion mechanism and prematurely lifts it to the top of the mast. This then violently hits the rods when it falls back, sometimes causing damage to the sounding rods and other equipment.

Testing was successfully performed in water 3m deep, and no excessive deflection of the unsupported sounding rods was noticed. The greatest depth of water that testing can be performed in, will have to be determined by trial and error.

Another factor limiting testing procedures is the physical endurance of the personnel involved with the operations. Although a test can be achieved within two hours, it has been found that three tests a day is very tiring. This is due to being totally exposed to the environment while working on the reef for several hours.

5.3.6 Test Results

By closely examining the results of any test, it is clear that the penetrometer can detect the geomechanic constituents of the coral quite precisely. The basic units of interest from an engineering point of view, are the various strata, deposits of sand and weak spots.

If test 14 (Figure 5.23) is used as an example, it is clear that different stratum and weak spots within the coral's composition can be detected. At the approximate depths of 5, 15 and 18m, distinct examples of holes or weak material that exists within the coral are shown. Basic layer formations are shown to exist at depths of 3-10, 10-17, 18-21 and 22 metres and have corresponding approximate N values of 3, 7, 10 and >50 respectively. Thus, from test results, it is possible to clearly record the composition of the subsurface at the particular test site.

However, from the results obtained from a test hole, it could not be assumed that the same conditions exist over a large area, due to the way that coral reefs are formed. To determine the variation of formations with distance, five tests were performed within a small area to determine the sensitivity of the coral limestone.

5.3.6.1 Sensitivity Test

From the sensitivity test (tests 10-14) some distinct geological features have been reproduced, but their respective depths and soundness varies considerably over an area of approximately nine square metres.

Five of the most noticeable features detected by the tests have been listed below.

1) A very weak layer exists within the depth range of 3-5 metres and has an approximate average N value of 0-1.
2) For depths below 10m, an increase in the average penetration resistance occurs.
3) Around the depths of 14 and 18m a general decrease in the average penetration resistance exists.
4) A very distinct geological discontinuity is clearly shown in all five tests at an approximate depth of 22 metres.
5) The discontinuity (4) is accompanied by a noticeable drop in the penetration resistance for one metre prior to the discontinuity.

For the five tests, only points 1, 4 and 5 are reproduced with any degree of accuracy for all the tests. Other noticeable features vary considerably for the tests within a space of 3 metres. Thus, for tests within a small area, it could be assumed that major geological features lie within a $\pm 20\%$ range, both for depth and N values.

However, geological features that exist on one section of the reef differ from those determined 500m away. By inspecting the tests performed along the transect bearing 165^{o}, several distinct variations in coral development have been noticed. The four most noticeable variations have been listed below.

1) The surface hardness increases from the seaward edge $N_{15} = 10)^{*}$ to the top of the reef flats ($N_1 = 60$) and then gradually decreases ($N_3 = 10$, $N_2 = 5$) and stabilizes on the lagoon floor ($N_5 = 5$, $N_8 = 3$).
2) A distinct discontinuity occurs at varying depths across the reef. Shallower on the seaward edge than in the lagoon.
3) For over 500m within the lagoon region, two general decreases in penetration resistances have been recorded at approximate depths of 6 and 13m ($\pm$1m) respectively.
4) The general penetration resistance within the reef flats, suffers very large variations over short distances (20m).

As very few tests were completed on the leeward edge of the reef, a complete picture of the underground geology cannot be made. However, the geology of the seaward edge has been investigated further in Chapter 7.

As the results of sixteen tests have been collected, it is possible to correlate the relative density of the limestone with its corresponding N values. Thus, the relative density can be scaled over the range of penetration resistance values obtained from the tests. However, material with N values greater than 50 have all been classified as extremely hard, although the hardest material sounded required 516 blows to penetrate approximately 1cm. Given in Table 5.29 is the physical properties of the coral limestone with

* Subscript indicates test numbers.

its corresponding penetration resistance.

TABLE 5.29

N blows/10cm	Relative Density of Coral Limestone	Possible Material
0-2	Very weak (VW)	Holes - sand
2-5	Soft (S)	Loose-porous coral
5-10	Firm (F)	Soft coral limestone
10-20	Hard (H)	Hard coral deposits
20-50	Very Hard (VH)	Hard to cemented
>50	Extremely hard (EH)	Cemented Limestone

5.3.7 Errors

Several errors have been found to exist in the recording procedures used for performing some of the thirteen tests in June and August. The errors are a direct result of the relative movement of the raft while subject to tide, waves and wind. The cause of the largest errors in the recorded values, is the resultant rise or fall in the tide that occurs while testing is carried out.

As the arbitrary point chosen to record the depth of penetration was fixed to the raft, values recorded do not account for the effects of the tide.

If a test was performed during a falling tide, a resultant error equal to the fall in height of the tide would occur. Thus, if a 0.5m fall in the tide was experienced, then the depth of the sounding rods would be 0.5m deeper than that recorded. Also the number of blows recorded to penetrate 10cm would also be higher than the actual penetration resistance of the material being penetrated. This is due to the extra length of rod that has to be sounded into the sub-surface for each 10cm recorded.

Similarly, a reverse situation occurs when a test is recorded on a rising tide. This results in the recorded depth being deeper than the actual case, and N values becoming progressively under-estimated with depth.

As a test continues, the error occurring from the tide, progressively increases as the depth of penetration increases. This results in the error being largest at the end of a test, and zero at the beginning. However, the variation is not linear, due to the different rates of penetration that occurs with depth.

To observe the effect that this error can have on tests, it is possible to correct the depth of the discontinuity (N>50) detected in the sensitivity test. As testing was stopped at the discontinuity, the tide correction is equal to the tidal variation for each test. Thus by adding or subtracting the appropriate tidal variation to the depth of the discontinuity, the recorded depths can be corrected. Shown in Table 5.30 are the recorded and corrected depths to the beginning of the discontinuity.

TABLE 5.30

Test	Recorded Depth	Tidal Variation	Corrected Depth
10	21.2m	-0.9m*	22.1
11	22.2	+0.4	21.8
12	21.8	-0.8	22.6
13	21.7	-0.1	21.8
14	22.6	+0.3	22.3
	$\bar{D}$=21.9m		$\bar{D}_c$=22.1

* Negative denotes a falling tide.

As can be seen, the average depth to the discontinuity has not been affected greatly, $\bar{D}$ = 21.9 and $\bar{D}_c$ = 22.1m. However, the range of depths has been decreased by 0.6m, giving a total range of 0.8m compared with 1.4m before correction.

Thus the correction for the tide can account for a major part of the variation of depths, that occurs for some geological features. Although it would be possible to correct a complete test for depths and N values using a computer program, it would be far more advantageous to prevent the errors than to correct for them.

When recording the number of blows required to penetrate a 10cm section, errors occur due to the fluctuation of the raft from wave effects. If the rate of penetration is slow, difficulties arise in defining the cut off point for each 10cm sections, if the raft is fluctuating by 2cm. Therefore the errors in the N values increases as the rate of penetration decreases.

Errors also occur in calculating the tidal datum which was used as the datum for plotting the test results. For calculating the datum the times and height for Townsville's tides have been used, assuming the turn of tide occurs 20 minutes earlier on Keeper Reef. As the tidal data for Keeper Reef is unknown, the size of the errors arising from this assumption cannot be calculated.

To overcome the errors caused by the relative movement of the raft, a datum fixed to the reef surface would have to be used for all recording purposes. One method of producing a fixed reference point, is to use a length of sounding rod embedded into the surface, until it becomes stable. Once a reference rod has been established the raft could be shifted aside, until sufficient room is available for the operation of the equipment. A test could then be performed, using the measuring grooves on the reference rod as the datum for recording purposes. The reference rod could be retrieved in the normal way, after completion of the test.

By using calibrated sounding rods, no errors can occur when recording the depth and corresponding penetration resistance. An accurate measure of the depth of water at the start and finish of each test, can also be obtained using this method.

To decrease the errors created in calculating a datum, a semi-permanent tide gauge in the form of a calibrated rod, could be established on the reef prior to a series of tests. By taking a reading from the tide gauge at the end of each test, a series of tests could be related accurately to any chosen datum. For future purposes, a permanent mark could be installed on the reef, so as the datum used for each series of tests can be related to a known height.

5.3.8 Comments and Recommendations

Testing operations showed that penetration testing while afloat, is a very feasible technique for determining the geomechanical units (coral limestone, sand and holes) of coral reefs. Though there are a few geological and technical problems which restricts the technique, forward planning can overcome or moderate these restrictions.

For future testing series, the trips should be planned to align with the neap tides, instead of the extremely low tides. This will allow far more flexibility into the testing procedure, as testing for a day will not be restricted to the one position, as in previous cases.

The introduction of a fixed reference and a tide gauge into future testing procedures, would allow a higher degree of accuracy to be achieved in the test results.

In carrying out site investigation work, a very thorough grid of tests would be required to obtain a clear indication of subsurface formations, over a large area. If the location of proposed foundations are known, spot checks could readily be performed, saving time and money.

CHAPTER 6 - CORRELATION OF TEST RESULTS

6.1 INTRODUCTION

As three site investigation techniques were tested, the results can be compared enabling the degree of correlation between methods to be ascertained. However, comparisons between the drilling-penetration and seismic-penetration tests can only be made, as information derived from the drilling tests is inadequate to allow comparison with the seismic spreads.

6.2 DRILLING AND PENETRATION

As the monitored drilling test (test 2) was performed close to the site of a penetration test (test 1), results can be readily compared. The results of both tests are shown in Figure 6.1 and 6.2 respectively. Three distinct sections or layers have been detected by both tests at approximately the same depths. The depth of the layers are shown in Table 6.3.

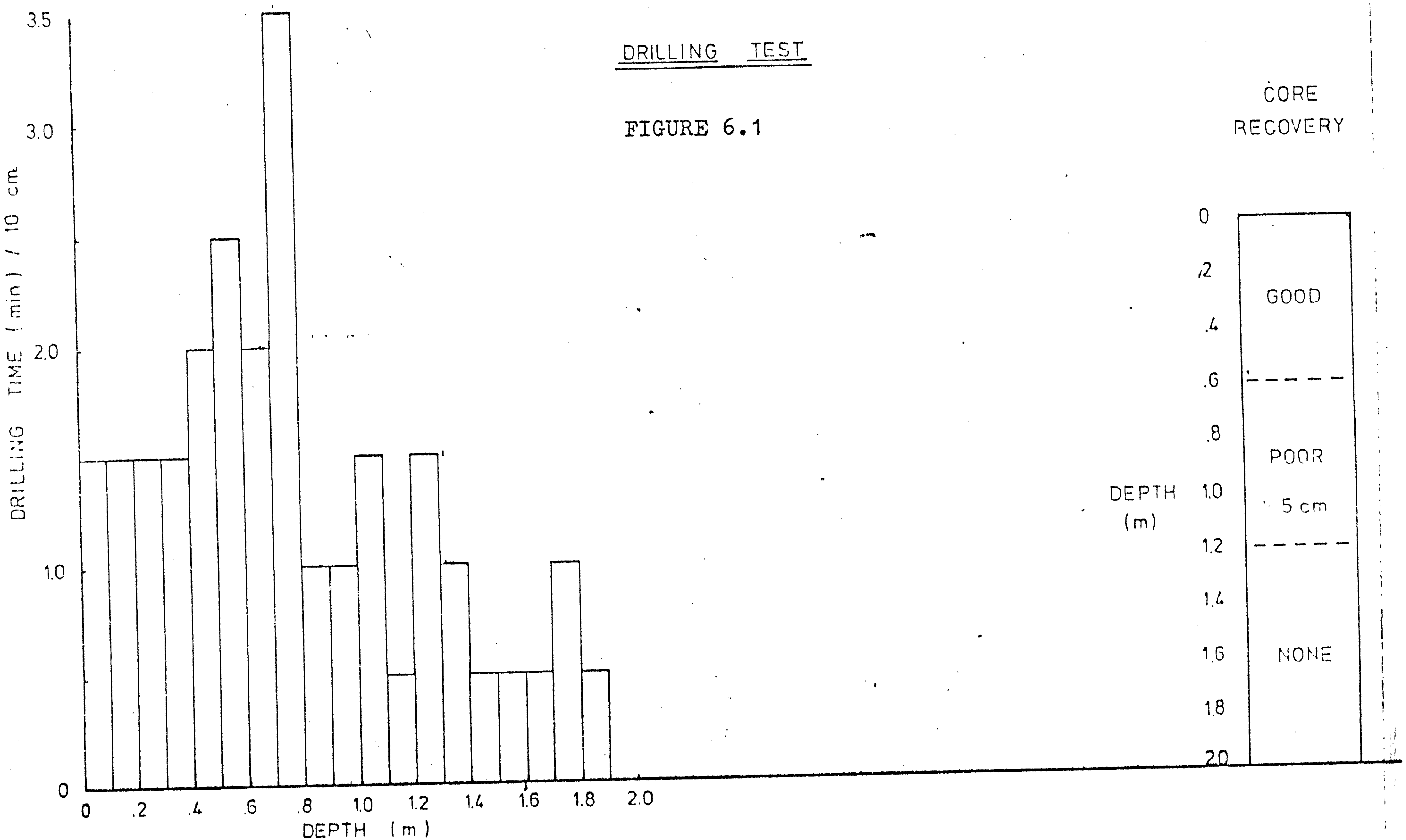
TABLE 6.3

LAYER	DRILLING DEPTH OF PENETRATION	PENETROMETER DEPTH OF INTERFACE
1	-	-
2	0.8m	0.7m
3	1.4m	1.4m

Although the general layers have been detected quite accurately by both tests, the soundness for individual 10cm sections varies considerably. The differences could be accounted for by either one or two factors:

1) The hardness of the limestone is different between both test sites.

2) A constant pressure was not maintained on the cutting face, which would have caused variations in the drilling rate.



DRILLING TEST

FIGURE 6.1

PENETRATION TEST

FIGURE 6.2

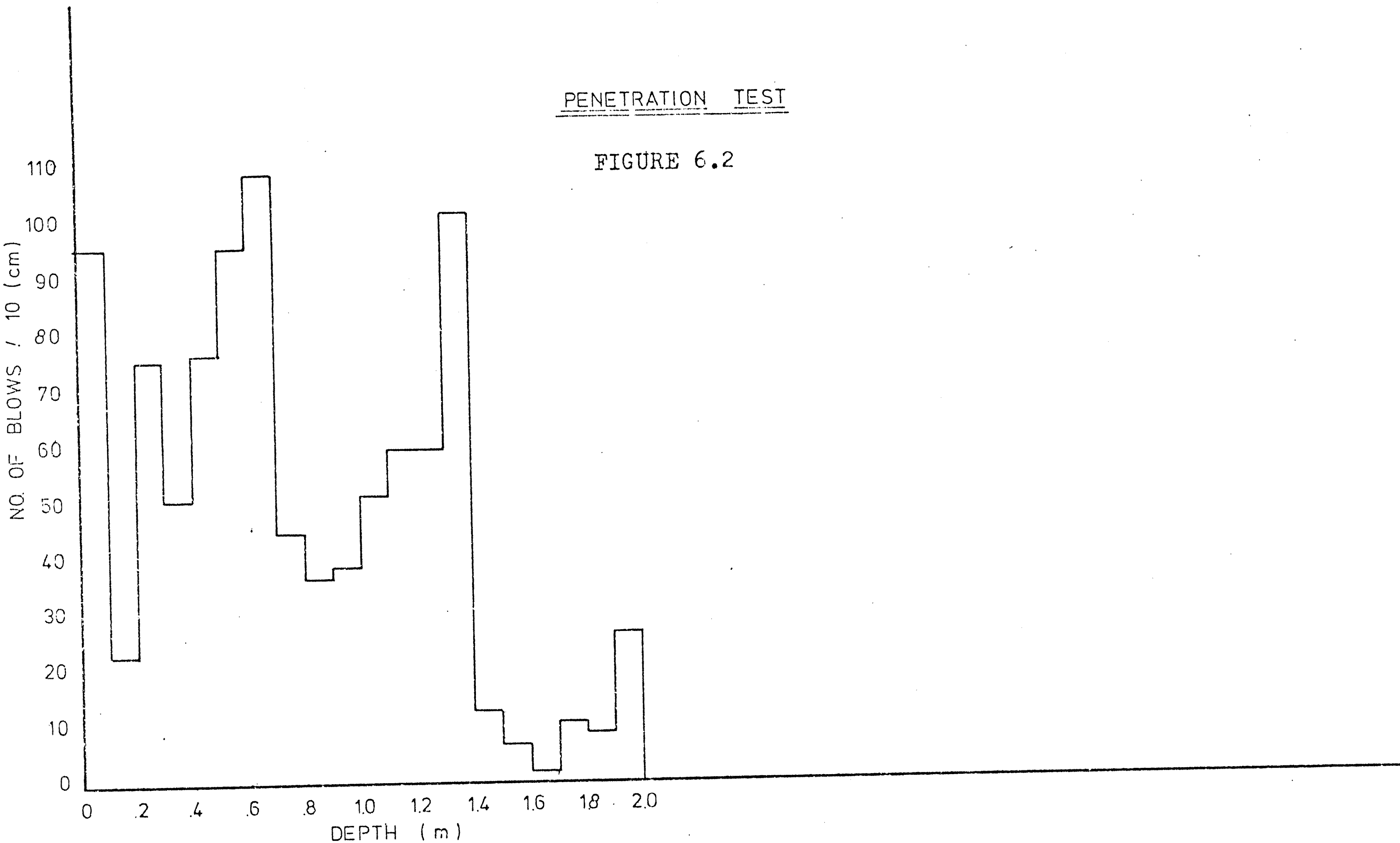
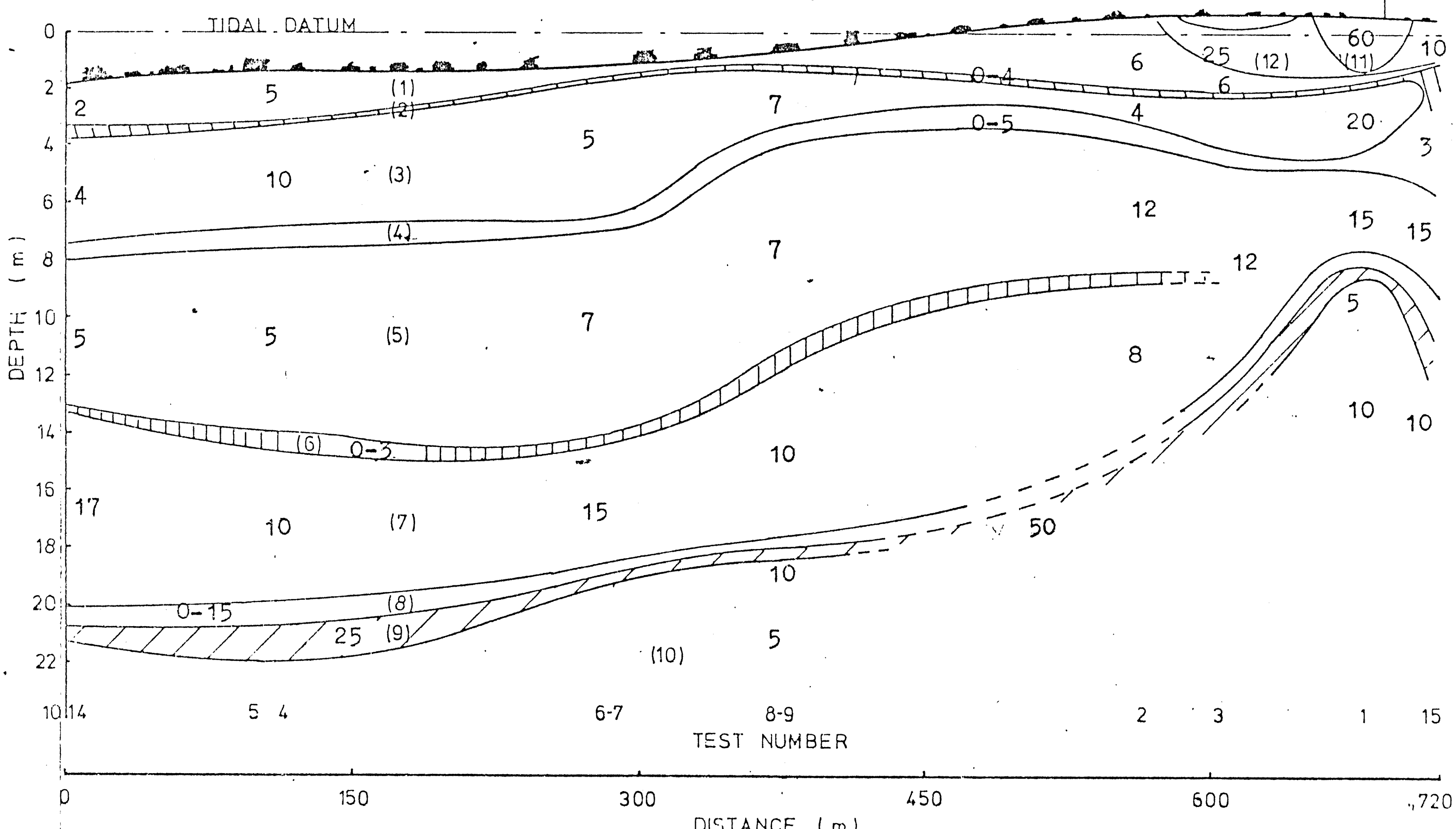


FIGURE 6.4


TIDAL DATUM
DEPTH (m)
0
2
4
6
8
10
12
14
16
18
20
22
(1)
(2)
(3)
(4)
(5)
(6)
(7)
(8)
(9)
(10)
(11)
(12)
0-4
0-5
0-3
0-15
50
60
25
20
TEST NUMBER
10 14
5 4
6-7
8-9
2
3
1
15
DISTANCE (m)
0
150
300
450
600
720

Even though differences do exist between sections, the relative hardness of each layer can still be determined from both tests.

As an insignificant amount of drilling was performed, it is not possible to ascertain whether the monitoring of drilling tests, is a feasible site investigation technique.

6.3 SEISMOGRAPH AND PENETROMETER

To readily compare the depth of subsurface formations determined by the seismograph with those derived from penetration tests, a profile of various disconformities detected by the penetrometer have been plotted.

By assuming all the penetration tests lie along the same longitudinal section, the depth of distinct features can be plotted and contoured. This allows underground formations to be visualized easier than a series of graphs can portray.

The following list of disconformities have been plotted and contoured for fifteen penetration tests carried out on the seaward edge of the reef over a 720m section.

1) Layers of generally weak material

2) A layer of extremely hard material (N>50)

3) Zones of isolated material of relatively high resistances

The profile of this section is shown in Figure 6.4.

The results of only four of the five seismic spreads performed on the reef will be considered, as gross errors are known to exist in the calculations of depths for run IV.

For shallow formations, the calculated depth of a stratum, is the depth of the layer at an offset distance from the start of a run. Thus the results of the nearest penetration test to the beginning of a seismic run, will be used for comparison purposes. Therefore the results of penetration tests 3, 1, 1 and 6-7 will be compared with the depths of formations derived from seismic surveys I, II, III and V respectively.

The depth of the lower stratum determined by the seismic spreads, have been tabulated along the depths of the corresponding formations detected by the penetrometer (Table 6.5).

TABLE 6.5

SEISMIC SURVEYS		PENETRATION TESTS		VARIATION IN DEPTHS %
RUN NO.	DEPTH OF LOWER STRATUM	TEST NO.	DEPTH OF CLOSEST LAYER	
I	6.0	3	5.4	10
II	5.0	1	5.7	12.3
III	5.9	1	5.7	3.4
V	5.1	6-7	5.8	12

From the table, a maximum variation of 12.3% exists between the depths determined by the seismograph and penetrometer for the same formation. However, when considering the error of the seismograph ($\pm$10%) and those in the results of a penetration test (tide, datum etc.) a 12.3% error is quite acceptable.

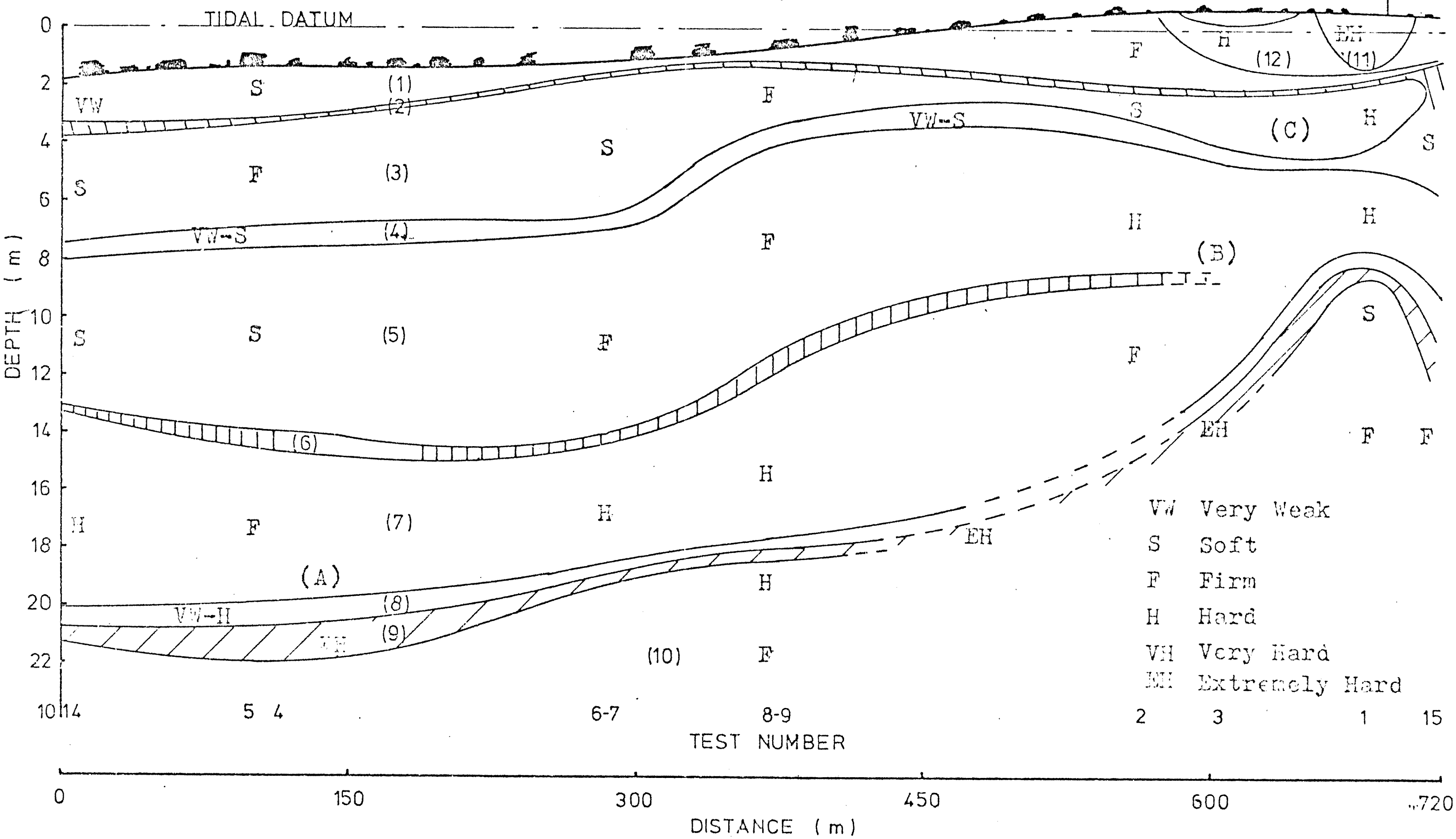
6.4 COMMENTS AND CONCLUSIONS

Although the results of the monitored drilling test compared closely to those of the corresponding penetration test, the degree of correlation cannot be estimated as an insignificant number of drilling tests were performed.

From the comparison between the penetrometer and seismograph, the results show a possible 12.3% variation in depths for shallow formations. As no results for deeper formations were obtained from the seismograph, comparisons cannot be made for deeper strata.

The variations that have occurred may possibly be decreased, if the testing procedure for penetration tests are modified as recommended in Chapter 5.

FIGURE 7.1


TIDAL DATUM
DEPTH (m)
0
2
4
6
8
10
12
14
16
18
20
22
VW
S
F
H
EH
VW-S
VW-H
(A)
(B)
(C)
(1)
(2)
(3)
(4)
(5)
(6)
(7)
(8)
(9)
(10)
(11)
(12)
10 14
5 4
6-7
8-9
2
3
1
15
TEST NUMBER
0
150
300
450
600
720
DISTANCE (m)
VW Very Weak
S Soft
F Firm
H Hard
VH Very Hard
EH Extremely Hard

CHAPTER 7 - GEOLOGY OF KEEPER REEF

7.1 INTRODUCTION

When referring to the geology of Keeper Reef, only a 720m section of a transect, bearing 165^{0} with respects to the radar beacon on the seaward edge will be considered. Only formations above an average depth of 20m can be considered, as testing was limited to approximately this depth.

7.2 BASIC GEOLOGY

The type of material within this 20m section cannot be specifically classified, as very little core recovery was obtained throughout the section. Thus the coral limestone can only be described in terms of its relative density, which is directly proportional to its penetration resistance.

Therefore, the results from the penetration tests will be used to determine the basic geology in this section, due to the significant amount of testing performed along the transect.

Using Table 5.29 (Correlation of penetration resistance with the physical properties of coral limestone) in conjunction with Figure 6.4 (profile of distinct geological features along the transect) the general relative density of formations to a depth of 20m along the transect can be outlined as shown in Figure 7.1.

The characteristics that have been indicated on the profile are the distinct geological features that were detected over the section. Although fifteen tests were achieved it is not possible to clearly plot the general geology in some regions, due to the distances between tests. Within the areas marked (A), (B) and (C) in Figure 7.1, some doubt does exist to the limits of the various geological formations.

In region (A), the lower disconformity (9) has been detected with a relatively lower penetration resistance (N = 25) than elsewhere (N>50). Either the actual disconformity is located at a deeper depth and has not been sounded, or the relative hardness in this region is less than normal. However, as the lower layer (9) is accompanied by a weak layer (8), it is probable that it is softer in this region. This is based on the observation that a weak layer exists on top of the extremely hard disconformity throughout the section.

The actual thickness of the lower disconformity (9) has not been ascertained in different positions, as most tests were discontinued once this layer had been located.

In area (B) the weak layer (6) has not been continued as its position is not clearly defined in tests 1 to 3. This is due to large variations in the general hardness and depth of formations that occur from test to test within this region.

Within zone (C) the limits of the underground formations (11), (12) and (3) have only been estimated, as insufficient information is available to allow accurate contouring. In test 1, a 2m layer of very hard limestone has been detected with an average N value of 60. However, a distance of 33m away, test 15 showed the near surface layer as having an average N value of 10. Thus the boundary of these formations can only be found by further testing within this section.

However, from the information available a few general characteristics of the reefs geology can be obtained. These are listed below.

1) The coral growth on the seaward edge of the reef is developed more than the lagoon region.
2) The seaward edge is approximately 2m higher than the lagoon floor.
3) The coral limestone is harder for the first 10m on the seaward edge than in the centre of the lagoon.

4) A general increase in hardness occurs with depth, more so beneath the lagoon than the seaward edge.

5) Several thin weaker layers exist at various depths throughout the reef.

6) An extremely hard layer occurs between a depth of 8-22m across the section.

7) Disconformities occur at deeper depths beneath the lagoon than under the seaward edge.

From the section tested, the seaward edge is much harder at shallow depths (0-10m) than the lagoon. Therefore, it would provide the most favourable foundation site on the reef. However, structures erected on the seaward edge would be subject to the full force of the prevailing seas.

As all testing was performed on one side of the reef, very little is known of the geology on the leeward side. Thus, further testing is required on different sectors to determine the full potential of the reef in respects to favourable foundation sites.

7.3 COMMENTS AND CONCLUSIONS

The general geology of the reef is very complex, and distinct geological features vary considerably for different sections of the reef. The seaward edge is the most complex and developed area of the reef. Further testing is required on the leeward side, to obtain a complete profile of geological features.

To detail the geology of the whole reef, further testing is required along several longitudinal and cross sections of the reef. Extensive testing would be required on the reef flats, due to the complex coral development in this region.

CHAPTER 8 - CONCLUSIONS AND RECOMMENDATIONS

8.1 CONCLUSIONS

Of the three site investigation techniques tested, the penetrometer is the most feasible method for determining the engineering properties of coral reefs. A penetration test can detect all the basic geomechanical units of coral reefs to a depth of 25m.

Single channel portable seismographs are not capable of detecting all the engineering properties of coral reefs. Basic layer formations can be recorded, but holes and deposits of weak coral and sand cannot be detected. However, seismographs allow a fast method of determining the depth and relative firmness of layer formations.

Core drilling operations are an inefficient method of determining the engineering properties of coral reefs. The percentage of useful core samples recovered from the coral is negligible and very little data with respects to foundation properties can be derived from them. Until the percentage of useful core recovery can be improved, this type of drilling operation on coral reefs should be restricted from further use.

8.2 RECOMMENDATIONS

For future testing series or site investigation work, with the penetrometer, changes to the testing procedure as recommended in section 5.3.8 should be adhered to. This would ensure a high degree of accuracy in the test results.

Further testing could be performed on Keeper Reef, to ascertain its complete geology to a depth of 20m. This may allow favourable foundation sites to be related to the genesis of the reef. Thus, for reefs of a similar structure, promising foundation sites may be located without major site investigation work.

As a recommendation for future research, several test piles could be driven into previously sounded coral limestone and typical load test performed. Thus, the loading capacity of various piles could be related to the data obtained from penetration tests, allowing basic design criteria for piles in coral limestone to be formulated.

APPENDIX A

When the portable seismograph was initially used in an aquatic environment (May, 1977, Keeper Reef) several technical problems occurred which rendered the instrument inoperative. The main problem was the seepage of sea water into various electrical connections and switches.

As connections were open to the air, sea water was able to penetrate and cause high resistances between terminals, preventing the reopening of the triggering circuit. To overcome the problem, connections, switches and electrical cable had to be waterproofed.

From previous usage, the insulation on the original triggering cable had been damaged in several places. Therefore, the cable was replaced with 100m of two core double insulated electrical cable. Due to the thicker insulation, the new cable had more protection from abrasion than the original.

To waterproof the connections on the triggering cable, the end of each electrical plug was encased in a solid cylindrical fibreglass block. This prevented the soldered joints from coming in contact with air or water and being damaged from general handling. For simplicity, double pole single pin electrical plugs were used for all connections.

To form the fibreglass block around the connections, a piece of clear plastic tubing (18mm inside diameter) was used as the mould. By using the tubing as a mould, a similar piece of slightly reduced diameter could then be used to form a skintight casing.

To assemble the connection, the connectors were forced into the casing and insulated at each end with silicone rubber. Thus once the silicone rubber had set, a very waterproof connection was obtained. A diagram of the connections used are shown in Figure 1. The inertia switch used for triggering the seismograph for each hammer blow, is shown in Figure 2 completely insulated.

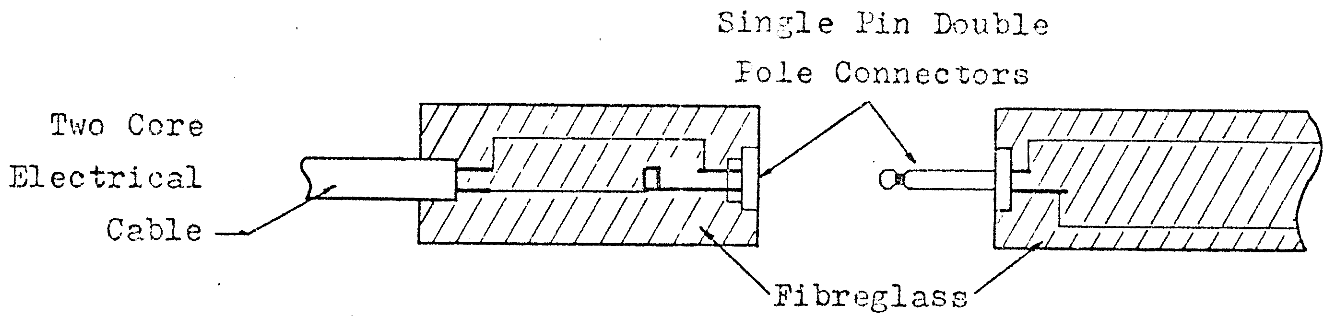
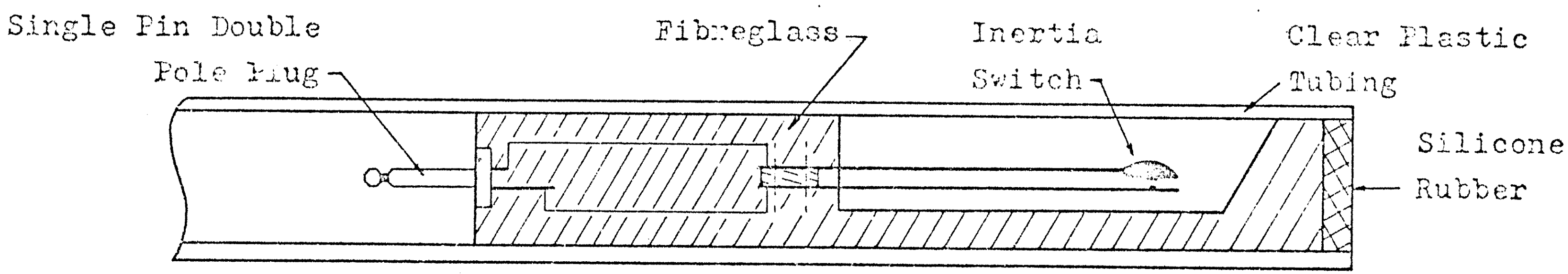


FIGURE A1



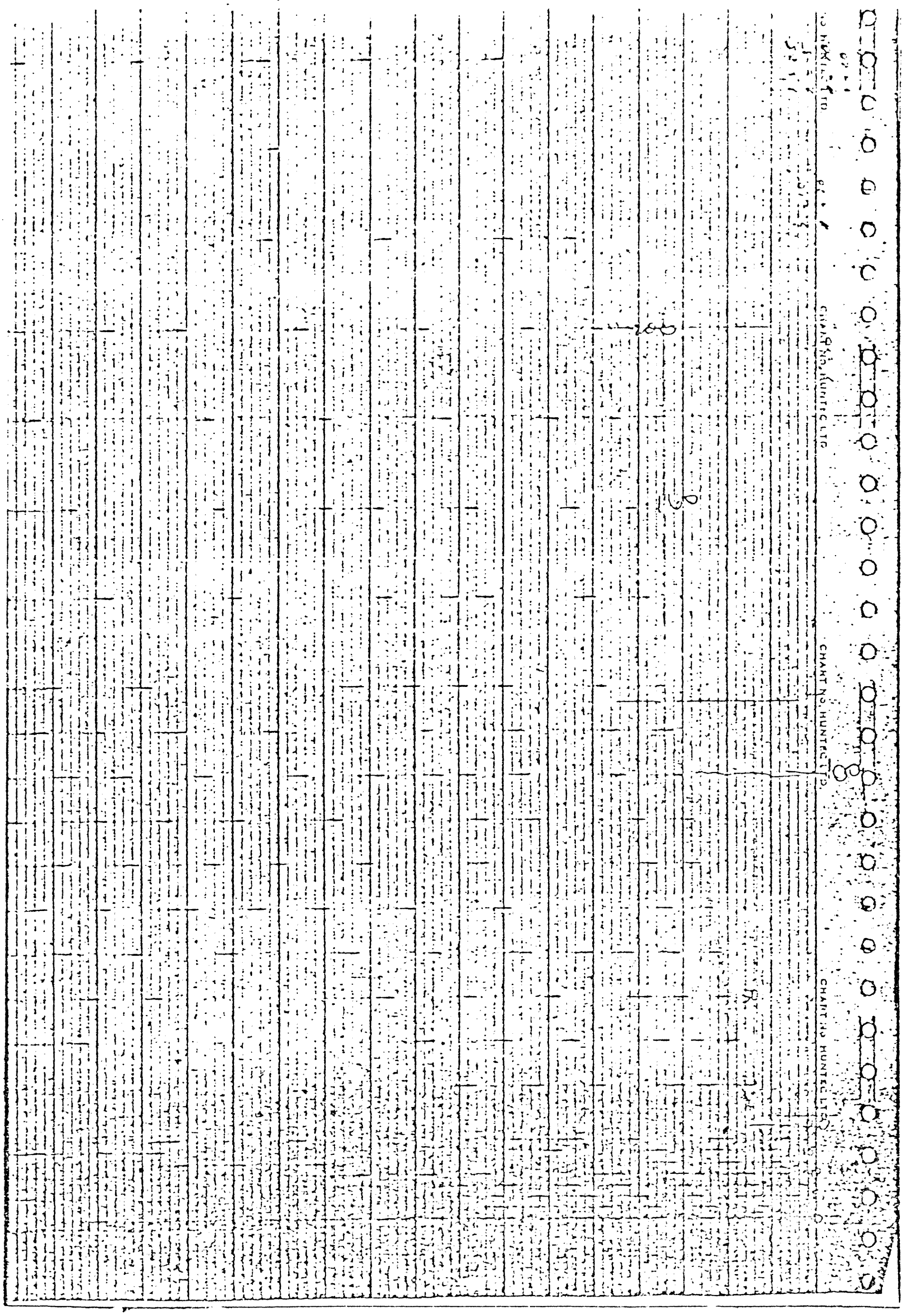
INERTIA SWITCH

FIGURE A2

APPENDIX B

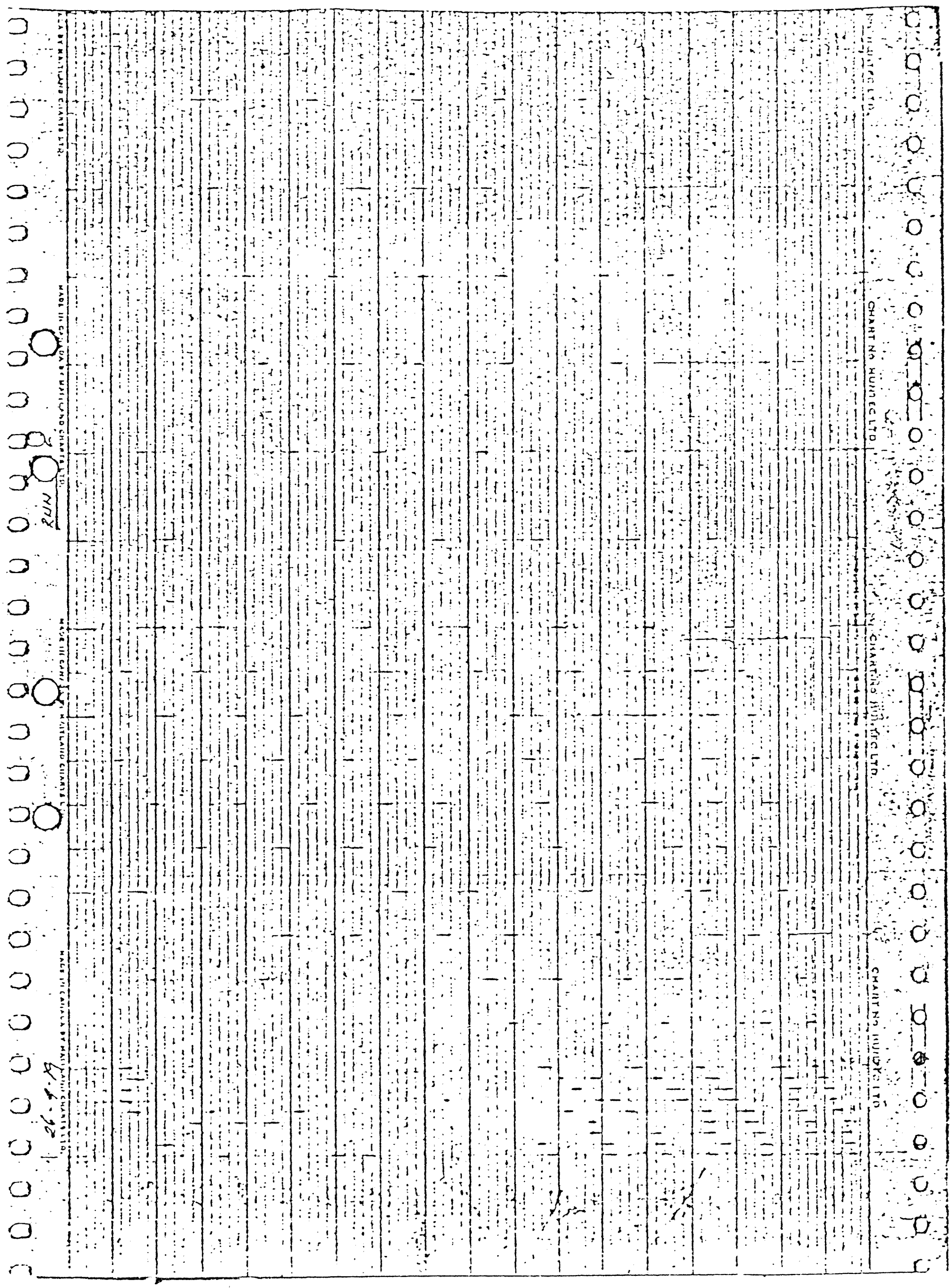
Seismic Run 1 (Douglas Campus:April)

FIGURE B1.

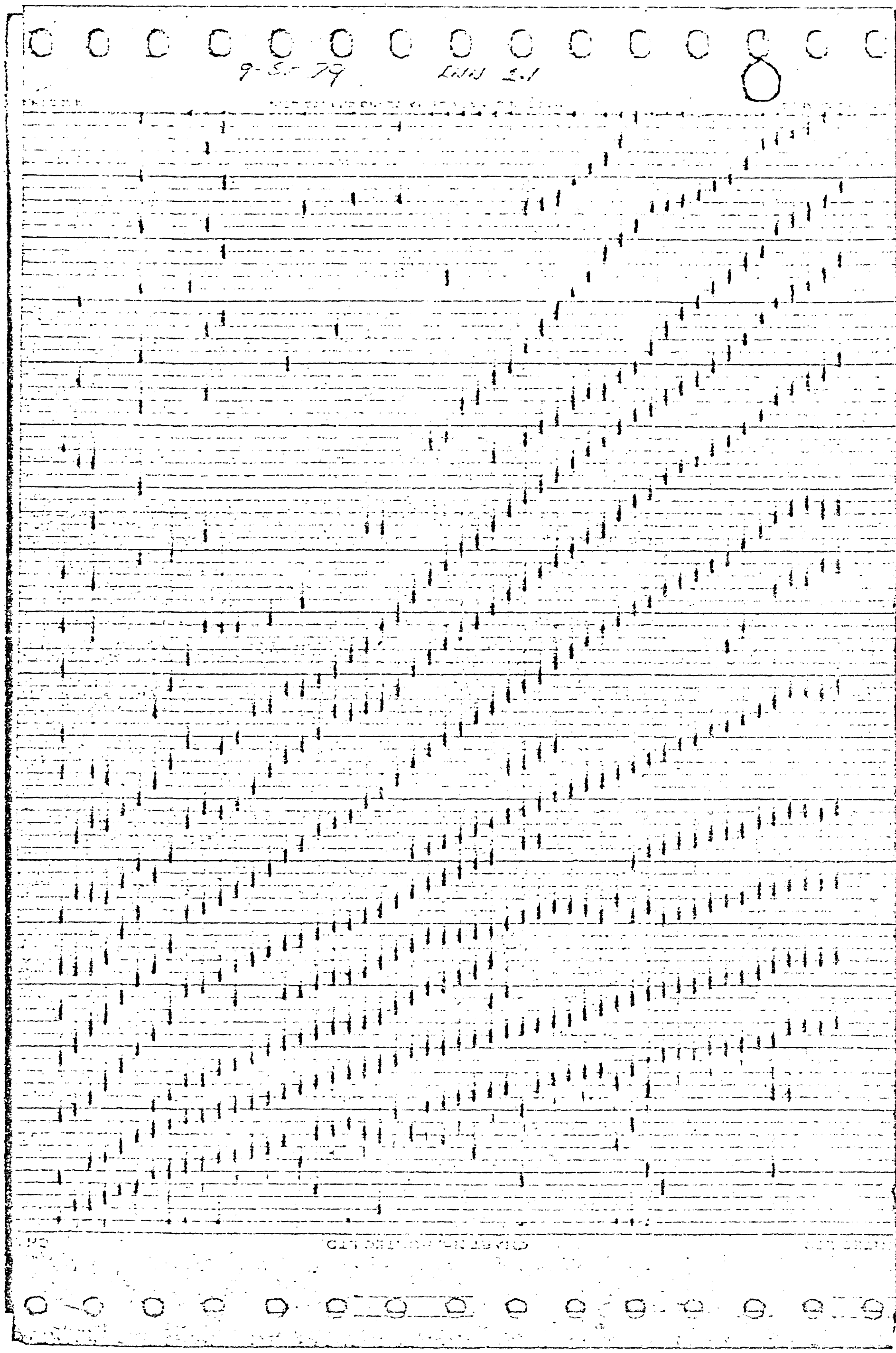


Seismic Run 2 (Douglas Campus:April)

FIGURE B2 -

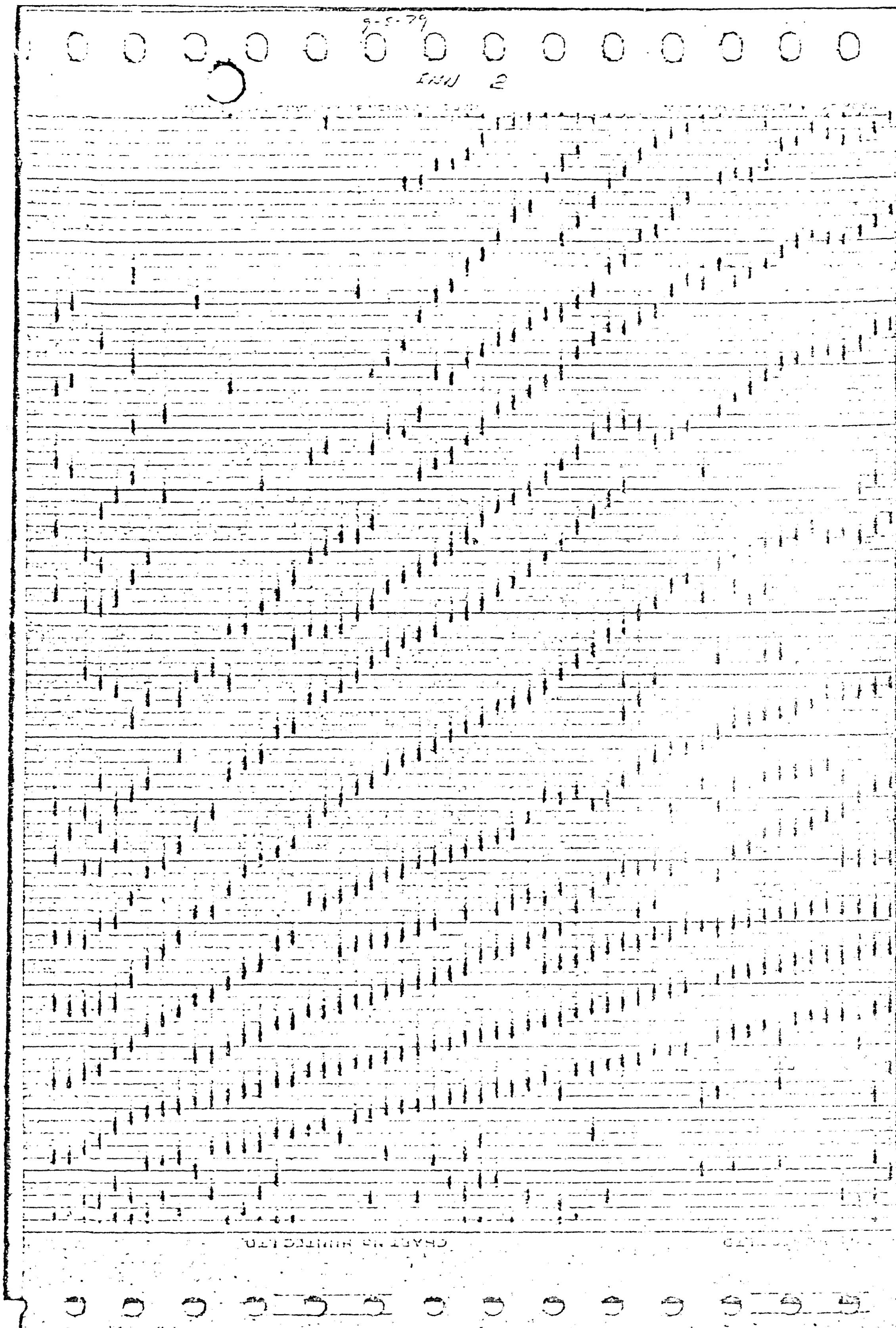


Seismic Run 1 (Douglas Campus:May)
FIGURE B3.



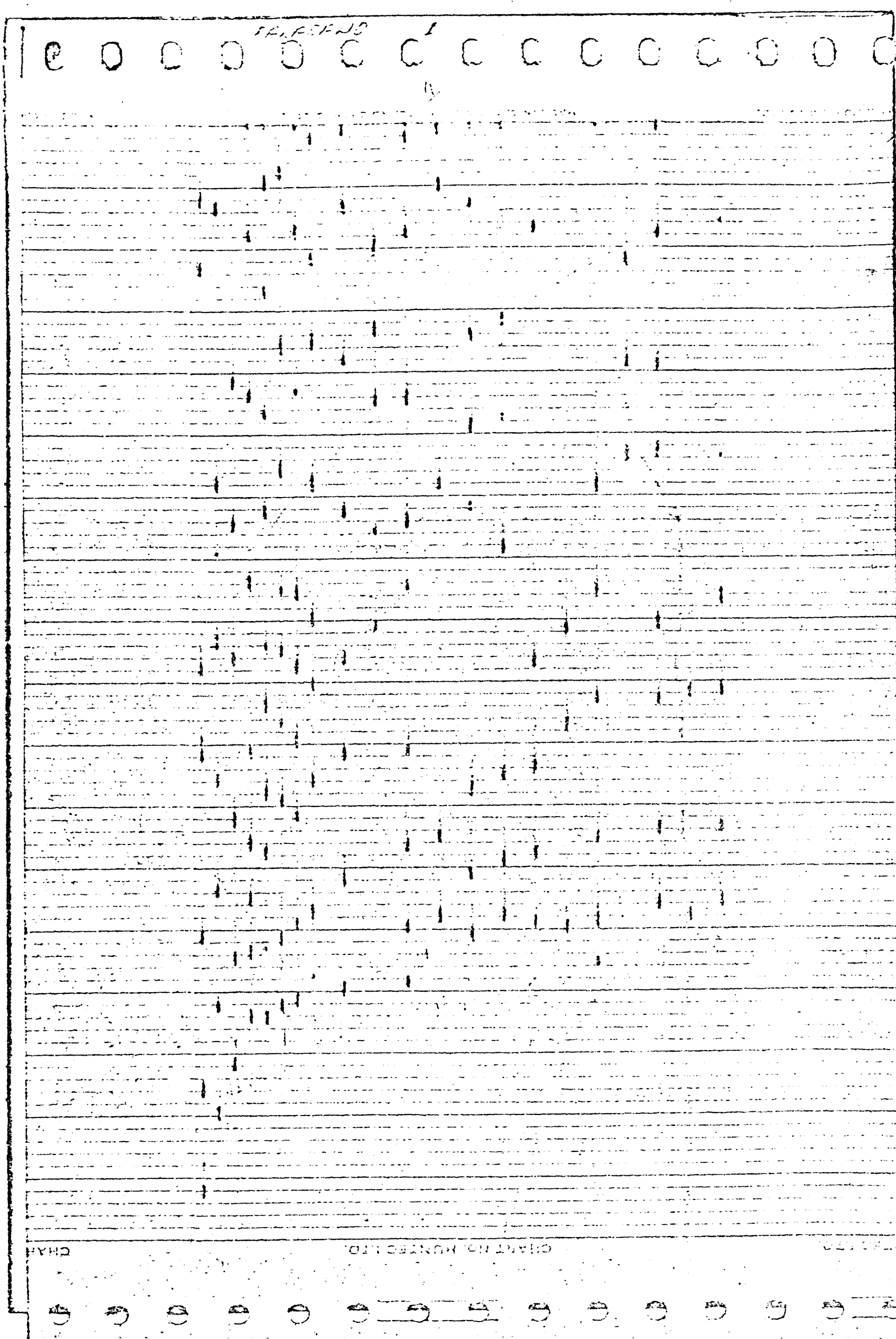
Seismic Run 2 (Douglas Campus:May)

FIGURE B4.

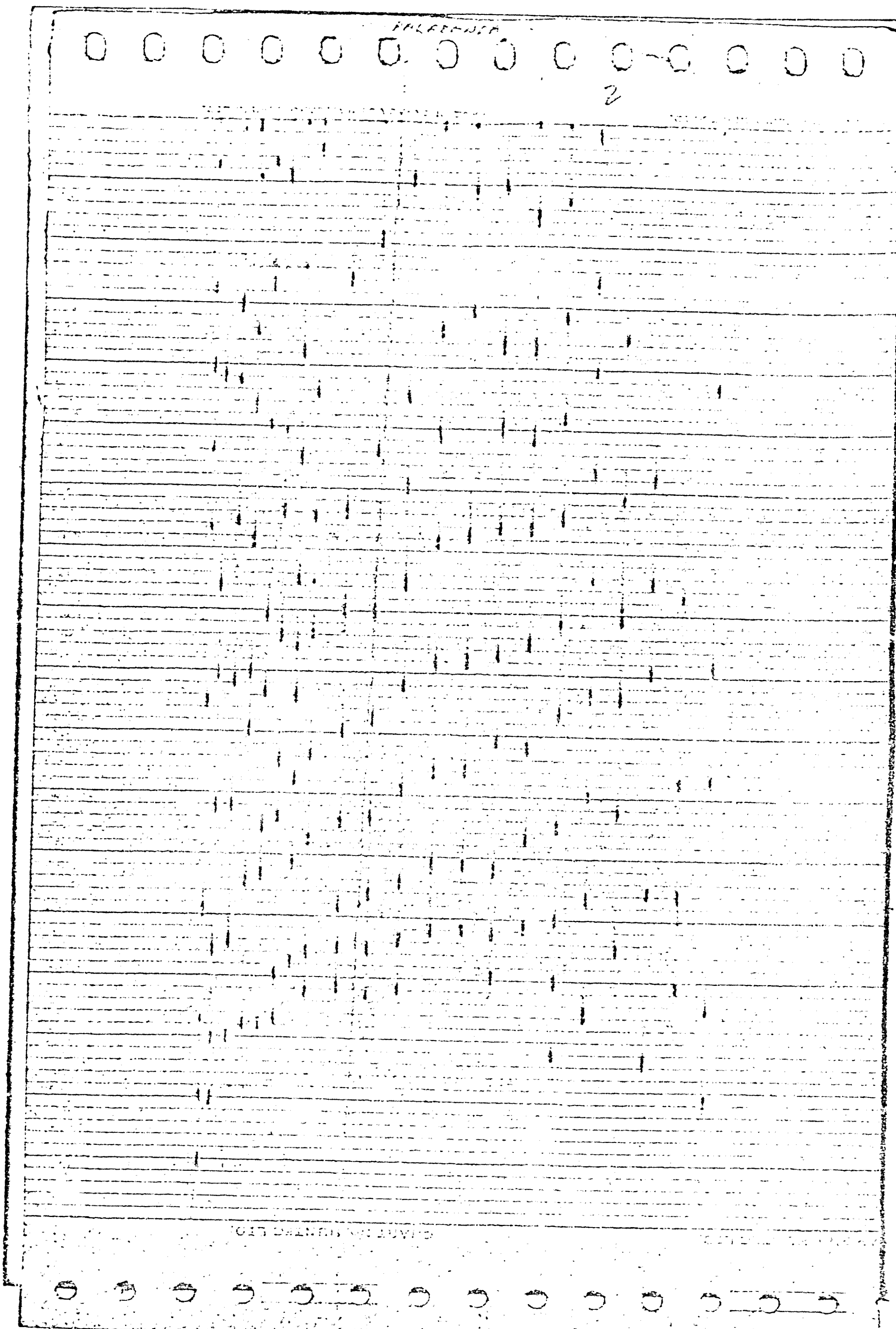


Seismic Run 1 (Pallarenda)

FIGURE B5.



Seismic Run 2 (Pallarenda)
FIGURE B6.



Seismic Run I (Keeper Reef)

FIGURE B7.

FIELD RECORD

SEISMOGRAPH TYPE

RUN NO. 1 /F

DATE 21/6/79 TIME 1:30 pm

NAME S.H.B.

LOCATION LEEWARD DIRECTION FROM T.T. NO. 6

[illegible] /

DEPTH OF WATER 1 m

[illegible]

NO. OF GEOPHONES

[illegible]

SIZE

[illegible] /

TYPE

SOIL CONDITION MOSTLY SANDY

WEATHER CONDITION 15 KNOTS FAIR AMOUNT OF SURFING BREAKING

INITIAL NOISE LEVEL 12 dB

SCALING ON PAPER 1 : 25

FROM 0 m TO 30 m SPACING 5 m

FROM 30 m TO 90 m SPACING 10 m

FROM .....m TO .....m SPACING .... m

LENGTH OF RUN 90 m

PROFILING

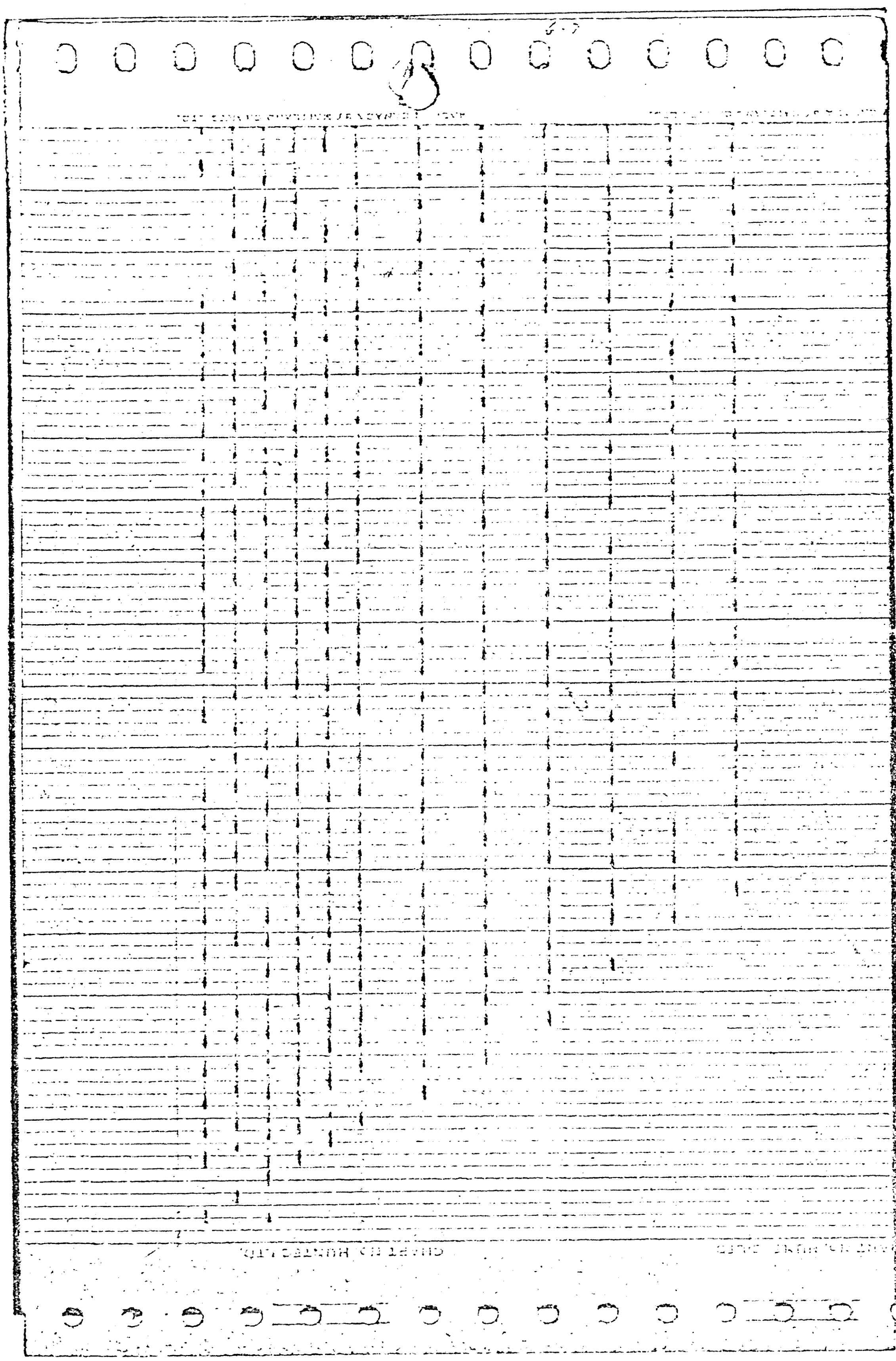
INITIAL POINT ....m END POINT ....m

BUOY NO. .... BUOY NO. ....

ELEVATION OF SHOTPOINTS

REMARKS: Floats on required a buzzing noise. Only one run was performed as only one boat was available.

SKETCHES



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Seismic Run II (Keeper)
FIGURE B8.

FIELD RECORD
SEISMOGRAPH TEST

RUN No. 14

DATE 7.1.8.177 TIME 1:00
NAME S.L.B.
LOCATION EDGE OF REEF VERY EDGE PAST BEACON

DEPTH OF WATER 0 m

NO. OF GEOPHONES

SIZE 12 lbs

TEMPERATURE

TYPE

SOIL CONDITION VERY SOUND FLAT CORAL

WEATHER CONDITION SLIGHT SWELL

INITIAL NOISE LEVEL 21 dB

SCALING ON PAPER 1 : 10

FROM 0 m TO 90 m SPACING 5 m

FROM m TO m SPACING m

FROM m TO m SPACING m

LENGTH OF RUN 90 m

PROFILING

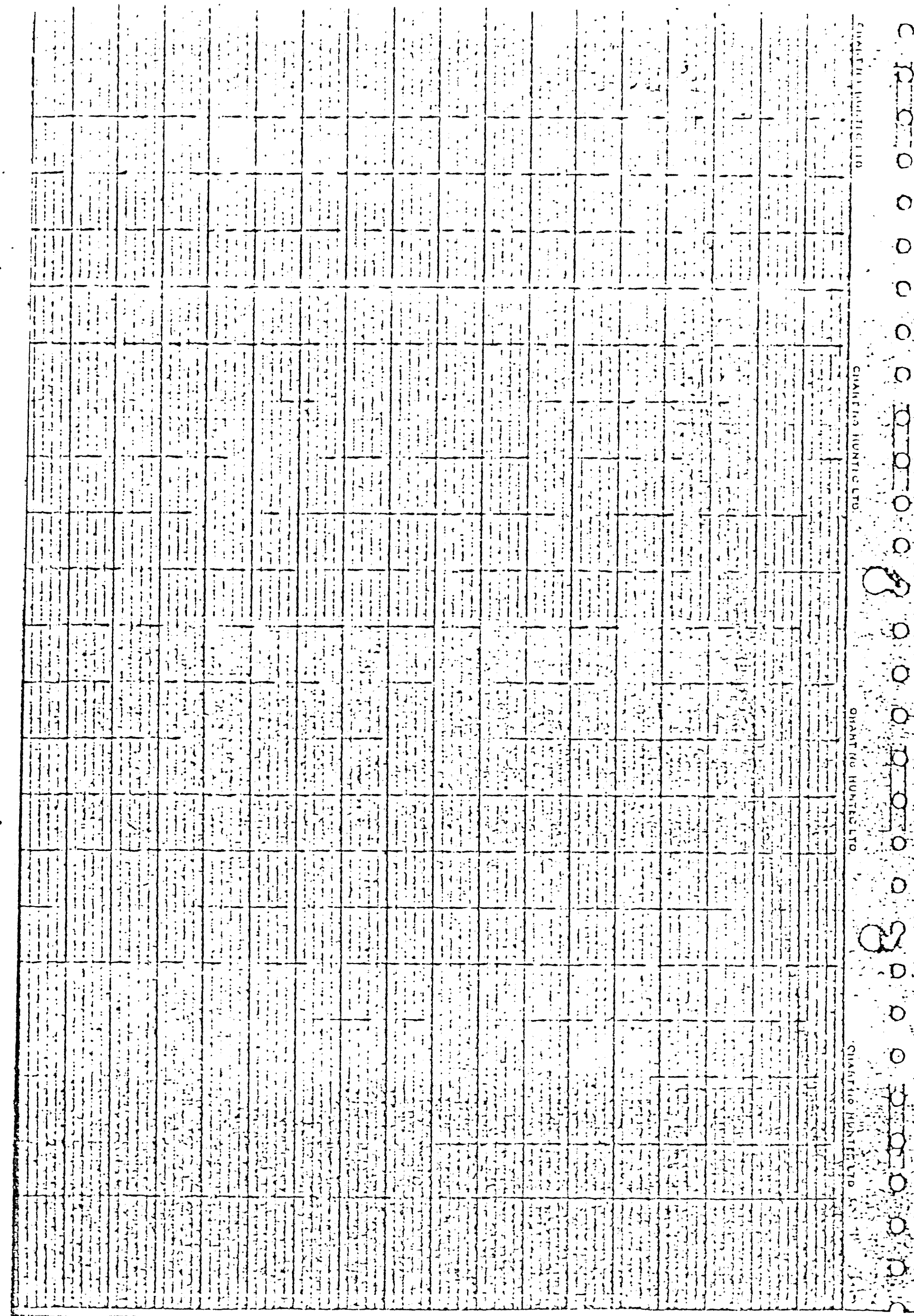
INITIAL POINT m END POINT m

BUOY NO. BUOY NO.

ELEVATION OF SHOTPOINTS

REMARKS: Results are not real good as a lot of noise is present. This can not be over come as the hammer signal is not strong enough

SKETCHES



Seismic Run III (Keeper Reef)
FIGURE B9.

FIELD RECORD
SEISMOGRAPH TEST

RUN NO. 1 R/

DATE 7/8/79 TIME 1:30

LINE S.6.6

LOCATION

NO. CHARGE ✓

GEOPHONE

HAMMER ✓

INDICATORS

DEPTH OF WATER

NO. OF GEOPHONES

SITE 12.14

TYPE

SOIL CONDITION VERY SANDY FLAT CORAL

WEATHER CONDITION SLIGHT SWELL

INITIAL NOISE LEVEL 29 dB

SCALING ON PAPER 1:1m

FROM 0 m TO 90 m SPACING= 5 m

FROM m TO m SPACING m

FROM m TO m SPACING m

LENGTH OF RUN 90 m

PROFILING

INITIAL POINT m END POINT m

BUOY NO. BUOY NO.

ELEVATION OF SHOTPOINTS

REMARKS: See sheet 1/F

SKETCHES

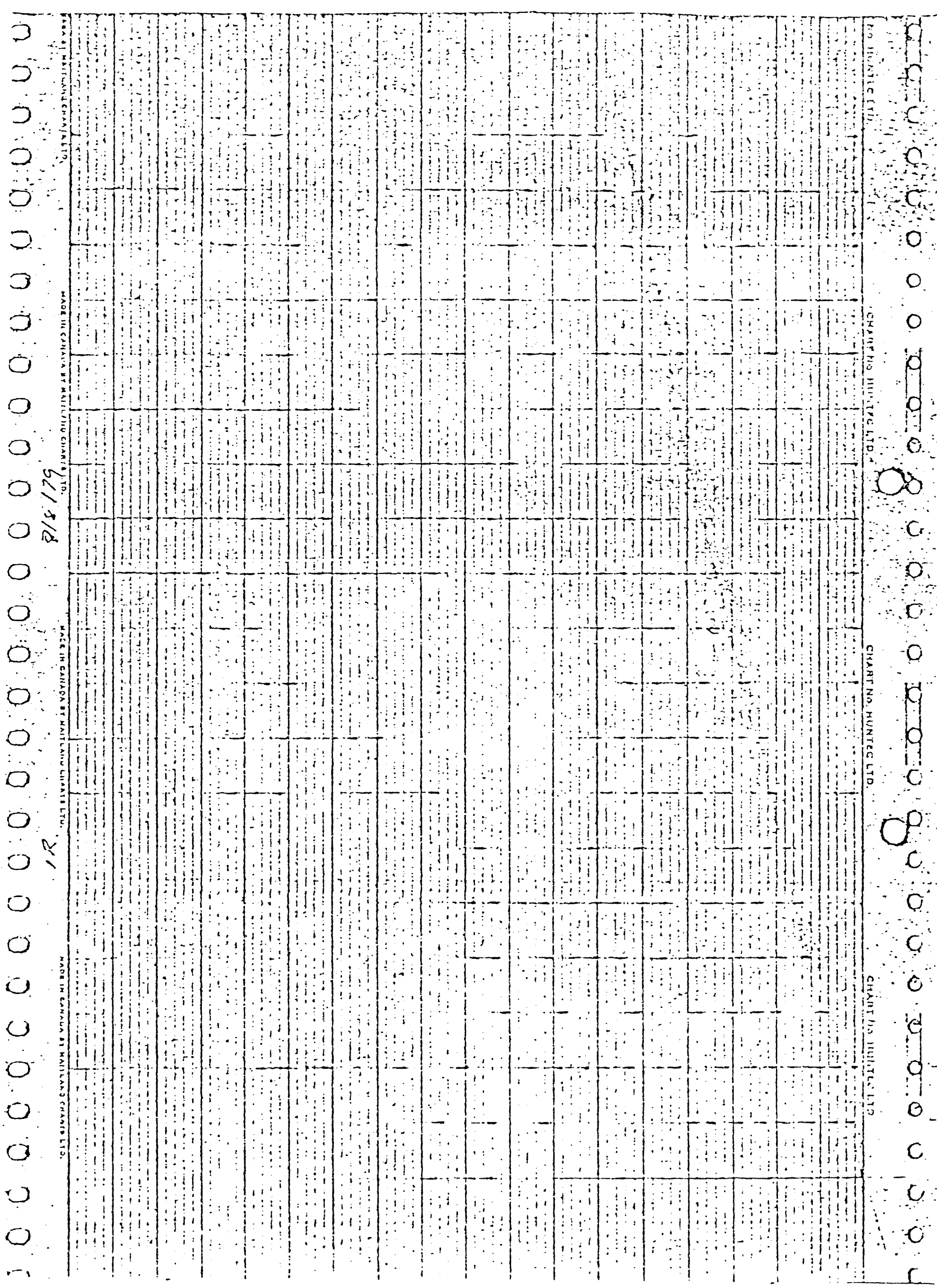


CHART No. HUNTEC LTD.
MADE IN CANADA BY MAITLAND CHARTS LTD.

Seismic Run IV (Keeper Reef)

FIGURE B10.

FIELD RECORD
SEISMOGRAPH TEST

RUN NO. 1 ✱/F

DATE 9.1.8.1.79 TIME: ....
NAME ..S.L.B..................
LOCATION .SEAWARD EDGE... LEEWARD SIDE OF BEACON..............

HYDROPHONE ✓....
DEPTH OF WATER .2 m......

GEOPHONES ......
NO. OF GEOPHONES ....

HAMMER .....
SIZE ....

DETONATORS ✓....
TYPE ....

SOIL CONDITION.. HARD FLAT CORAL → LOOSE CORAL ....

WEATHER CONDITION ..5 m.. SWELL ... NO SURF BREAKING ....

INITIAL NOISE LEVEL ....dB

SCALING ON PAPER ..1.:2.5

FROM ..0..m TO 90..m SPACING= .10......m

FROM .....m TO .....m SPACING ....m

FROM .....m TO .....m SPACING .... .m

LENGTH OF RUN .90.......m

PROFILING

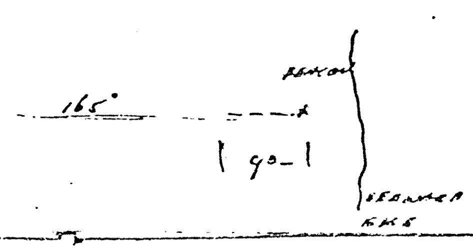
INITIAL POINT ....m END POINT ....m

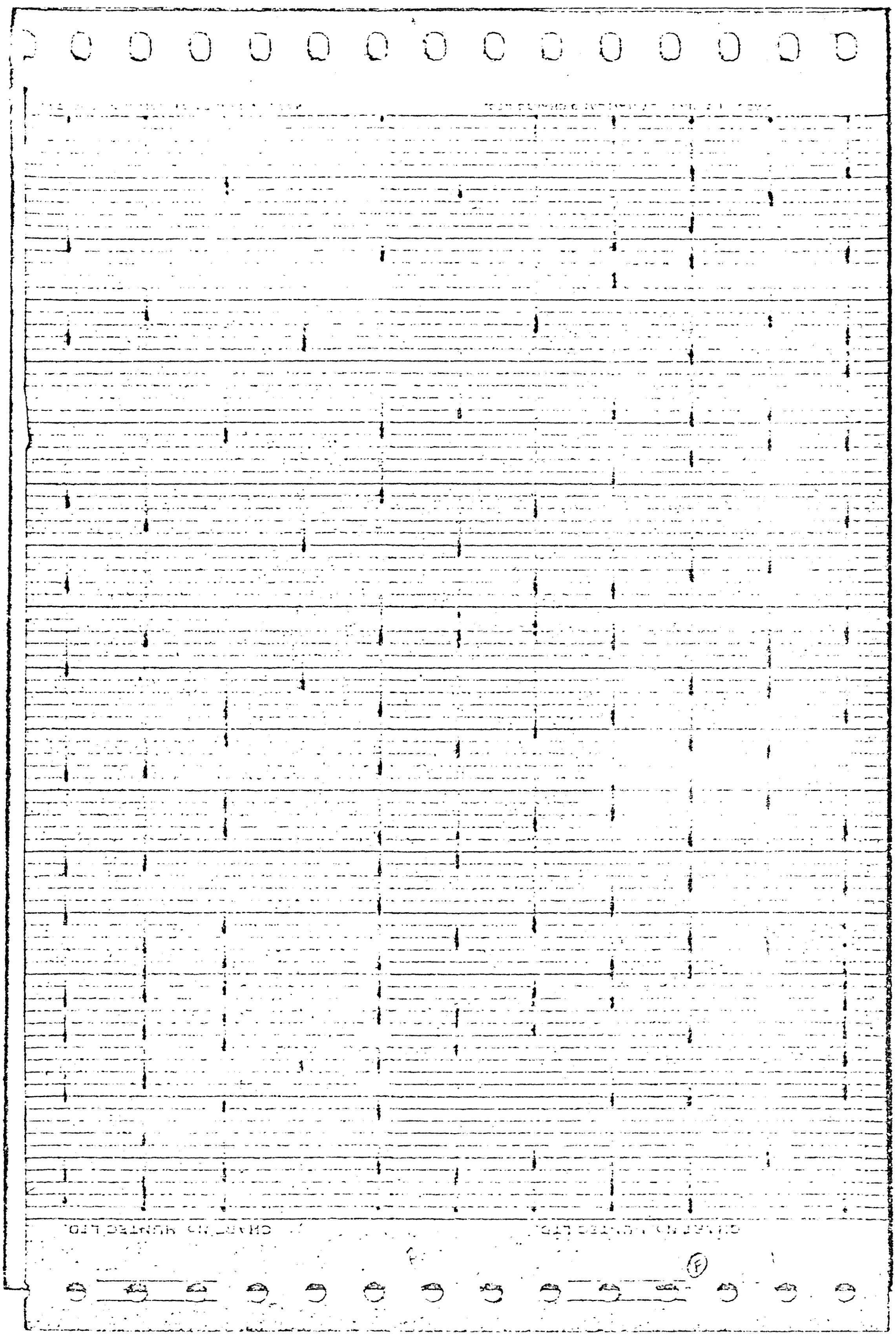
BUOY NO. .... BUOY NO. ....

ELEVATION OF SHOTPOINTS ....
....
....

REMARKS: Only two short 90 m runs could be performed as there was insufficient detonators to do longer runs
....
....

SKETCHES





Seismic Run V (Keeper Reef)

FIGURE B11.

FIELD RECORD
SEISMOGRAPH TEST

RUN NO. 1 R/[illegible]

DATE 9.1.8.79 TIME ....

NAME S.L.B.

LOCATION NEAR BEACON

HIT PLATE ✓ [illegible] 2 m

GEOPHONES ...... NO. OF GEOPHONES ....

HAMMER ..... SIZE ....

DETONATORS ✓ TYPE ....

SOIL CONDITION HARD - LOOSE CORAL

WEATHER CONDITION 0.5 m SWELL

INITIAL NOISE LEVEL ....dB

SCALING ON PAPER 1 : 2 m

FROM 0 m TO 90 m SPACING 10 m

FROM .....m TO .....m SPACING ....m

FROM .....m TO .....m SPACING .... m

LENGTH OF RUN 90 m

PROFILING

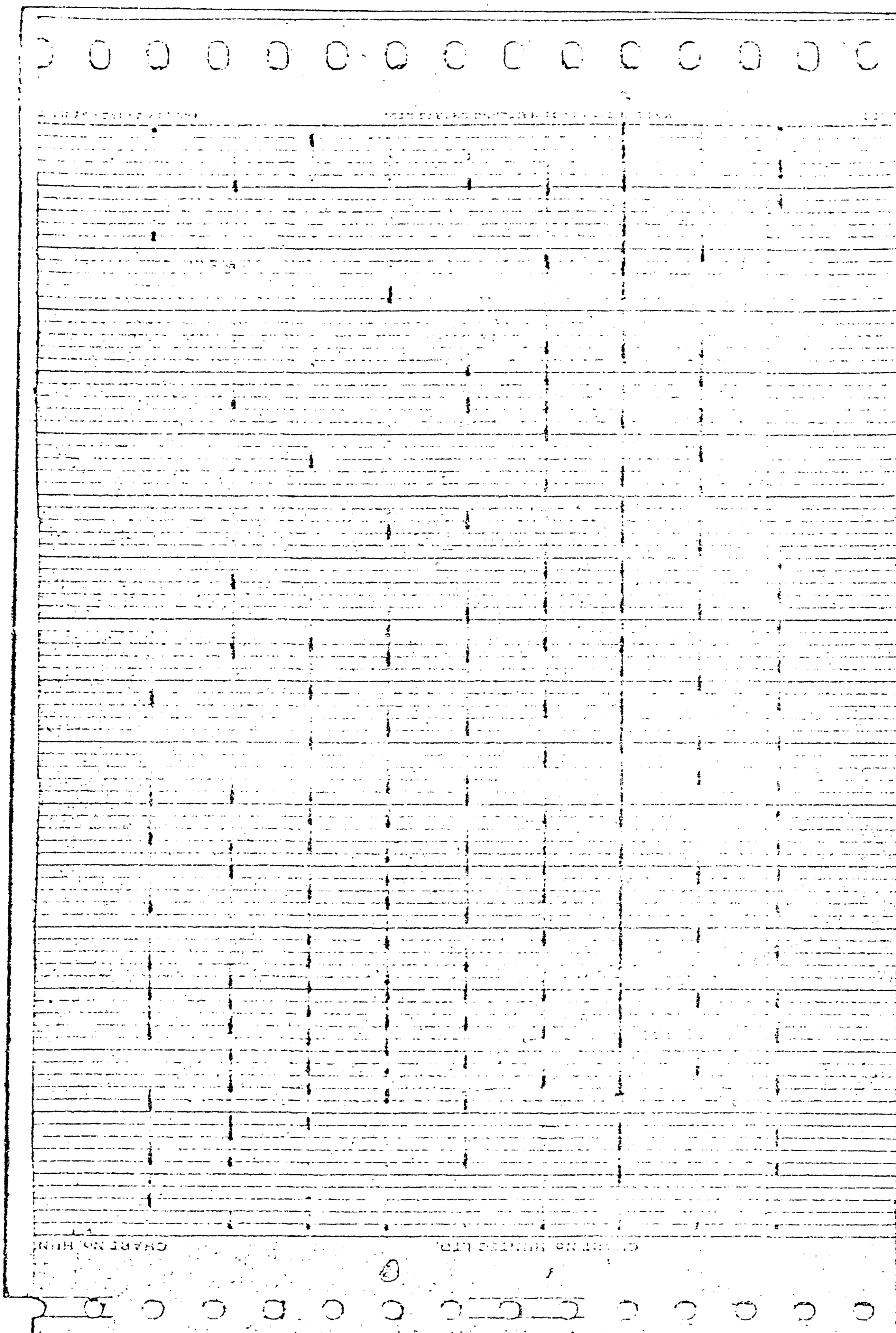
INITIAL POINT ....m END POINT ....m

BUOY NO. .... BUOY NO. ....

ELEVATION OF SHOTPOINTS ....

REMARKS: Better anchors are required for the float line. Caused [illegible] the cable is supposed to make a catenary of the float line.

SKETCHES



APPENDIX C

Penetration Test No. 4
FIGURE C1.

TEST NO. 1

Field Record

HEAVY PENETROMETER TEST

Observer: [illegible]
Date: [illegible] 6.197[illegible]
Location: [illegible]

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth	0–1 m S	0–1 m N	1–2 m S	1–2 m N	2–3 m S	2–3 m N	3–4 m S	3–4 m N	4–5 m S	4–5 m N	5–6 m S	5–6 m N
0.1	3	3	59	9	110	[illegible]	[illegible]	9	229	4	[illegible]	3
0.2	5	2	65	6	115	5	182	6	234	5	276	4
0.3	7	2	72	7	120	5	189	7	239	5	280	4
0.4	11	4	78	6	125	5	196	7	[illegible]	4	285	3
0.5	19	8	82	4	129	4	201	5	247	4	287	4
0.6	25	6	88	6	137	8	206	5	252	5	291	4
0.7	29	4	93	5	145	8	[illegible]	6	[illegible]	4	295	4
0.8	34	5	96	3	153	8	217	5	260	4	300	5
0.9	42	8	100	4	160	7	221	4	265	5	303	3
1.0	50	8	104	4	167	7	225	4	[illegible]	4	306	3

Depth	6–7 m S	6–7 m N	7–8 m S	7–8 m N	8–9 m S	8–9 m N	9–10 m S	9–10 m N	10–11 m S	10–11 m N	11–12 m S	11–12 m N
0.1	310	4	[illegible]	7	[illegible]	5	[illegible]	5	[illegible]	4	[illegible]	7
0.2	313	3	[illegible]	6	[illegible]	6	[illegible]	4	[illegible]	15	[illegible]	4
0.3	316	3	[illegible]	6	[illegible]	7	[illegible]	4	[illegible]	10	[illegible]	4
0.4	319	3	[illegible]	6	[illegible]	5	[illegible]	1	[illegible]	10	[illegible]	4
0.5	323	4	[illegible]	6	[illegible]	6	[illegible]	6	[illegible]	19	[illegible]	2
0.6	327	4	[illegible]	5	[illegible]	5	[illegible]	7	[illegible]	11	[illegible]	5
0.7	[illegible]	5	[illegible]	4	[illegible]	9	[illegible]	5	[illegible]	6	[illegible]	4
0.8	[illegible]	8	[illegible]	3	[illegible]	4	[illegible]	7	[illegible]	12	[illegible]	6
0.9	[illegible]	6	[illegible]	5	[illegible]	4	[illegible]	6	[illegible]	7	[illegible]	6
1.0	[illegible]	7	[illegible]	5	[illegible]	5	[illegible]	10	[illegible]	7	[illegible]	9

Depth	12–13 m S	12–13 m N	13–14 m S	13–14 m N	14–15 m S	14–15 m N	15–16 m S	15–16 m N	16–17 m S	16–17 m N	17–18 m S	17–18 m N
0.1	[illegible]	10	[illegible]	13	[illegible]	7	[illegible]	7	47	12	[illegible]	9
0.2	[illegible]	5	[illegible]	16	[illegible]	9	[illegible]	7	61	15	[illegible]	10
0.3	[illegible]	3	[illegible]	11	[illegible]	8	[illegible]	13	75	15	[illegible]	11
0.4	[illegible]	4	[illegible]	7	[illegible]	7	[illegible]	10	85	10	[illegible]	10
0.5	[illegible]	3	[illegible]	2	[illegible]	12	[illegible]	8	95	10	[illegible]	10
0.6	[illegible]	2	[illegible]	4	[illegible]	12	[illegible]	12	[illegible]	8	[illegible]	9
0.7	[illegible]	2	[illegible]	5	[illegible]	10	[illegible]	10	[illegible]	11	[illegible]	7
0.8	[illegible]	2	[illegible]	8	[illegible]	15	12	12	[illegible]	17	[illegible]	7
0.9	[illegible]	3	[illegible]	10	[illegible]	14	21	9	[illegible]	10	[illegible]	8
1.0	[illegible]	4	[illegible]	12	[illegible]	11	35	13	[illegible]	11	[illegible]	7

Depth	18–19 m S	18–19 m N	19–20 m S	19–20 m N	20–21 m S	20–21 m N	21–22 m S	21–22 m N	22–23 m S	22–23 m N	23–24 m S	23–24 m N
0.1	[illegible]	6	330	11	[illegible]	26						
0.2	[illegible]	9	[illegible]	17	[illegible]	25						
0.3	[illegible]	11	[illegible]	75								
0.4	[illegible]	10	[illegible]	17								
0.5	[illegible]	9	[illegible]	18								
0.6	[illegible]	9	[illegible]	[illegible]								
0.7	[illegible]	7	[illegible]	30								
0.8	[illegible]	8	[illegible]	34								
0.9	[illegible]	9	[illegible]	23								
1.0	[illegible]	11	[illegible]	27								

Remarks :

Penetration Test No. 5

FIGURE C2.

NO 2

Observer: S.L.G
Date: 19/6...1979
Location: .. [illegible]

F i e l d   R e c o r d

H E A V Y   P E N E T R O M E T E R   T E S T

S = total number of blows (Reading of counter)

N = number of blows to penetrate 10 cm

Depth	0–1 m S	0–1 m N	1–2 m S	1–2 m N	2–3 m S	2–3 m N	3–4 m S	3–4 m N	4–5 m S	4–5 m N	5–6 m S	5–6 m N	6–7 m S	6–7 m N	7–8 m S	7–8 m N	8–9 m S	8–9 m N	9–10 m S	9–10 m N	10–11 m S	10–11 m N	11–12 m S	11–12 m N
0.1		0	[illegible]	[illegible]	[illegible]	1	[illegible]	2	[illegible]	12		0	[illegible]	2	[illegible]	7	[illegible]	9	[illegible]	[illegible]	[illegible]	5	[illegible]	2
0.2		0	[illegible]	1	[illegible]	1	[illegible]	1	[illegible]	6		0	[illegible]	6	[illegible]	12	[illegible]	1	[illegible]	16	[illegible]	10	[illegible]	2
0.3		0	52	8	[illegible]	1	[illegible]	0	[illegible]	3		0	[illegible]	6	[illegible]	0	[illegible]	16	405	6	[illegible]	8	[illegible]	5
0.4		0	[illegible]	9	[illegible]	1	91	1	[illegible]	0		0	200	4	[illegible]	5	[illegible]	24	[illegible]	2	[illegible]	3	[illegible]	8
0.5	[illegible]	0	63	2	[illegible]	1	99	8	[illegible]	0		0	204	4	267	5	[illegible]	6	[illegible]	4	[illegible]	3	[illegible]	8
0.6	[illegible]	0	[illegible]	1	[illegible]	1	112	13	[illegible]	0		0	[illegible]	4	[illegible]	3	[illegible]	5	[illegible]	8	[illegible]	3	[illegible]	10
0.7	0	0	[illegible]	3	[illegible]	1	122	10	[illegible]	0		0	[illegible]	10	[illegible]	2	[illegible]	7	431	12	[illegible]	2	[illegible]	7
0.8	5	5	69	2	[illegible]	4	135	13	[illegible]	1	[illegible]	0	[illegible]	8	[illegible]	8	[illegible]	6	[illegible]	14	[illegible]	2	[illegible]	6
0.9	10	5	71	2	[illegible]	3	146	11		0	[illegible]	0	[illegible]	3	290	10	[illegible]	12	[illegible]	10	[illegible]	7	[illegible]	7
1.0	[illegible]	16	[illegible]	1	[illegible]	1	160	14		0	[illegible]	6	[illegible]	8	296	6	[illegible]	0	460	4	[illegible]	3	[illegible]	4

S N

Depth	12–13 m S	12–13 m N	13–14 m S	13–14 m N	14–15 m S	14–15 m N	15–16 m S	15–16 m N	16–17 m S	16–17 m N	17–18 m S	17–18 m N	18–19 m S	18–19 m N	19–20 m S	19–20 m N	20–21 m S	20–21 m N	21–22 m S	21–22 m N	22–23 m S	22–23 m N	23–24 m S	23–24 m N
0.1	[illegible]	5	[illegible]	5	63	7	[illegible]	6	[illegible]	8	19	21	1017	73	112	7	215	9	[illegible]	27				
0.2	[illegible]	4	[illegible]	4	66	3	[illegible]	6	85	9	[illegible]	17	41	24	122	10	225	10	[illegible]	25				
0.3	[illegible]	1	[illegible]	6	68	2	14	5	99	10	20	14	57	18	132	11	235	10	[illegible]	18				
0.4	[illegible]	5	[illegible]	3	[illegible]	3	19	5	104	10	37	12	67	8	143	10	[illegible]	13	[illegible]	18				
0.5	[illegible]	6	[illegible]	2	74	3	25	6	13	9	47	10	74	7	153	11	265	17	487	21				
0.6	[illegible]	5	[illegible]	2	[illegible]	4	33	8	21	8	57	9	[illegible]	7	[illegible]	10	[illegible]	21	504	17				
0.7	[illegible]	7	[illegible]	3	[illegible]	3	43	10	29	8	66	10	[illegible]	6	[illegible]	10	[illegible]	22	579	15				
0.8	[illegible]	6	[illegible]	2	[illegible]	5	51	8	37	10	77	11	93	6	[illegible]	11	323	15	545	26				
0.9	[illegible]	6	[illegible]	1	[illegible]	5	[illegible]	6	51	12	83	11	[illegible]	5	195	11	[illegible]	23	[illegible]	26				
1.0	[illegible]	5	[illegible]	9	97	6	67	7	66	17	99	11	105	7	[illegible]	11	[illegible]	22						

Remarks :

Penetration Test No. 6

FIGURE C3.

NO. 3

Field Record

T_s 10:00

T_f 12:15

HEAVY PENETROMETER TEST

Observer: ....
Date: 20/6...1979
Location: ..
....

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth	0–1 m S	0–1 m N	1–2 m S	1–2 m N	2–3 m S	2–3 m N	3–4 m S	3–4 m N	4–5 m S	4–5 m N	5–6 m S	5–6 m N
0.1	1	1	41	6	20	7	94	7	[illegible]	6	[illegible]	6
0.2	3	2	49	8	30	10	201	7	54	4	300	6
0.3	9	1	56	7	40	10	[illegible]	7	[illegible]	5	[illegible]	8
0.4	[illegible]	4	62	6	46	6	14	6	[illegible]	4	11	3
0.5	11	3	[illegible]	8	53	7	[illegible]	6	[illegible]	6	[illegible]	[illegible]
0.6	15	4	[illegible]	7	60	7	26	6	73	4		0
0.7	21	6	66	9	67	7	31	5	78	5	18	5
0.8	[illegible]	5	91	5	76	9	35	5	[illegible]	4	23	5
0.9	[illegible]	4	100	9	81	5	39	3	85	3	27	4
1.0	[illegible]	5	113	13	87	6	44	5	88	2	31	4

Depth	6–7 m S	6–7 m N	7–8 m S	7–8 m N	8–9 m S	8–9 m N	9–10 m S	9–10 m N	10–11 m S	10–11 m N	11–12 m S	11–12 m N
0.1	35	4	[illegible]	8	[illegible]	9	[illegible]	9	[illegible]	9	20	5
0.2	39	4	12	6	[illegible]	5	[illegible]	8	[illegible]	10	25	5
0.3	46	7	[illegible]	5	[illegible]	6	[illegible]	9	57	10	31	6
0.4	53	7	[illegible]	6	93	4	[illegible]	9	[illegible]	12	37	6
0.5	61	8	[illegible]	8	[illegible]	5	[illegible]	10	[illegible]	9	[illegible]	5
0.6	70	9	[illegible]	6	105	7	[illegible]	7	[illegible]	9	44	2
0.7	74	4	79	6	[illegible]	10	[illegible]	8	95	7	47	3
0.8	82	8	[illegible]	8	[illegible]	9	11	8	[illegible]	8	49	2
0.9	92	10	[illegible]	5	[illegible]	11	20	9	10	7	51	2
1.0	[illegible]	12	[illegible]	7	[illegible]	8	29	9	15	5	[illegible]	3

Depth	12–13 m S	12–13 m N	13–14 m S	13–14 m N	14–15 m S	14–15 m N	15–16 m S	15–16 m N	16–17 m S	16–17 m N	17–18 m S	17–18 m N
0.1	56	2	38	9	56	11	70	12	88	19	12	12
0.2	65	9	47	9	67	11	98	20	[illegible]	14	27	15
0.3	71	6	[illegible]	3	78	11	107	9	12	10	39	12
0.4	78	7	[illegible]	5	88	10	17	10	21	9	55	16
0.5	[illegible]	6	61	6	100	14	31	19	36	15	97	27
0.6	94	10	72	11	9	7	64	22	48	12	[illegible]	15
0.7	[illegible]	4	[illegible]	13	17	8	78	14	65		22	10
0.8	10	12	9	24	39	[illegible]	[illegible]	31	80		33	11
0.9	21	11	26	15	56	17	51	35	90		44	11
1.0	29	8	45	9	[illegible]	12	69	18	200		57	13

Depth	18–19 m S	18–19 m N	19–20 m S	19–20 m N	20–21 m S	20–21 m N	21–22 m S	21–22 m N	22–23 m S	22–23 m N	23–24 m S	23–24 m N
0.1	61	11	15	35	21	29						
0.2	79	11	34	19	34	11						
0.3	95	11	55	21	47	13						
0.4	518	21	77	17	[illegible]	19						
0.5	[illegible]	16	89	17	62	16						
0.6	66	30	7	18	106	24						
0.7	86	20	25	18	[illegible]	52						
0.8	[illegible]	25	45	20	[illegible]	161						
0.9	58	39	66	21								
1.0	80	22	91	26								

Remarks :

Penetration Test No. 7

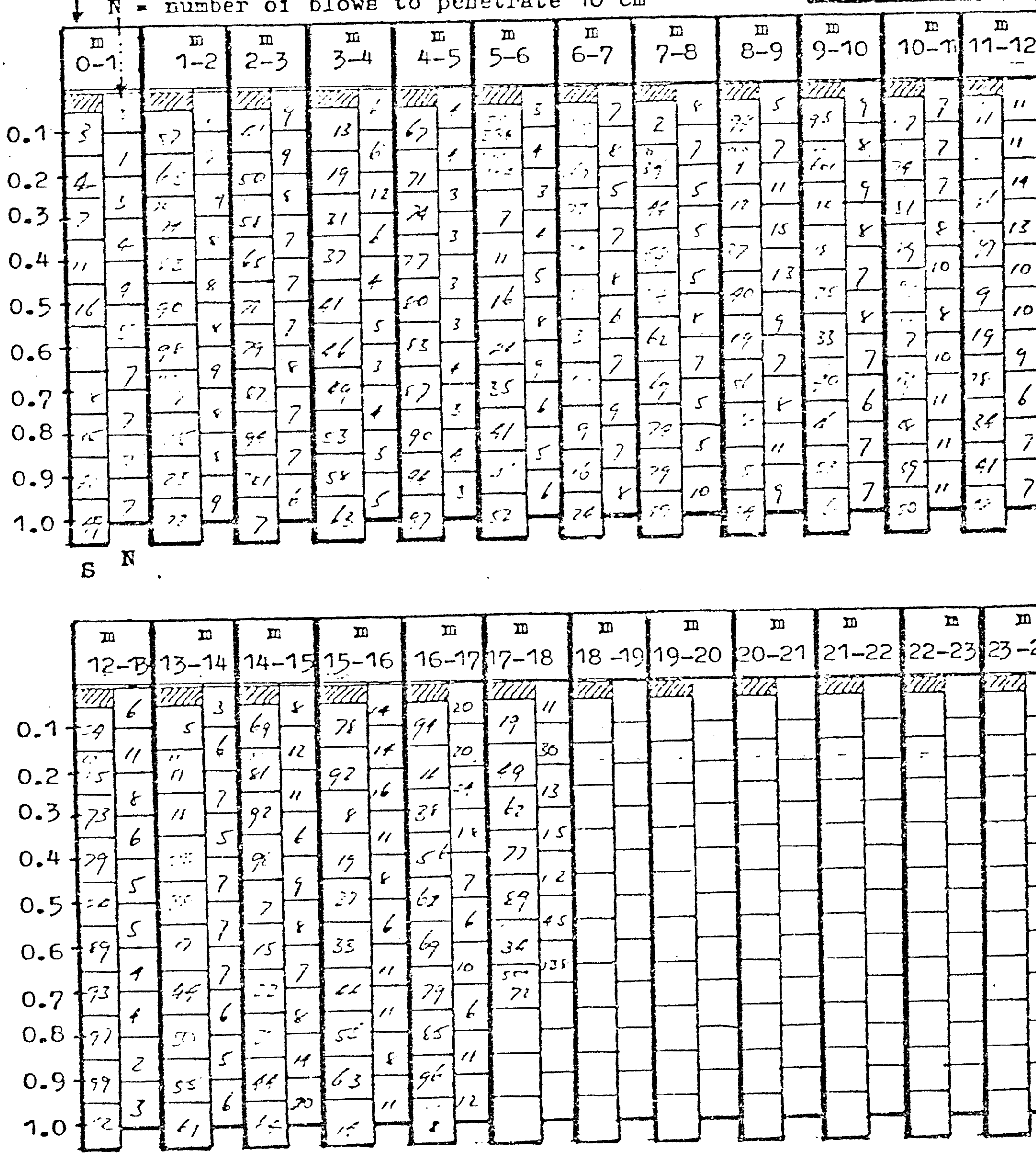
FIGURE C4.

Field Record

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)

N = number of blows to penetrate 10 cm



Remarks :

Observer: ...
Date: 10.16...1979
Location: ..
....

Penetration Test No. 8

FIGURE C5.

NO 5 T 11:00 D 1.44
T 12.45 D 1.1

Observer: [illegible]
Date: 2/6....1979
Location: ..
....

Field Record

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth (m)	0–1 m S	0–1 m N	1–2 m S	1–2 m N	2–3 m S	2–3 m N	3–4 m S	3–4 m N	4–5 m S	4–5 m N	5–6 m S	5–6 m N
0.1	3	3	40	5	[illegible]	11	[illegible]	4	[illegible]	3	52	5
0.2	6	3	44	4	[illegible]	7	[illegible]	5	97	4	60	8
0.3	9	3	48	4	10	8	[illegible]	4	[illegible]	5	66	6
0.4	12	3	[illegible]	3	[illegible]	10	[illegible]	4	[illegible]	5	72	6
0.5	15	3	[illegible]	3	[illegible]	8	72	4	12	5	78	6
0.6	18	3	58	4	[illegible]	6	75	3	19	7	86	8
0.7	21	3	[illegible]	6	21	7	79	4	[illegible]	7	92	6
0.8	[illegible]	4	[illegible]	6	[illegible]	4	83	4	34	8	[illegible]	8
0.9	20	4	[illegible]	5	[illegible]	3	[illegible]	5	40	6	7	7
1.0	[illegible]	5	[illegible]	9	[illegible]	3	[illegible]	4	47	7	[illegible]	9

Depth (m)	6–7 m S	6–7 m N	7–8 m S	7–8 m N	8–9 m S	8–9 m N	9–10 m S	9–10 m N	10–11 m S	10–11 m N	11–12 m S	11–12 m N
0.1	25	5	[illegible]	7	[illegible]	7	[illegible]	10	[illegible]	6	11	11
0.2	32	7	[illegible]	9	[illegible]	9	[illegible]	8	[illegible]	5	[illegible]	8
0.3	40	8	[illegible]	8	[illegible]	8	[illegible]	8	[illegible]	5	[illegible]	9
0.4	[illegible]	9	[illegible]	10	[illegible]	9	[illegible]	9	[illegible]	4	[illegible]	11
0.5	[illegible]	9	44	10	[illegible]	6	[illegible]	7	[illegible]	6	30	8
0.6	65	7	[illegible]	7	31	7	[illegible]	7	[illegible]	6	55	5
0.7	[illegible]	10	60	9	[illegible]	8	[illegible]	8	[illegible]	6	58	3
0.8	[illegible]	6	[illegible]	10	[illegible]	8	[illegible]	8	[illegible]	5	[illegible]	5
0.9	[illegible]	14	[illegible]	8	[illegible]	8	[illegible]	6	[illegible]	6	[illegible]	8
1.0	[illegible]	8	[illegible]	7	[illegible]	9	[illegible]	7	[illegible]	9	72	6

Depth (m)	12–13 m S	12–13 m N	13–14 m S	13–14 m N	14–15 m S	14–15 m N	15–16 m S	15–16 m N	16–17 m S	16–17 m N	17–18 m S	17–18 m N
0.1	[illegible]	3	45	13	[illegible]	24	37	9	[illegible]	11	[illegible]	14
0.2	[illegible]	5	[illegible]	14	[illegible]	15	45	8	[illegible]	16	77	17
0.3	[illegible]	5	[illegible]	10	[illegible]	19	[illegible]	5	[illegible]	13	[illegible]	15
0.4	[illegible]	7	[illegible]	12	[illegible]	12	[illegible]	7	[illegible]	14	[illegible]	38
0.5	[illegible]	6	[illegible]	18	[illegible]	8	[illegible]	15	[illegible]	30	55	25
0.6	[illegible]	6	[illegible]	21	[illegible]	7	[illegible]	17	[illegible]	20	[illegible]	20
0.7	[illegible]	6	30	10	[illegible]	15	[illegible]	10	[illegible]	31	[illegible]	13
0.8	[illegible]	6	[illegible]	17	[illegible]	19	[illegible]	17	[illegible]	17	[illegible]	13
0.9	[illegible]	6	[illegible]	11	[illegible]	15	[illegible]	26	[illegible]	16	13	12
1.0	[illegible]	11	[illegible]	17	[illegible]	16	[illegible]	18	[illegible]	12	[illegible]	14

Depth (m)	18–19 m S	18–19 m N	19–20 m S	19–20 m N	20–21 m S	20–21 m N	21–22 m S	21–22 m N	22–23 m S	22–23 m N	23–24 m S	23–24 m N
0.1	[illegible]	65	51	22	95	12	90	5	44	14		
0.2	31	39	70	19	5	10	99	4	17	33		
0.3	[illegible]	19	91	21	15	10	[illegible]	7				
0.4	[illegible]	13	[illegible]	[illegible]	22	13	11	10				
0.5	[illegible]	11	19	[illegible]	44	17	[illegible]	5				
0.6	[illegible]	10	70	[illegible]	55	11	[illegible]	2				
0.7	[illegible]	9	44	12	[illegible]	[illegible]	[illegible]	5				
0.8	3	9	58	15	75	9	30	7				
0.9	13	10	[illegible]	14	[illegible]	7	49	7				
1.0	[illegible]	16	85	19	[illegible]	5	70	11				

Remarks :

Penetration Test No. 9

FIGURE C6.

No = 6

Field Record

D = 14 T 2.15

2.06 3:45

HEAVY PENETROMETER TEST

Observer: SLB

Date: 21/6..1979

Location: ..

S = total number of blows (Reading of counter)

N = number of blows to penetrate 10 cm

Depth	0–1 m S	0–1 m N	1–2 m S	1–2 m N	2–3 m S	2–3 m N	3–4 m S	3–4 m N	4–5 m S	4–5 m N	5–6 m S	5–6 m N	6–7 m S	6–7 m N	7–8 m S	7–8 m N	8–9 m S	8–9 m N	9–10 m S	9–10 m N	10–11 m S	10–11 m N	11–12 m S	11–12 m N
0.1	2	2	20	2	52	6	17	2	7	3	45	5	15	9	14	7	12	9	93	7	7	8	73	5
0.2	4	2	22	2	56	4	18	1	10	3	49	4	23	8	26	9	20	8	4	11	15	8	77	4
0.3	5	1	25	3	60	4	19	1	14	4	53	6	30	7	36	10	29	9	14	10	23	8	83	6
0.4	6	1	27	2	62	2	50	1	17	3	62	7	37	7	47	11	36	7	31	17	31	8	88	5
0.5	7	1	29	2	66	4	52	1	21	4	68	6	45	8	56	9	44	8	54	23	38	7	94	6
0.6	7	0	31	2	70	4	92	13	23	2	76	8	53	8	67	11	53	9	66	11	46	8	100	6
0.7	8	1	34	3	75	5	97	5	26	3	82	6	62	9	76	9	61	8	74	9	51	5	6	6
0.8	12	4	37	3	79	4	100	3	30	4	89	7	71	9	86	10	69	8	82	8	56	4	12	6
0.9	15	3	40	3	[illegible]	3	2	2	36	5	96	7	99	8	94	8	76	7	90	8	62	6	18	6
1.0	[illegible]	3	[illegible]	6	[illegible]	3	4	2	40	4	6	10	5	9	3	7	84	8	99	9	68	6	24	6

Depth	12–13 m S	12–13 m N	13–14 m S	13–14 m N	14–15 m S	14–15 m N	15–16 m S	15–16 m N	16–17 m S	16–17 m N	17–18 m S	17–18 m N	18–19 m S	18–19 m N	19–20 m S	19–20 m N	20–21 m S	20–21 m N	21–22 m S	21–22 m N	22–23 m S	22–23 m N	23–24 m S	23–24 m N
0.1	29	5	9	8	6	10	87	11	9	14	30	11	13	10	67	19	24	23	32	16	77	9	16	4
0.2	34	5	17	8	16	10	99	12	24	15	38	8	25	12	54	17	53	29	40	8	52	5	50	2
0.3	40	6	27	10	28	8	10	11	38	14	46	8	38	13	100	16	73	20	[illegible]	0	54	2	1	
0.4	49	9	37	10	36	12	20	10	57	13	55	9	50	12	[illegible]	16	25	12	[illegible]	0	62	3		
0.5	57	8	46	9	61	15	30	10	75	14	64	9	66	16	37	21	35	10	[illegible]	0	59	3		
0.6	64	7	57	11	4	33	39	9	96	11	73	9	82	16	57	20	44	9	40	0	61	1		
0.7	73	9	67	10	24	20	52	13	17	11	[illegible]	6	96	14	75	18	64	4	43	2	63	2		
0.8	83	10	77	10	48	23	66	14	37	10	[illegible]	7	13	17	95	20	69	21	47	1	71	1		
0.9	92	9	86	9	63	16	79	13	[illegible]	11	94	8	27	16	29	29	92	23	57	8	7	8		
1.0	1	9	96	10	76	13	95	16	17	11	3	9	44	19	51	22	14	24	69	12	12	5		

Remarks :

Penetration Test No. 10

FIGURE C7.

FIELD RECORD

HEAVY PENETROMETER TEST

TEST NO. 115

OBSERVER: SLB

DATE: 5.8.79

LOCATION: KEEPERS REEF

TEST DATA

STARTING TIME: 10.15

FINISHING TIME: 11.45

TIME OF LOW TIDE: 1235

TIME OF HIGH TIDE: 650

DEPTH OF WATER: 3.1 m

DEPTH OF WATER: 2.2

DEPTH OF WATER: 2

DEPTH OF WATER: 2.3

AVERAGE DEPTH OF DATUM: 2.96

DEPTH OF WATER BELOW DATUM: 1.9

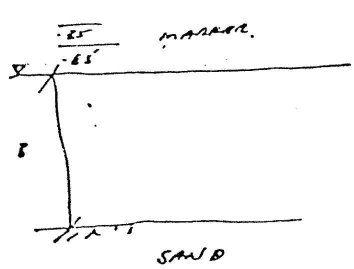
REMARKS: Stop penetration at a depth of 19.8 m as progress was very small. Bottom of lagoon is all sand, should be third first half and of fine

NB. ERRORS: TIME ELIMINATION & ROD DEFLECTIONS

SKETCHES:

0-1 NO RESISTANCE

1-3 IRRATIC.



Observer: S.A.E.
Date: 5/8...1979
Location: KEELEL AKEF

Field Record

No 1/5

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth (m)	0–1 m S	0–1 m N	1–2 m S	1–2 m N	2–3 m S	2–3 m N	3–4 m S	3–4 m N	4–5 m S	4–5 m N	5–6 m S	5–6 m N	6–7 m S	6–7 m N	7–8 m S	7–8 m N	8–9 m S	8–9 m N	9–10 m S	9–10 m N	10–11 m S	10–11 m N	11–12 m S	11–12 m N
0.1		0	1	1	16	2	40	7	71	4	26	8	83	3	82	7	35	4	80	6	30	5	72	7
0.2			2	1	17	2	45	5	81	3	35	9	85	2	77	5	40	5	87	7	39	9	102	10
0.3			4	2	20	3	50	5	85	4	45	10	87	4	92	5	44	4	90	3	45	6	12	10
0.4			7	3	22	2	53	3	88	3	51	6	[illegible]	13	96	4	47	3	96	6	54	9	19	7
0.5			10	3	27	5	56	3	91	3	55	4	19	17	[illegible]	4	51	4	107	6	60	6	28	9
0.6			12	2	29	2	60	4	95	4	61	6	33	14	3	3	53	2	10	8	66	6	39	6
0.7			13	1	20	1	63	3	101	6	67	6	47	14	10	7	55	2	15	5	72	6	41	7
0.8			[illegible]	0	21	1	66	3	105	4	71	4	56	9	18	8	62	7	17	2	75	6	53	12
0.9			[illegible]	0	22	1	70	4	109	4	76	5	[illegible]	8	25	7	68	6	21	4	81	8	65	10
1.0	0	0	13	0	23	1	74	4	116	9	[illegible]	4	[illegible]	11	31	6	74	6	25	4	[illegible]	7	[illegible]	27

Depth (m)	12–13 m S	12–13 m N	13–14 m S	13–14 m N	14–15 m S	14–15 m N	15–16 m S	15–16 m N	16–17 m S	16–17 m N	17–18 m S	17–18 m N	18–19 m S	18–19 m N	19–20 m S	19–20 m N	20–21 m S	20–21 m N	21–22 m S	21–22 m N	22–23 m S	22–23 m N	23–24 m S	23–24 m N
0.1	450	60	95	9	60	11	42	10	29	7	56	12	75	17	124	221								
0.2	4	14	53	8	81	21	49	7	41	9	69	13	31	13	204	300								
0.3	77	13	63	10	[illegible]	29	56	7	60	12	87	15	57	14										
0.4	86	9	75	12	[illegible]	14	60	4	70	10	91	16	64	12										
0.5	95	9	88	13	40	16	61	8	79	9	221	23	69	6										
0.6	703	8	[illegible]	14	57	17	[illegible]	6	57	8	42	21	71	2										
0.7	10	7	12	10	[illegible]	12	93	9	99	12	61	19	79	8										
0.8	15	8	27	10	71	22	1109	16	[illegible]	12	77	16	88	9										
0.9	25	7	37	15	[illegible]	20	20	11	27	16	93	16	95	7										
1.0	36	11	49	12	[illegible]	21	32	12	44	17	8	15	155 3	8										

Remarks :

Penetration Test No. 11

FIGURE C8.

FIELD RECORDS

HEAVY PENETROMETER TEST

TEST NO. 2/5

OBSERVER: S.L.B.

DATE: 5./8./79

LOCATION: KEELER RF. W LAGOON BEFORE THICK MATRIX OF OUTCROPS

TEST DATA

STARTING TIME: 12:15 pm

FINISHING TIME: 1:12 pm

TIME OF LOW TIDE: 1238

TIME OF HIGH TIDE: 1910

DEPTH OF WATER: 2.2 m

DEPTH OF WATER: 2.6

DEPTH OF WATER: 0.2

DEPTH OF WATER: 3.0

AVERAGE DEPTH OF DATUM: 252 D BIR

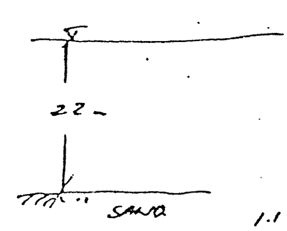
DEPTH OF WATER BELOW DATUM: 2.35

REMARKS: 2/5 WAS SOFTER THAN 1/5

COUNTER STUCK ON VERY LAST 10cm

PENETRATION WAS ABANDON DUE TO REACHING HARD LAYER

SKETCHES:



1.1 and no resistance

CHOSE LAGOON DUE TO ACCESSIBILITY

ABOUT 3m DEEP @ MW.

Observer: S.S.
Date:
8./.8...1979
Location: ..
K.K.........

Field Record

no 5/5

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

	m 0–1 S	m 0–1 N	m 1–2 S	m 1–2 N	m 2–3 S	m 2–3 N	m 3–4 S	m 3–4 N	m 4–5 S	m 4–5 N	m 5–6 S	m 5–6 N
0.1			0	0		0		1	58	3	[illegible]	4
0.2			2	2	17	1	23 25	2	62	4	7	3
0.3				0	18	1	26	1	66	4	10	3
0.4			2	0		0	29	3	68	2	14	4
0.5			3	1		1	36	4	71	3	17	3
0.6			5	2		0	39	4	73	2	19	2
0.7			10	5		0	44	5	77	4	22	3
0.8			13	3		0	48	4	82	6	28	6
0.9				2		1	51	3	92	10	33	6
1.0			15	1		0	55	4	100	8	[illegible]	1

	m 6–7 S	m 6–7 N	m 7–8 S	m 7–8 N	m 8–9 S	m 8–9 N	m 9–10 S	m 9–10 N	m 10–11 S	m 10–11 N	m 11–12 S	m 11–12 N
0.1		1	68	6	8	2	44	4	85	6	43	13
0.2	[illegible]	4	72	4	11	3	47	3	87	2	48	5
0.3	[illegible]	1	77	6	15	4	49	2	90	3	53	5
0.4	[illegible]	2	86	8	18	3	51	2	93	3	55	2
0.5	[illegible]	1	90	6	21	3	53	2	98	5	59	4
0.6	[illegible]	1	93	3	23	7	58	5	102	4	67	8
0.7	[illegible]	2	98	6	27	4	61	3	8	6	77	10
0.8	[illegible]	3	101	3	[illegible]	6	69	8	13	5	84	7
0.9	55	6	4	3	[illegible]	3	75	6	21	8	90	6
1.0	62	7	6	2	40	3	79	4	30	9	94	4

	m 12–13 S	m 12–13 N	m 13–14 S	m 13–14 N	m 14–15 S	m 14–15 N	m 15–16 S	m 15–16 N	m 16–17 S	m 16–17 N	m 17–18 S	m 17–18 N
0.1	100	6	97	5	92	9	5	13	91	6	75	2
0.2	4	4	106	9	104	12	20	15	94	3	82	7
0.3	9	5	18	12	18	14	30	10	103	9	91	9
0.4	17	8	29	11	34	16	40	10	14	11	100	9
0.5	31	14	41	12	43	9	54	14	22	8	911	11
0.6	51	20	47	6	49	6	62	8	32	10	20	9
0.7	70	19	57	10	61	12	69	7	51	9	30	10
0.8	76	6	66	9	75	14	75	6	61	10	41	11
0.9	83	7	77	11	84	9	80	5	66	5	57	16
1.0	92	9	83	6	92	8	86	6	71	5	73	16

	m 18–19 S	m 18–19 N	m 19–20 S	m 19–20 N	m 20–21 S	m 20–21 N	m 21–22 S	m 21–22 N	m 22–23 S	m 22–23 N	m 23–24 S	m 23–24 N
0.1	84	11	86	3	56	62						
0.2	96	12	88	2	135	114						
0.3	1010	14	93	5								
0.4	25	15	97	4								
0.5	37	12	106	9								
0.6	47	10	14	8								
0.7	57	10	35	21								
0.8	69	12	49	14								
0.9	78	9	54	5								
1.0	83	5	74	20								

Remarks :

Penetration Test No. 12

FIGURE C9.

FIELD RECORDS

HAND PENETROMETER TEST

TEST NO. 5/5

OBSERVER: S.L.B.

DATE: 6/8/79

LOCATION: ....

TIDE DATA

STARTING TIME: 10:40 — DEPTH OF WATER: 3.2

FINISHING TIME: 12:00 — DEPTH OF WATER: 2.4 1.8

TIME OF LOW TIDE: 1313 — DEPTH OF WATER: 0.0

TIME OF HIGH TIDE: 6.41 — DEPTH OF WATER: 2.1

AVERAGE DEPTH OF DATUM: ....

DEPTH OF WATER BELOW DATUM: 2.2

REMARKS: ....

SKETCHES:

Observer: S.L.B
Date: 6/1...1979
Location: ..

Field Record 3/5

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth	0-1 m S	0-1 m N	1-2 m S	1-2 m N	2-3 m S	2-3 m N	3-4 m S	3-4 m N	4-5 m S	4-5 m N	5-6 m S	5-6 m N
0.1			9	1	9	0	16	1	53	2	84	4
0.2				0		1	18	2	55	2	88	4
0.3				0		0	20	2	59	4	92	4
0.4			..	0		0	24	4	62	3	95	3
0.5				0	10	1	28	4	65	3	99	4
0.6	0	0		0	11	1	33	5	69	4	102	3
0.7	2	2		0	12	1	38	5	71	2	7	5
0.8	4	2		0	13	1	42	4	74	3	9	2
0.9	7	3		0		1	46	4	77	3	12	3
1.0	8	1		0		1	51	5	80	3	15	3

Depth	6-7 m S	6-7 m N	7-8 m S	7-8 m N	8-9 m S	8-9 m N	9-10 m S	9-10 m N	10-11 m S	10-11 m N	11-12 m S	11-12 m N
0.1	17	2	77	10	32	10	14	8	8	10	68	8
0.2	20	3	84	7	40	8	24	10	16	8	77	9
0.3	25	5	91	7	49	9	33	9	22	6	85	8
0.4	30	5	95	4	55	6	43	10	28	6	94	9
0.5	33	3	98	4	63	8	55	12	33	5	403	9
0.6	36	3	[illegible]	4	70	7	65	10	38	5	12	9
0.7	38	2	209	6	76	6	72	7	45	7	25	13
0.8	43	5	12	4	86	10	83	11	50	5	40	15
0.9	51	8	17	5	94	8	91	8	55	5	51	11
1.0	67	16	[illegible]	5	306	12	98	7	60	5	60	9

Depth	12-13 m S	12-13 m N	13-14 m S	13-14 m N	14-15 m S	14-15 m N	15-16 m S	15-16 m N	16-17 m S	16-17 m N	17-18 m S	17-18 m N
0.1	68	8	10	2	68	12	84	21	32	12	45	17
0.2	74	6	13	3	80	12	713	29	42	10	56	11
0.3	80	6	16	3	91	11	31	18	52	10	68	12
0.4	85	5	18	2	602	11	48	17	66	14	76	8
0.5	88	3	20	2	11	9	64	16	82	16	96	20
0.6	91	3	27	7	22	11	76	12	909	27	116	20
0.7	96	5	34	7	32	10	87	11	35	26	36	20
0.8	500	4	37	3	41	9	801	14	69	34	49	13
0.9	5	5	40	3	50	9	11	10	97	28	65	16
1.0	8	3	56	16	63	13	20	9	1028	31	81	16

Depth	18-19 m S	18-19 m N	19-20 m S	19-20 m N	20-21 m S	20-21 m N	21-22 m S	21-22 m N	22-23 m S	22-23 m N	23-24 m S	23-24 m N
0.1	98	17	67	13	1530	61						
0.2	209	11	77	10	1730	200						
0.3	25	16	89	12								
0.4	29	4	98	9								
0.5	32	3	320	22								
0.6	38	6	51	31								
0.7	41	3	79	28								
0.8	45	4	408	29								
0.9	48	3	38	30								
1.0	54	6	69	31								

Remarks :

Penetration Test No. 13

FIGURE C10.

FIELD RECORDS

HEAVY PENETROMETER TEST

TEST NO. 4/5

OBSERVER: S.L.B.

DATE: 6./8./79

LOCATION: KEELERS RF ..... SMALL LAGOON

TEST DATA

STARTING TIME: 12:20 DEPTH OF WATER: 2.4m

FINISHING TIME: 1:35 DEPTH OF WATER: 2.3m

TIME OF LOW TIDE: 1813 DEPTH OF WATER: 0.0

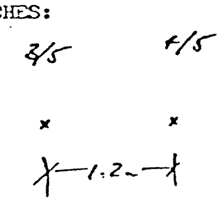
TIME OF HIGH TIDE: 1915 DEPTH OF WATER: 3.2

AVERAGE DEPTH OF DATUM: .02

DEPTH OF WATER BELOW DATUM: 2.275

REMARKS: PENETRATION WAS ABANDON DUE TO NO PROGRESS 123 L 60 5cm. EAST WEATHER WIND HAR

SKETCHES:



Observer: SAR
Date:
..6/5...1979
Location: ..
Kh.........

Field Record 4/5

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth (m)	0-1 m S	0-1 m N	1-2 m S	1-2 m N	2-3 m S	2-3 m N	3-4 m S	3-4 m N	4-5 m S	4-5 m N	5-6 m S	5-6 m N
0.1		0	18	0		0	27	2	66	3	92	3
0.2		0	19	1		0	29	2	68	2	94	2
0.3	0	0	20	1		0	22	4	70	2	98	4
0.4	4	4	21	1		0	31	6	72	2	101	3
0.5	5	1	22	1		0	43	5	74	2	103	2
0.6	9	4	23	1		0	49	6	76	2	8	5
0.7	12	3		0		0	58	5	79	3	12	4
0.8	15	3		0		0	57	3	82	3	15	3
0.9	17	2		0		0	60	3	85	3	18	3
1.0	18	1		0	25	2	63	3	89	4	21	3

Depth (m)	6-7 m S	6-7 m N	7-8 m S	7-8 m N	8-9 m S	8-9 m N	9-10 m S	9-10 m N	10-11 m S	10-11 m N	11-12 m S	11-12 m N
0.1	23	2	59	3	45	17	80	13	71	8	29	4
0.2	26	3	62	3	60	15	91	11	77	6	34	5
0.3	28	2	64	2	72	12	100	9	84	7	38	4
0.4	30	2	68	4	83	11	10	10	94	10	42	4
0.5	33	3	70	2	95	12	18	8	500	6	44	2
0.6	36	3	75	6	6	11	28	10	9	9	47	3
0.7	40	4	86	11	17	11	36	8	15	6	51	4
0.8	44	4	98	12	35	18	46	10	19	4	56	5
0.9	50	6	711	13	49	14	35	9	23	4	61	5
1.0	56	6	28	17	67	18	63	8	25	2	66	6

Depth (m)	12-13 m S	12-13 m N	13-14 m S	13-14 m N	14-15 m S	14-15 m N	15-16 m S	15-16 m N	16-17 m S	16-17 m N	17-18 m S	17-18 m N
0.1	73	7	43	8	77	8	66	7	83	5	29	32
0.2	77	4	52	9	600	23	88	22	91	8	79	50
0.3	81	4	70	8	617	17	901	13	1000	9	1214	35
0.4	86	5	85	15	628	11	11	10	17	17	28	14
0.5	92	6	97	12	31	3	24	13	39	22	41	13
0.6	[illegible]	8	23	26	35	4	35	11	58	19	53	12
0.7	7	7	36	13	39	4	38	3	75	17	62	9
0.8	14	7	45	9	47	8	57	19	83	8	72	10
0.9	25	9	58	13	54	7	71	14	88	5	81	9
1.0	35	10	69	11	59	5	78	7	97	9	91	10

Depth (m)	18-19 m S	18-19 m N	19-20 m S	19-20 m N	20-21 m S	20-21 m N	21-22 m S	21-22 m N	22-23 m S	22-23 m N	23-24 m S	23-24 m N
0.1	1300	9	16	43								
0.2	1310	10	88	72								
0.3	17	7	1509	21								
0.4	25	8	1588	79								
0.5	29	4	1711	123								
0.6	35	6										
0.7	42	7										
0.8	50	8										
0.9	60	10										
1.0	73	13										

Remarks :

Penetration Test No. 14

FIGURE C11.

FIELD RECORDS

HEAVY PENETROMETER TEST

TEST NO. 5/5

OBSERVER: S.L.B.

DATE: 6.8.'79

LOCATION: ....

TEST DATA

STARTING TIME: 2:00 — DEPTH OF WATER: 6.5

FINISHING TIME: 3:00 — DEPTH OF WATER: 7.6

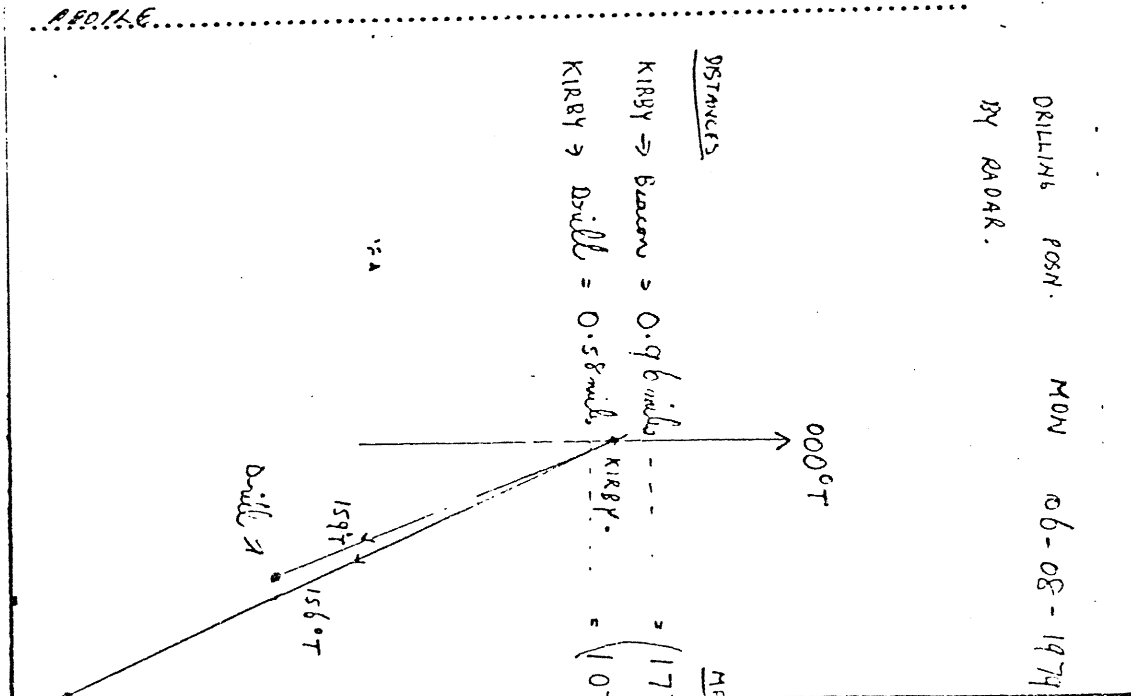
TIME OF LOW TIDE: 1313 — DEPTH OF WATER: 0.0

TIME OF HIGH TIDE: 1915 — DEPTH OF WATER: 8.2

AVERAGE DEPTH OF DATUM: .55

DEPTH OF WATER BELOW DATUM: 1.06

REMARKS: LAST OF A SET OF 5 PENETRATION STOP DUE TO 188 BLOWS/10cm 3 HOLES IN ONE DAY COMES TO LIMIT OF PEOPLE



Field Record

5/5

Observer: SAK
Date:
6/8...1979
Location: ..
....

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

m	0-1 S	0-1 N	1-2 S	1-2 N	2-3 S	2-3 N	3-4 S	3-4 N	4-5 S	4-5 N	5-6 S	5-6 N	6-7 S	6-7 N	7-8 S	7-8 N	8-9 S	8-9 N	9-10 S	9-10 N	10-11 S	10-11 N	11-12 S	11-12 N
0.1			6	3	34	2	40	0	63	4	94	3	30	4	73	3	16	7	04	8	96	7	59	7
0.2			8	4	35	1	41	1	67	4	100	6	35	5	76	4	23	7	13	9	3	7	66	7
0.3			10	2	35	0		1	70	3	4	4	39	4	77	1	32	9	26	13	8	5	71	5
0.4			15	5	36	0	43	1	73	3	7	3	42	3	80	3	44	12	36	10	13	5	76	5
0.5			19	4	36	0	44	1	77	4	11	4	46	4	84	4	53	9	46	10	18	5	82	6
0.6			21	3	37	1	48	4	80	3	14	3	50	4	87	3	62	9	56	8	25	7	87	5
0.7			23	2	37	0	50	2	83	3	16	2	55	5	91	4	69	7	65	11	33	8	94	7
0.8	4		26	3	38	0	52	2	85	2	18	2	60	5	96	5	78	9	73	8	41	8	1	7
0.9	0.	0	29	3	39	1	55	3	88	3	21	3	65	5	102	6	87	9	80	7	47	6	7	6
1.0	1	1	32	3	40	1	59	4	91	3	26	5	69	4	109	7	96	9	89	9	52	5	12	5

S N

m	12-13 S	12-13 N	13-14 S	13-14 N	14-15 S	14-15 N	15-16 S	15-16 N	16-17 S	16-17 N	17-18 S	17-18 N	18-19 S	18-19 N	19-20 S	19-20 N	20-21 S	20-21 N	21-22 S	21-22 N	22-23 S	22-23 N	23-24 S	23-24 N
0.1	18	6	92	5	37	8	34	7	73	7	52	13	50	11	89	9	95	6						
0.2	26	8	96	4	46	9	40	6	80	7	60	8	63	13	97	8	51	6						
0.3	32	6	1	5	53	7	44	4	83	3	67	7	81	18	3	6	55	4						
0.4	38	6	2	1	57	4	50	6	90	7	74	7	97	18	7	4	69	14						
0.5	47	9	3	1	65	8	54	4	97	7	19	25	14	15	10	3	16	17						
0.6	56	9	5	2	75	10	58	4	5	8	44	25	28	14	15	5	54	138						
0.7	65	9	9	4	90	15	62	4	11	6	78	34	40	12	21	6								
0.8	75	10	15	6	2	12	63	1	23	12	4	26	53	13	25	4								
0.9	81	6	21	6	17	16	64	1	28	5	30	26	67	14	31	6								
1.0	87	6	29	8	27	10	66	2	39	11	39	9	80	13	39	8								

Remarks :

Penetration Test No. 15
FIGURE C12.

FIELD RECORD

HEAVY PENETROMETER TEST

TEST NO. 2

OBSERVER: SLB

DATE: 7.8.'79

LOCATION: KRF .... VERY EDGE OF REEF

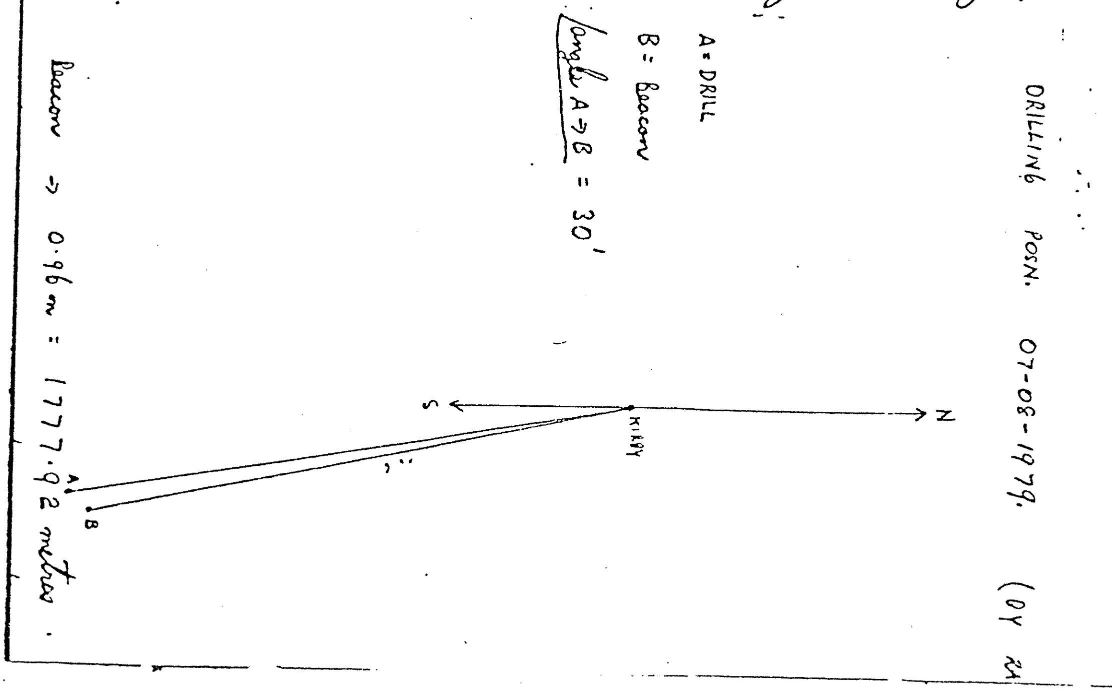
TEST DATA

STARTING TIME: 12:15 → 2:30	DEPTH OF WATER: .5 m .3
FINISHING TIME: 1:00 5:55	DEPTH OF WATER: .25 m
TIME OF LOW TIDE: 1:40 pm	DEPTH OF WATER: 0.1
TIME OF HIGH TIDE: 8:20 pm	DEPTH OF WATER: 3.4

AVERAGE DEPTH OF BATUM: .356

DEPTH OF WATER BELOW DATUM: 0

REMARKS: Weather was fine in the morning wind got choppy when we reached the front of reef. Low went to about 2:00 ..... WATER JUST TOUCHED ROD AT 3:00 pm Had to stop due to incoming tide rocky reef.



Observer: S.LE
Date:
21.8...1979
Location: ..
.E06.E......

Field Record

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

m		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
0–1	N	8	12	15	17	12	9	11	17	6	4
0–1	S	8	20	35	52	64	73	84	01	7	11
1–2	N	4	5	7	12	15	10	6	4	2	5
1–2	S	15	20	27	39	54	64	70	74	76	81
2–3	N	6	3	4	4	8	9	5	6	3	8
2–3	S	86	89	93	97	5	14	19	25	28	36
3–4	N	8	9	7	8	4	7	4	5	9	6
3–4	S	44	53	60	68	72	79	83	88	97	3
4–5	N	9	8	10	5	3	5	5	5	12	11
4–5	S	12	20	30	35	38	43	48	53	65	76
5–6	N	8	6	4	5	6	4	5	6	16	34
5–6	S	84	90	94	99	5	9	14	19	35	69
6–7	N	32	56	11	29	12	17	29	62	20	15
6–7	S	1	57	68	97	9	26	55	17	37	52
7–8	N	10	10	11	11	12	12	22	12	12	8
7–8	S	62	72	83	94	6	18	40	52	64	72
8–9	N	8	26	19	22	15	12	12	10	15	12
8–9	S	80	6	25	47	62	74	86	96	11	23
9–10	N	8	14	22	20	18	16	15	9	8	9
9–10	S	31	45	67	87	5	21	36	45	53	62
10–11	N	15	24	37	15	14	19	19	29	16	15
10–11	S	77	1	38	53	67	86	5	34	50	65
11–12	N	45	66	66	93	43	34	103	26	27	79
11–12	S	10	76	42	35	78	12	15	41	68	47

S N

m		0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
12–13	N	48	75	26	26	22	84	52	36	41	30
12–13	S	95	70	46	22	44	48	100	36	77	7
13–14	N	30	13	9	20	26	53	40	36	14	10
13–14	S	37	50	59	79	5	58	98	34	48	58
14–15	N	10	17	15	13	10	11	13	10	8	9
14–15	S	68	85	100	13	23	34	47	57	65	74
15–16	N	18	15	11	10	14	12	14	12	10	9
15–16	S	92	7	18	28	42	54	68	80	90	99
16–17	N	10	14	8	11	13	15	20	18	13	11
16–17	S	9	23	31	42	55	70	90	8	21	32
17–18	N	12	15	19	19	17	22	21	14	15	18
17–18	S	44	59	78	97	14	36	57	71	86	4
18–19	N	22	9	10	10	16	16	18	14	16	13
18–19	S	26	35	45	55	71	87	5	19	35	48
19–20	N	25	30	18	12	15	18	17	13		
19–20	S	73	3	21	33	[illegible]	[illegible]	85	[illegible]		
20–21	N										
20–21	S										
21–22	N										
21–22	S										
22–23	N										
22–23	S										
23–24	N										
23–24	S										

Remarks :

Penetration Test No. 16
FIGURE C13.

FIELD RECORDS

HEAVY PENETROMETER TEST

TEST NO. 5
1 HOLE

OBSERVER: S.L.B.

DATE: 9./8./79

LOCATION: KEELERS ... WITHIN OUTER ...

TEST DATA

STARTING TIME: 1:55 — DEPTH OF WATER: ·8 m

FINISHING TIME: 8:00 — DEPTH OF WATER: ·5 m

TIME OF LOW TIDE: 15:15 — DEPTH OF WATER: 0.0

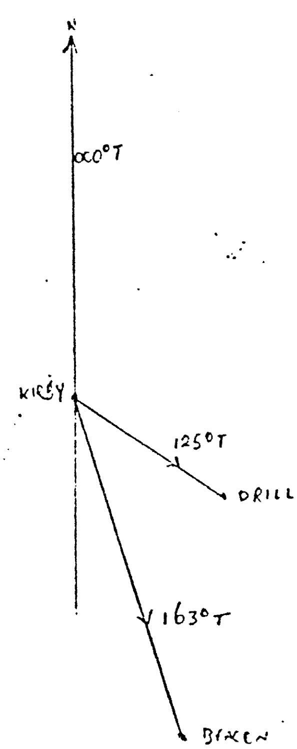
TIME OF HIGH TIDE: 21:45 — DEPTH OF WATER: 3.5

AVERAGE DEPTH OF DATUM: 0 m

DEPTH OF WATER BELOW DATUM: ·5 m

REMARKS: DRILLING POSN.
09-08-1979
(By Radar)

SKETCHES:



Observer: S.K.K.
Date:
9/8....1979
Location: ..
....

Field Record

HEAVY PENETROMETER TEST

S = total number of blows (Reading of counter)
N = number of blows to penetrate 10 cm

Depth (m)	0-1 S	0-1 N	1-2 S	1-2 N	2-3 S	2-3 N	3-4 S	3-4 N	4-5 S	4-5 N	5-6 S	5-6 N	6-7 S	6-7 N	7-8	8-9	9-10	10-11	11-12
0.1	2	2	74	7	69	8	63	11	83	12	13	8	18	16					
0.2	14	12	80	6	79	10	72	7	91	8	24	11	93	75					
0.3	21	7	86	6	86	7	83	11	7	16	37	13	7	516					
0.4	27	6	7	21	92	6	92	9	16	9	50	13							
0.5	32	5	17	10	99	7	5	13	27	11	57	7							
0.6	38	6	24	7	7	8	17	12	52	25	64	7							
0.7	43	5	32	8	17	10	30	13	75	23	71	7							
0.8	51	8	41	9	27	10	39	9	88	13	80	9							
0.9	60	9	49	8	38	11	54	15	97	9	92	12							
1.0	67	7	61	12	52	14	71	17	5	8	2	10							

Depth (m)	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24
0.1												
0.2												
0.3												
.4												
0.5												
0.6												
0.7												
0.8												
0.9												
1.0												

Remarks :

Penetration was terminated as no progress was made after 516 blows the progress was — so slow the [illegible] going [illegible] was increasing the [illegible]

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Appendix 3

BULGARELLI, G.:

"A Review of the Design and Construction Principles of Structures on Coral Reefs".

ABSTRACT

With the increasing interest being shown in utilizing the vast resources of the Great Barrier Reef, it is inevitable that major engineering structures, on the reef itself, will be required. The majority of significant structures on the reef, at present, are lighthouses and a few automatic weather stations. Many of these structures are founded on bare coral sites that are either fully, or partly, submerged.

Marine platform-type structures are used extensively by the petroleum industry on non-coral reef situations, but some of the basic design and construction principles employed for these structures can be applied to any marine environment.

This project involves a review of the design and construction principles of existing structures on coral reefs, and, a consideration of principles used in other marine situations that could be applied. The problems associated with both the design and construction are also discussed extensively. It is hoped that the information acquired from this review will assist future development on the Great Barrier Reef.

ACKNOWLEDGEMENTS

The author wishes to thank Dr. H. Bock, Lecturer in Civil Engineering, for his help and guidance throughout the project. Thanks are also due to Dr. R. Kenchington of the Great Barrier Reef Marine Park Authority for the keen interest shown by him in getting the project underway.

Mrs. J. Petersen is also thanked for all the typing work.

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CHAPTER 1

INTRODUCTION

The Great Barrier Reef has been described as one of the most beautiful natural wonders of the world and, due to its relative inaccessibility, it has remained in its natural state, virtually untouched by man. It is only in the last fifteen to twenty years that a real interest has been shown in utilizing the vast and varied resources of this natural wonderland.

The extent of the reef is in itself staggering: running for 2000 kilometres parallel to the Queensland coast from a latitude of 9½ degrees south in the Torres Strait to Lady Elliot Island situated in the parallel of 24 degrees south. The average distance of the barrier's outer edge from the mainland does not exceed 56 kilometres and at one or two isolated points, the distance is as little as 19 kilometres. Figure 1.1 shows the reef and its relationship to the Queensland coast.

The activities for which the reef's resources could most effectively be utilized are recreation and as a major food source, both of which are almost unlimited. Food production would not only involve the catching and processing of fish and marine organisms, but also planned cultivation and harvesting. With the exception of Heron Island, tourist activities on the reef proper are at present restricted to one-day or week-end trips on pleasure boats. A much more attractive proposition to the public would be permanent hotel-type structures on the reef itself. Permanent or semi-permanent structures would also be required for any cultivating activities or major scientific projects.

The activities mentioned above would mainly be restricted to two types of coral reef environments. These are:

(A) Cays and islands of coral reef origons. These are usually small and very flat, and construction in these is made easier by the fact that you have a firm terrestrial base. A structure in such an environment would be concerned more with the problems of construction in soft sand than on bare coral.

(B) Bare coral reefs. They have no terrestrial formation upon them whatsoever and are either completely submerged all of the time, or partly exposed at low tide. Although this type of environment is the most difficult for construction purposes, it is usually the most suitable for the activities concerned.

In the past, construction on these reef environments has been mainly restricted to lighthouses, and a few automatic weather stations. The types of structures found on the Great Barrier Reef at present, as well as those that may be used in the future, are discussed in Chapter 3.

If the resources of the reef are to be utilized, it will involve the construction of major engineering structures that will have to be commercially viable. That is, the structure will have to be as economic to build as possible, but at the same time serving its required purpose with maximum efficiency. Previous structures such as lighthouses and weather stations were considered essential, which meant that the economic aspect was not the primary consideration.

With the increasing interest being shown in the Great Barrier Reef, it is essential that some sort of general design and construction guidelines be set up in order to assist and control development on the reef. When setting up these guidelines, one thing should remain paramount: the delicate ecological balance of this remarkable environment should not be adversely affected by proposed development.

It is the aim of this thesis to review the design and construction principles employed for previous structures, to discuss the problems involved, and to suggest the best procedures with the view to setting up general guidelines. In order to effectively illustrate the special design principles employed for structures on coral reefs, the review of these principles will be divided into two sections: the foundations (Chapter 5) and the superstructure (Chapter 6). The special problems associated with the design will also be discussed in these chapters, while Chapter 3 will discuss general problems stemming from the fact that we are considering a coral reef situation. Chapter 7 will discuss the construction techniques and principles that have been used on coral reefs, as well as considering those that could be applied if necessary.

When considering design and construction on a coral reef, it is helpful to have an insight into the formation and type of reef that is present, and because of this a brief discussion on reef classification and description have been included (Chapter 2). The final chapter (Chapter 8) will discuss conclusions deduced throughout, as well as proposing suitable recommendations that could be employed in future construction on coral reefs.

Originally it was hoped to collect information concerning structures on coral reefs from all parts of the world, but due to the apparent limited experience in this field that exists overseas, and also the difficulty associated with contacting relevant organizations, the majority of the discussion will refer to structures on the Great Barrier Reef. Appendix B gives a summary of the contacts made, both in Australia and overseas, as well as the research activities undertaken.

Also, a detailed literature review was undertaken in order to ascertain what publications exist, concerning structure on coral reefs. Appendix A details the work undertaken and the results obtained, and also gives a list of articles that may be of interest when planning a structure in a coral reef environment.

It is hoped that the information that has been acquired from the abovementioned sources and collated in the following discussion will assist in the future design and construction of engineering structures on coral reefs, particularly the Great Barrier Reef.

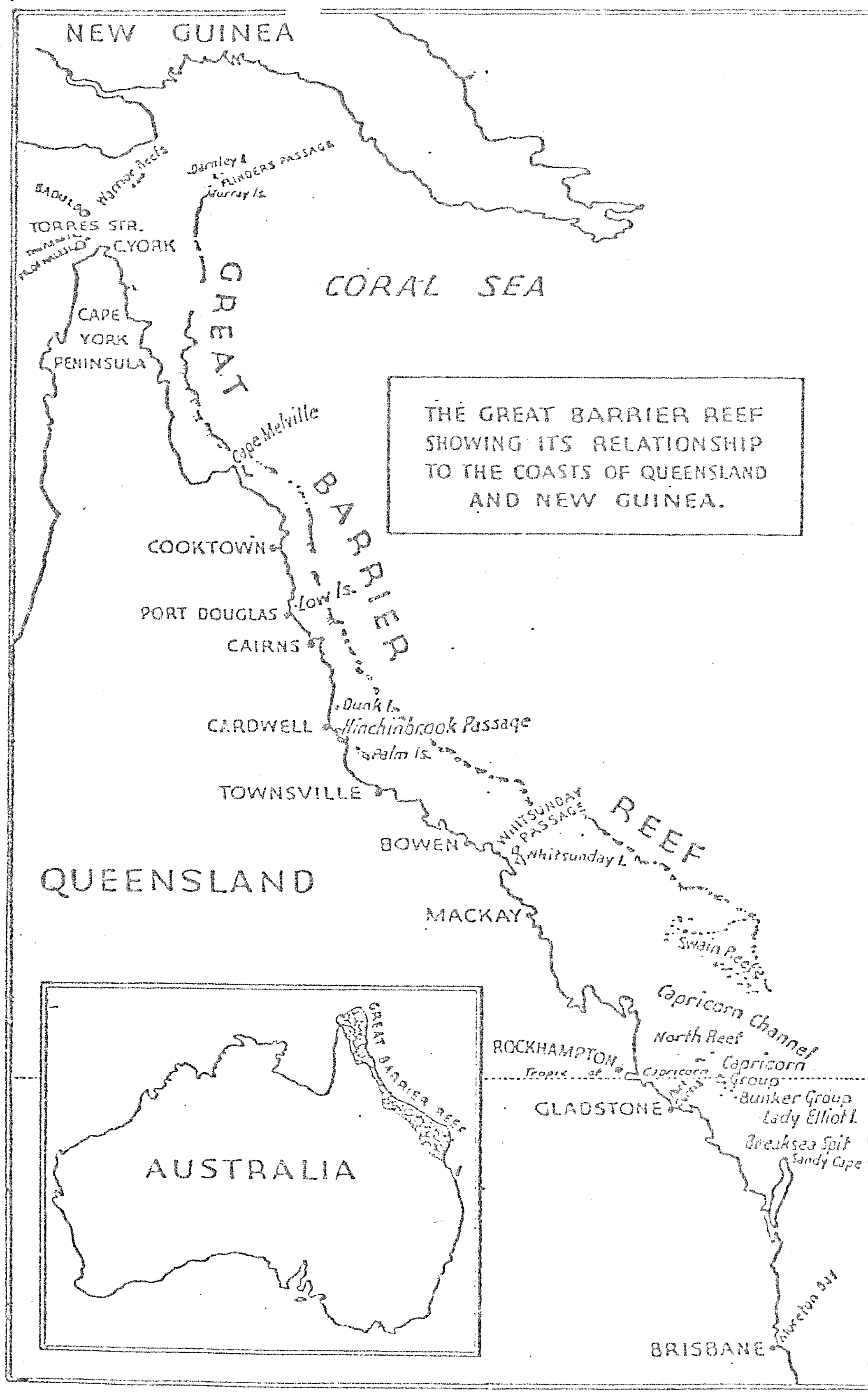


FIGURE 1.1 The Great Barrier Reef
(After Roughley, Ref. 4).

CHAPTER 2

REEF CLASSIFICATION AND DESCRIPTION

2.1 Introduction

Coral reefs develop as a result of the growth of tiny lime secreting plants and animals. The plants and animals require a certain amount of nutrient and oxygen, as well as suitable degrees of temperature and light penetration in order to survive. The proximity of the ocean floor to the surface of the water, along with the regional hydrology, are the main factors determining the degree of temperature and light penetration. The other requirements, such as nutrient, degree of aeration and amount of carbonate in the water, are supplied by the cold ocean currents sweeping up over the continental slopes from the deep ocean basins.

All of the above requirements are essential for reef growth, but varying degrees of them give rise to differing reef formations. When considering engineering structures on reefs, the type of reef formation present is important since it can assist in determining the condition of the coral mass, the condition of the lagoon floor, the presence and extent of individual coral masses within the overall structure, and other related factors that have to be considered in the design and construction.

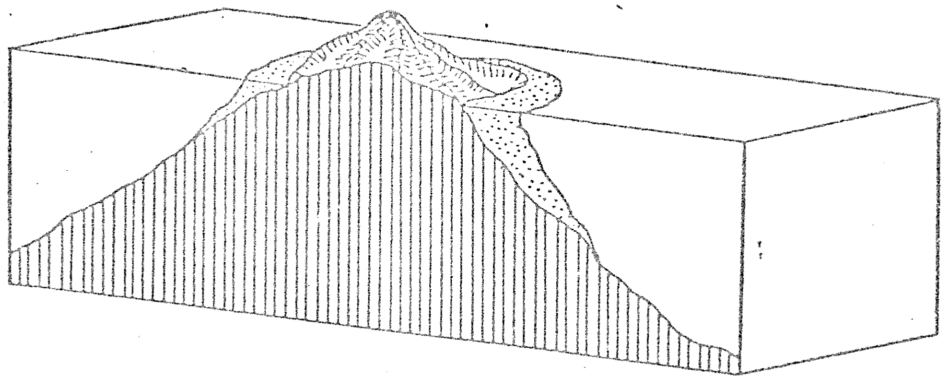
2.2 Darwin's Classification

Charles Darwin first proposed a reef classification theory in 1898, in which he divided reefs into three groups: fringing reefs, barrier reefs, and atolls. His classification was based on a subsidence theory that defined the three types of reefs as stages of an evolution process as shown in Figure 2.1.

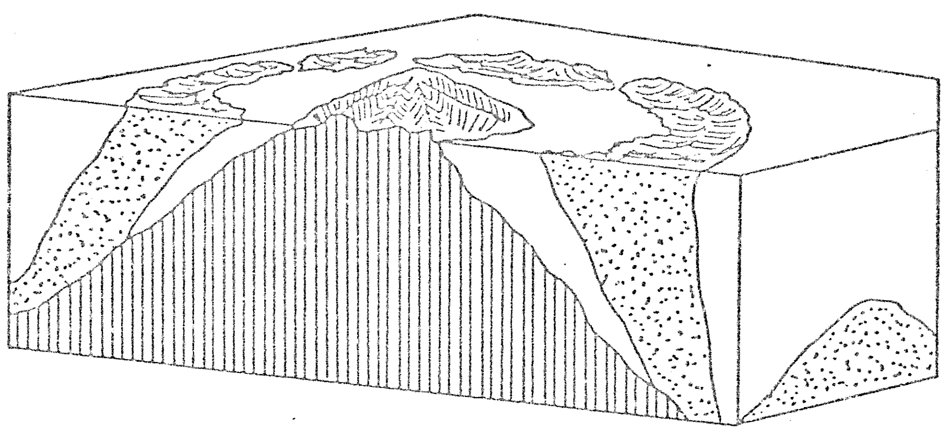
Fringing reefs are essentially organic extensions of insular shores (i.e. island shores). They form, when the various coral organisms and algae necessary for reef growth become established along rocky coasts. Fringing reefs are generally fairly narrow, flat platforms which can be awash or even exposed at low tides. The reef formations on places like Lizard Island and Palm Island are examples of fringing reefs.

Barrier reefs form due to further subsidence of the island in the fringing reef case. The coral platform becomes separated from the island shores by a relatively shallow body of water called a lagoon. The water in these lagoons is generally very calm since most of the energy of approaching ocean waves is dissipated on the reef. When the barrier reef is an appreciable distance from the shore it may fail to act as a natural breakwater, since the distance of exposed water within the lagoon is sufficient to form wind-generated waves of significant size. The Great Barrier Reef is an obvious example of such a situation. In a situation like this, it is possible for fringing reefs to develop along the coastline behind the barrier reef. There is virtually no limit to the size of a barrier reef.

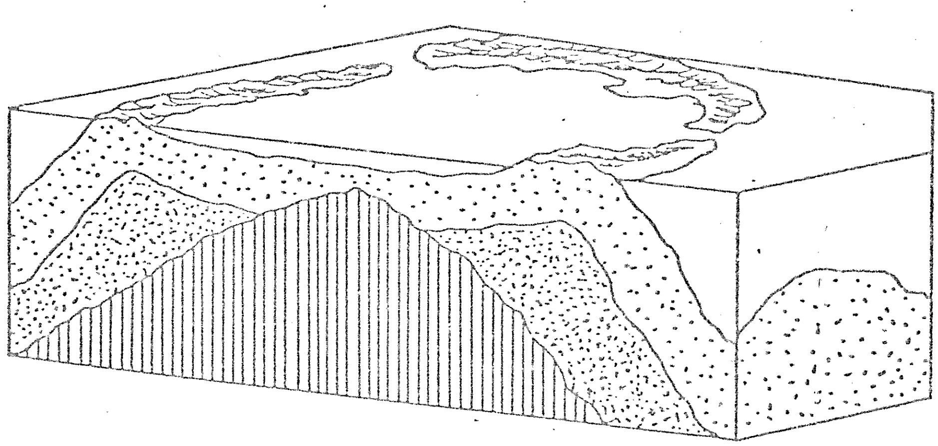
Attols are annular-shaped reef islands that may or may not completely enclose a lagoon. The majority of the atoll is either submerged or partly awash at low tide, although there are usually small islands or islets that project above sea level. These usually form less than one per cent of the total atoll area.



Fringing Reef.



Barrier Reef.



Atoll.

FIGURE 2.1 Darwin's Subsidence Theory

The atoll is usually breached by one or more deep channels that are found in the leeward or sheltered side of the atoll. The waters enclosed in the lagoon are always placid and the channels provide natural entrances to these quiet anchorages. There is a general relationship within the lagoon between water depth and atoll circumference. Many Pacific atolls have an average diameter of 32 kilometres and maximum lagoon depths of approximately 90 metres.

2.3 Maxwell's Classification

Maxwell proposed a scheme in 1968 that divided reefs into two broad categories: Oceanic and Shelf Reefs. Shelf reefs may be further subdivided into platform reefs or wall reefs as shown in Figure 2.2.

An oceanic reef has an evolution process that is the same as the Darwin theory. (See Figure 2.1). That is, an oceanic reef forms in a ring around an island in the middle of the ocean and evolves into a barrier reef which eventually becomes an atoll.

Continental or shelf reefs are usually found in the shallow water of the continental shelf off the coast of continents. These reefs have the same coral types as oceanic reefs, but the coral structures that are formed vary widely. Oceanic reefs are formed with a minimum of land area available for coral growth, but with an unrestricted food supply, carried by currents from the surrounding ocean depths. On the other hand, shelf reefs are formed with an unlimited surface area on which to build, but with a limited food supply, since the food supplying currents are restricted by the continental masses.

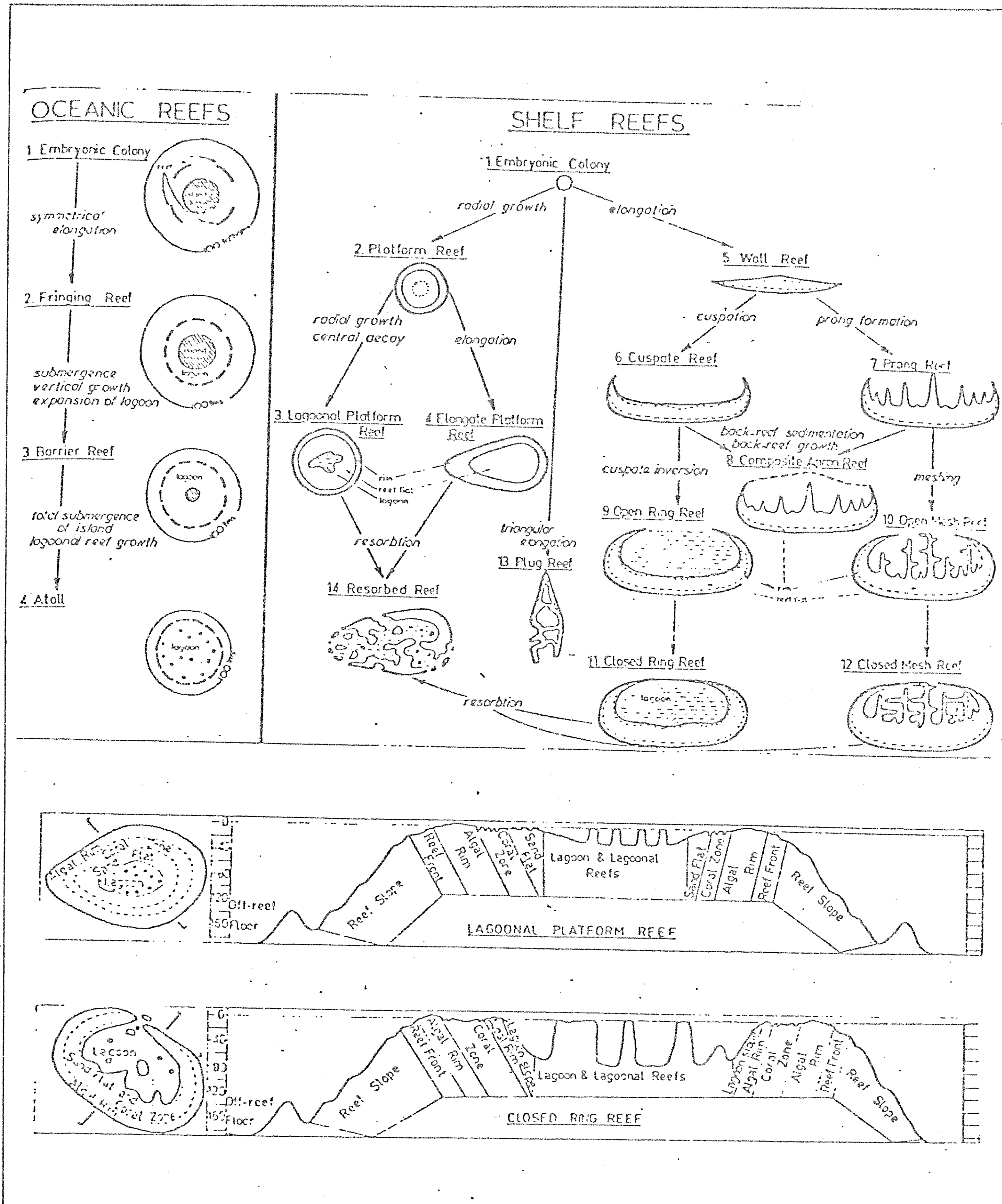


FIGURE 2.2 Maxwell's Reef Classification

The Great Barrier Reef as a whole can be classified as a shelf reef, but the numerous individual reefs constituting its overall structure can be subdivided according to their formation. The individual formations are controlled by growth restricting factors that are dependant on the environment.

A platform reef forms when the growth controlling factors are equal in all directions thus giving rise to an approximately circular platform. A lagoon eventually develops since the inner reef becomes more and more sheltered from the waves and currents, which means that less nutrients are available for growth, thus causing the outer reef parts to grow faster than the inner. Well developed lagoons may contain small reefs that are independant of the original reef structure. The size and development of the lagoon can be used to estimate the age of the reef.

A wall reef forms when a hydrologic or topographic feature causes the reef to develop in one direction only. This type of reef can be changed further by differing hydrological factors into a cusped shape or a pronged reef as shown in Figure 2.2. A pronged reef forms in areas of high tidal influence where strong currents are generated between reefs thus stimulating outward growth due to the ready supply of nutrient and well aerated water. Both cuspate and prong reefs develop into a closed ring structure with a lagoon in the centre.

Distinguishing between a platform reef and a wall reef is very easy when both types are at an early stage of growth and the wall reef has not yet formed a well developed closed ring. When the reefs are mature, the major discernible differences are the depth of the lagoon and the development of a protective layer called an algae rim.

An algae rim is a protective growth of red algae that grows on the seaward or open side of the reef. The inner lagoonal side of a platform reef has no need of protection so therefore no algae rim forms, while the inner side of a closed ring reef is exposed to the open sea for a short time which causes an underdeveloped algae rim to form. Also, the lagoons of platform reefs are significantly shallower than those of closed ring reefs.

A mature reef will eventually develop into a sand cay and then a coral island unless unfavourable topographic or hydrologic changes take place that could cause the reef to degenerate. This degeneration is called the resorbtion stage and is characterized by scattered reef projections left by the coherent shape of the reef as it disappears.

2.4 Lithification

Lithification describes the process which changes the uncon solidated mass of coral, algae, and animal deritus into a coherent rock mass. There are two types of rock found in the reef mass: beach rock and reef rock. Reef rock forms the greater proportion of the mass while beach rock forms a shallow layer overlying it. The layer of beach rock is generally not well developed and usually only occurs around islands where conditions for its development are favourable. So therefore, most of the reef mass is composed of reef rock covered with a thin layer of sand or living coral.

Lithification occurs by several processes that are not directly related to each other or influenced by the same factors. Recrystallization is the most effective of these processes, while compaction plays a lesser role and cementation by precipitation plays a subordinate role to both of the above.

Recrystallization is by far the most dominant process in the formation of shallow reef rock, since compaction is not effective due to the shallow depths of overburden involved. Basically, the process involves the recrystallization of aragonite, an unstable polymorph of calcite, into calcite, which in so doing gives a more coherent matrix structure. Leaching of the unstable aragonite inhibits the matrix recrystallization and causes the formation of cavities and unconsolidated sand pockets, in what is otherwise a coherent rock mass. These imperfections can be of critical importance when designing the foundations of structures on a reef mass.

Lithification by compaction is most effective where there is a fair proportion of the matrix structure compressed into the inter-granular spaces. That is, the process usually occurs when the reef mass is at a late stage of its development. The same applies for the process of cementation.

When considering construction on a reef mass, it is only the top 10 to 30 metres that is of interest, or alternatively, the young portion of the reef. So therefore because of the time dependant nature of all the lithification processes, it stands to reason that in this region, these processes are only partly completed.

CHAPTER 3

TYPES OF STRUCTURES ON CORAL REEFS

3.1 Lighthouses

The Commonwealth Department of Construction has built numerous lighthouses on the Great Barrier Reef over the last fifty years. Figure 3.1 shows the location of lighthouses that are situated on bare reefs or sand cays. The majority of them are situated in the north of the state where the reef is closest to the coast, thus posing the greatest risk to shipping. Those towers that will be discussed specifically in later chapters are indicated on the map.

The Department of Construction classifies reef environments into three different categories:

Type A - Coral reefs completely submerged at high tide and either dry or submerged by 5 to 10 feet at low tide. Figure 3.2 shows a tower in such an environment at Waterwitch Reef.

Type B - Similar to type A but with a sand cay on the reef. The cay may not be permanent but at this stage is above high water level.

Type C - Similar to type B but land area is stable, fairly flat, of low elevation and with tree growth (Figure 3.3).

The majority of existing light towers are lattice-type towers in piled foundations. Up until recently, these towers were almost exclusively constructed of painted or galvanised mild steel, but recent trends have been towards stainless steel since this material is virtually maintenance free in a marine environment.

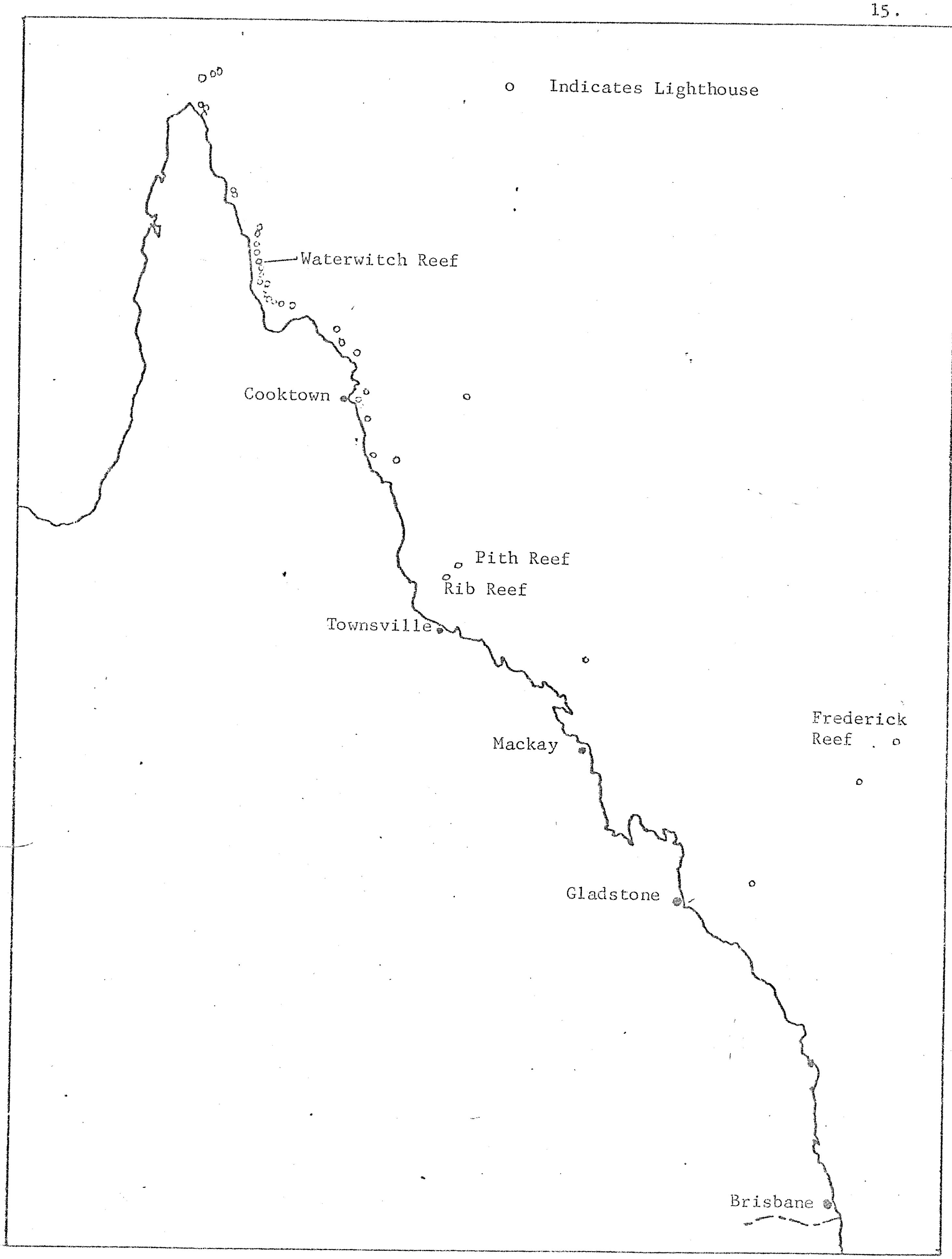


FIGURE 3.1 Location of Lighthouses on the Great Barrier Reef

FIGURE 3.2 Lighthouse in a Type 'A' Environment

(Waterwitch Reef)

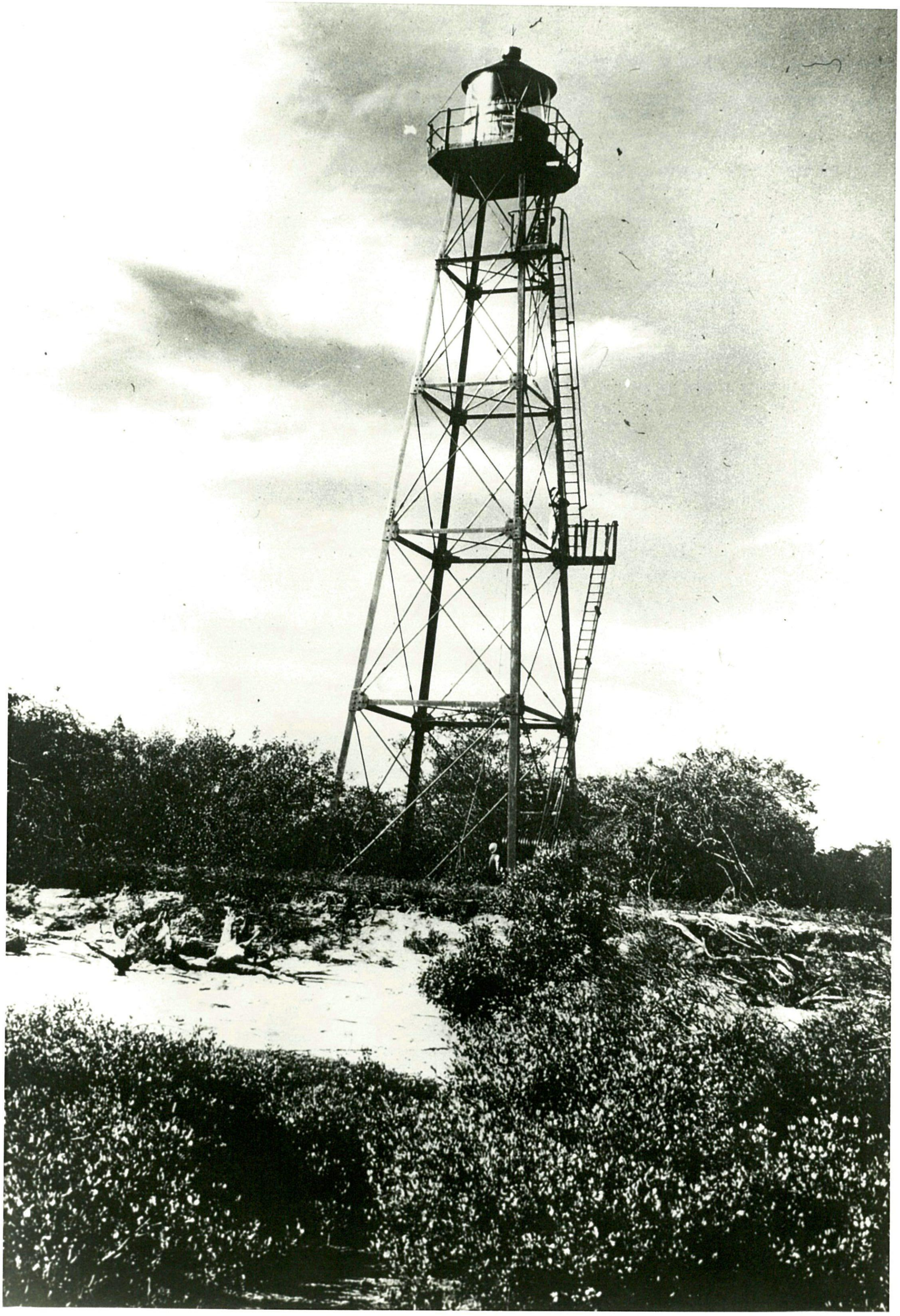


FIGURE 3.3 Lighthouse in a Type 'C' Environment

(Hanah Island)

Several mild steel and stainless steel free standing tubular towers have also been constructed. This type of tower is only suitable for construction on sand cays or small islands since the tower is transported in sections bolted or welded together on the ground and lifted into position, and in order to do this a level site with adequate working space is required.

3.2 Weather Stations

There are several unmanned, automatic weather stations situated on reefs off the Queensland coast. These stations are also designed and built by the Department of Construction and their function is to collect weather data for analysis by the Bureau of Meteorology. These structures are constructed of anti-corrosive stainless steel with piled foundation and with a typical form and dimension as shown in Figure 3.4.

3.3 Radar Beacons

Radar beacons are probably the simplest structures situated in the Great Barrier Reef. They act as navigation aids to mark various reefs and to indicate safe passages between them.

The design and construction of these structures is very simple and usually consists of a single mast with a reflector at the top. Cameron (1975) designed one such beacon on Keeper Reef off Townsville, as a part of his final year thesis and a photograph of it is shown in Figure 3.5. The beacon is 6.6 metres high with a reflector on the top made of three 1.73 metre squares of insect screen wire with light aluminium tube framing, placed in the X, Y and Z directions.

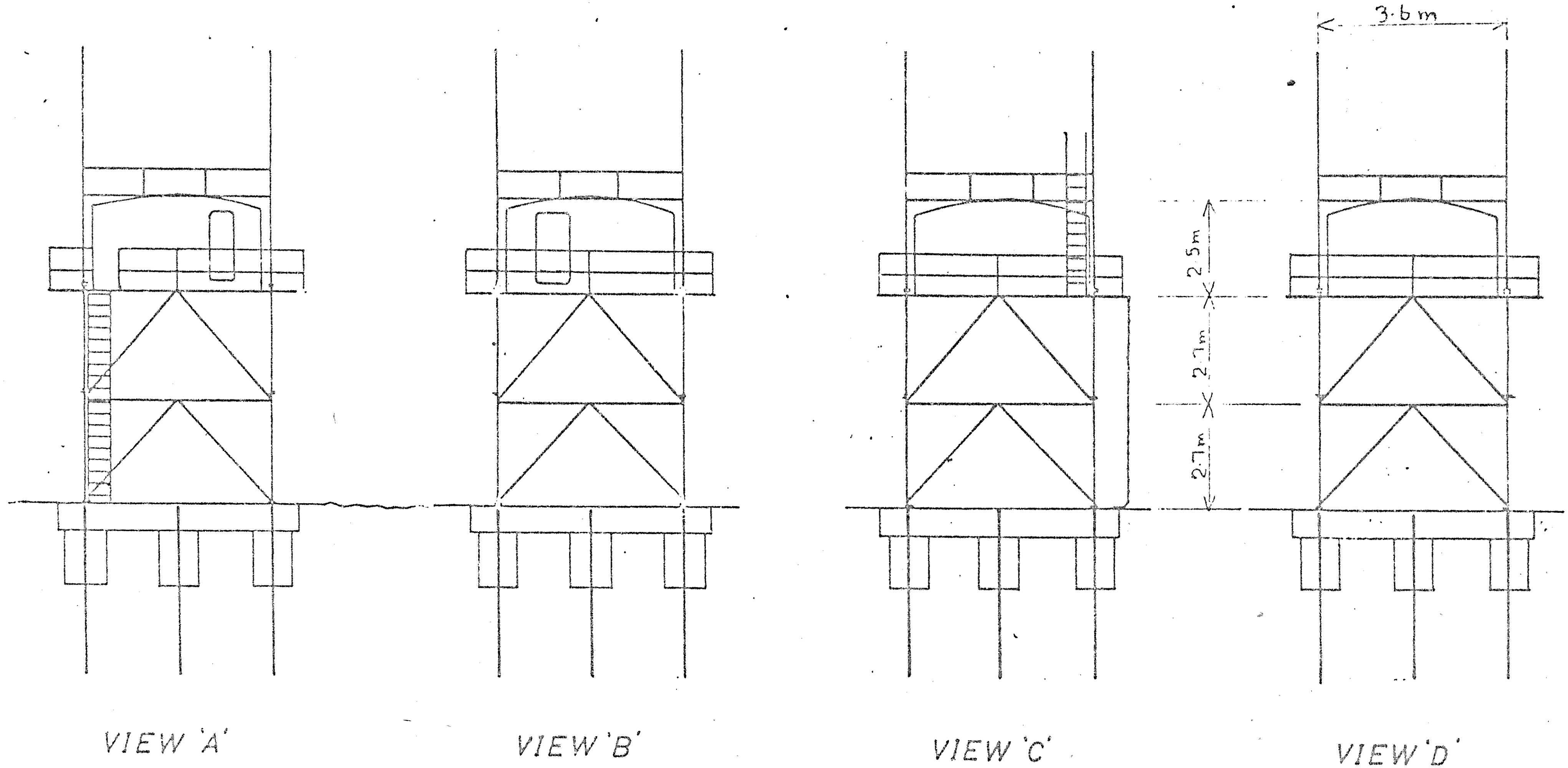


FIGURE 3.4 Elevations of a Typical Automatic Weather Station

The beacon has a range on radar of approximately thirteen kilometres.

Navigation through the maze of reefs that constitute the Great Barrier Reef has always been a major problem and a system of radar beacons such as the simple one described above would be all that is required to provide a suitable navigation system. The Department of Mapping and Surveying is contemplating such a scheme, that would involve the construction of hundreds of these beacons. The beacons would be placed all along the reef thus enabling safe navigation through it at any point.

3.4 General Marine Platforms

A marine platform is the general name given to offshore structures, other than towers, that have one or more levels of flat working space. An automatic weather station is a marine platform on a small scale. Marine platforms are used extensively in the petroleum industry for such things as oil drilling, offshore storage and loading complexes, and scientific research stations. These operations usually involve construction in deep water, whereas construction on the Great Barrier Reef would involve no more than 10 to 12 metres of water and in most cases less than 6 metres.

Marine platforms are used extensively by the United States Coastguard in place of their light ships, which were previously permanently moored at the required position. Figure 3.6 shows a typical design of these platforms. No large scale platforms have yet been built on the Great Barrier Reef, but structures similar to the one in Figure 3.6 would be quite feasible.

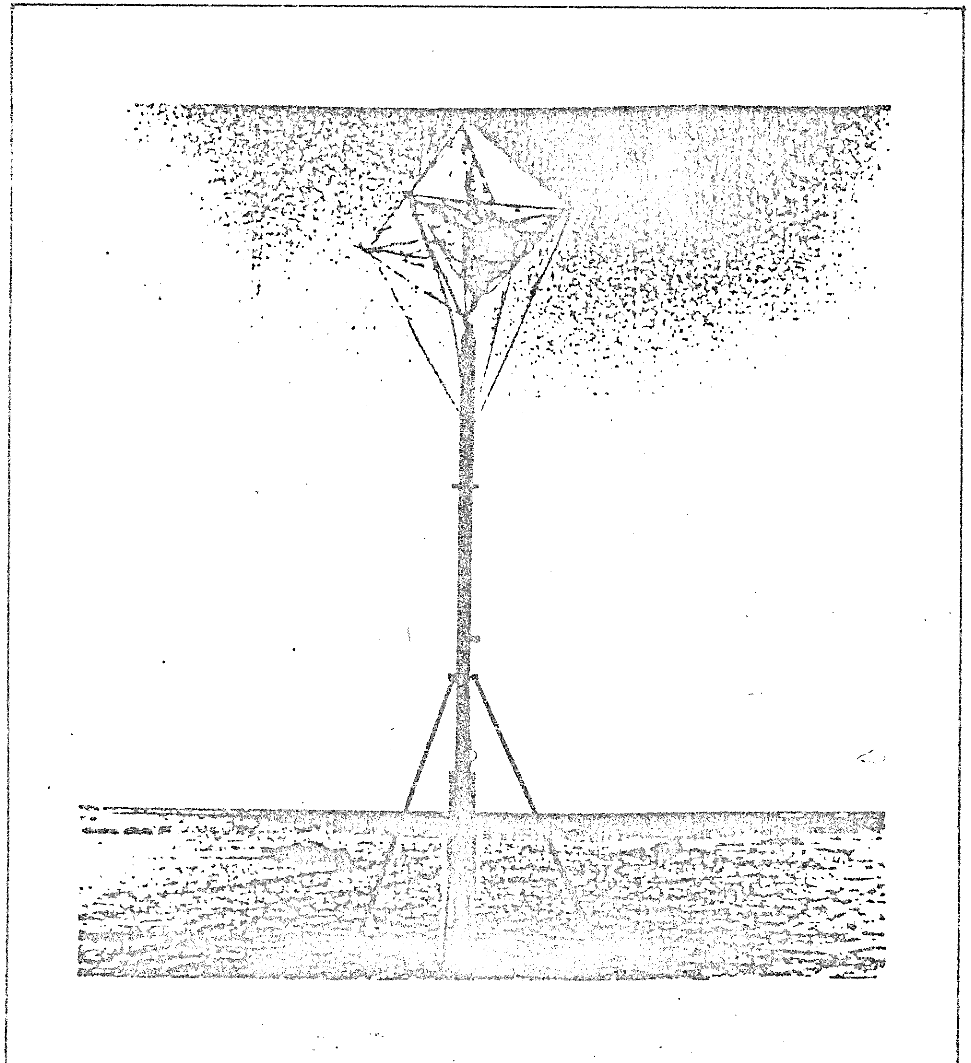


FIGURE 3.5 Radar Beacon on Keeper Reef

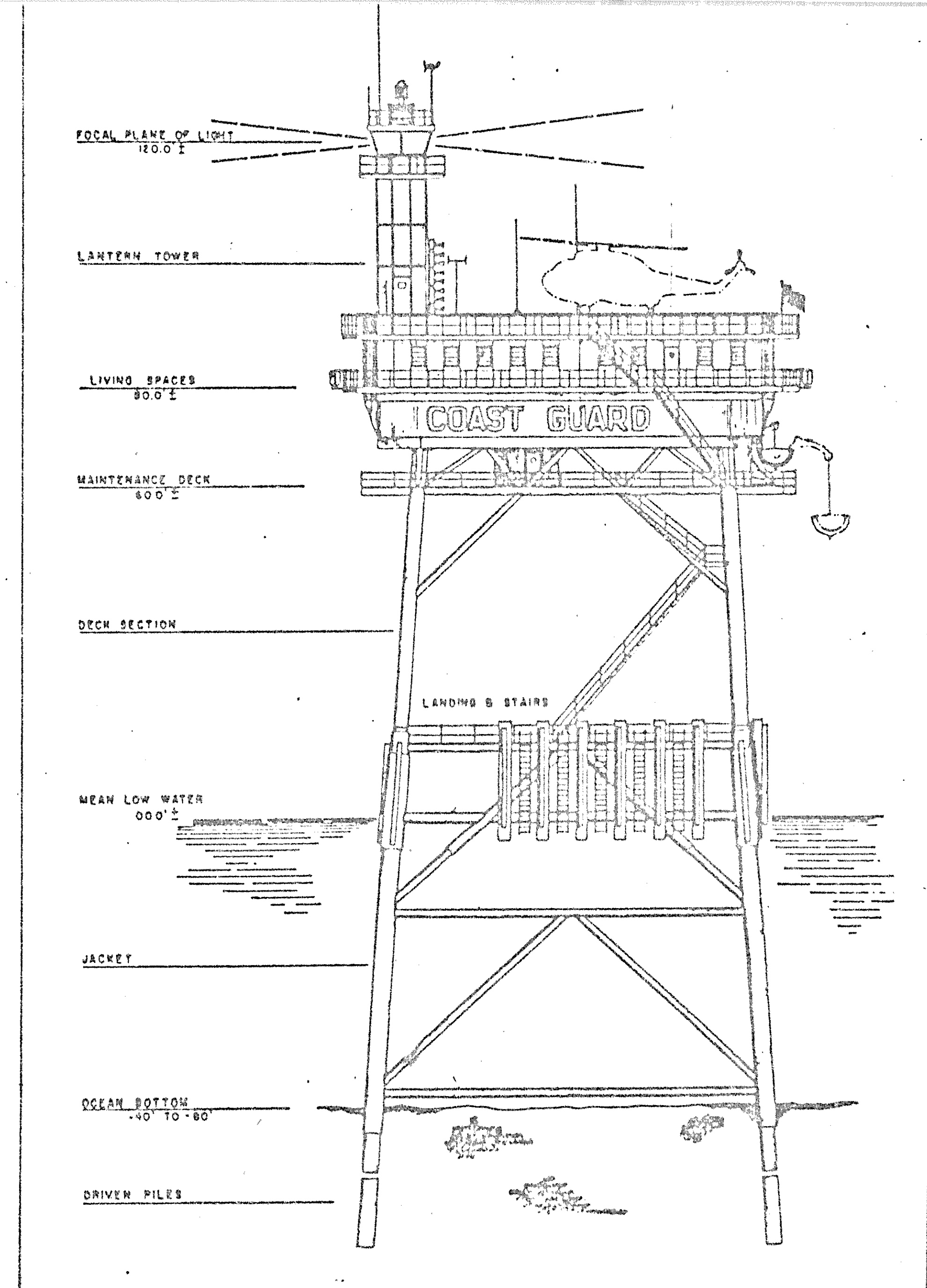


FIGURE 3.6 United States Coastguard Tower
(After Offshore Technology Conference 1975, Ref. 5)

The Australian Institute of Marine Science (AIMS) has obtained a preliminary design for a large scale research station on Britomart Reef, off Townsville. This structure will involve a self-contained platform large enough to accommodate up to six people for several months at a time, and with mooring facilities for different size vessels. A helicoptor landing pad will also be incorporated in the structure. The preliminary design was carried out by McIntyre and Associates, Consulting Engineers of Townsville, and at this stage no definite decision has been made by AIMS on whether to go ahead with the project.

Cameron (1975) carried out a preliminary design for a marine platform situated in a lagoon area of Broadhurst Reef. He proposed a basic framed structure on a caisson type foundation as shown in Figure 3.7. A caisson type foundation was practical due to the fact that the water in the lagoon is shallow and the bottom is flat and sandy.

The designs for ocean platforms are influenced greatly by the specific environment in which they have to operate. Construction of fixed platforms is affected by the geology of the bottom, water depth, currents, tides and the state of the sea.

3.5 Tourist Resorts

There is only one tourist resort on the reef proper in Queensland and this is on Heron Island, east of Rockhampton, in the Capricorn group of reefs. Other resorts that boast live coral would better be classified as offshore islands with small fringing reefs (e.g. Hook Island, Lizard Island).

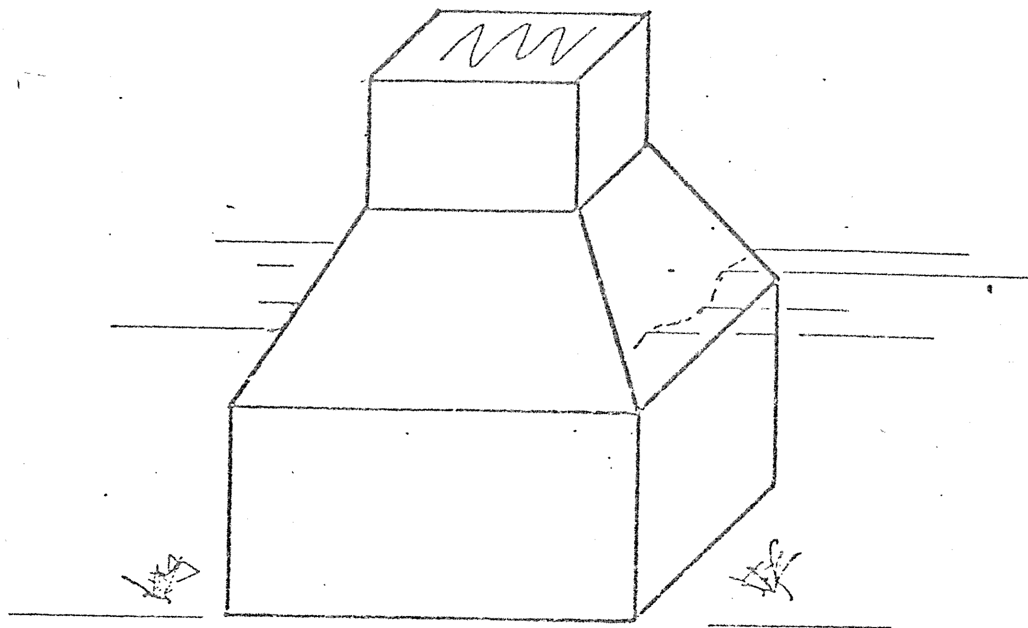


FIGURE 3.7 a. Caisson-Type Foundation

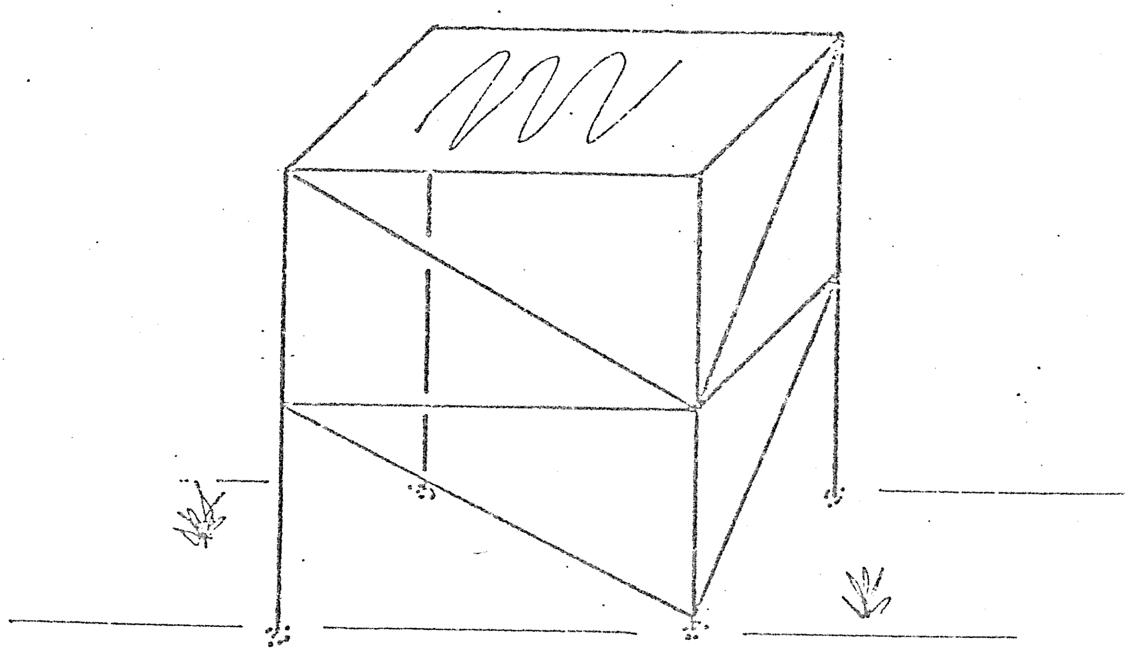


FIGURE 3.7 b. Basic Framed Structure

Although these types of resorts do allow tourists to view coral and associated marine organisms in their natural state, exploring the Great Barrier Reef itself is unsurpassed. When the full potential of this activity is realized, it is inevitable that tourist resorts on the reef itself will become much more commonplace. Although the development of the tourist potential of the reef should be encouraged, it should also be sensibly controlled so as not to spoil the natural beauty as has been the case in some parts of the world.

CHAPTER 4

GENERAL PROBLEMS ASSOCIATED WITH DESIGN AND CONSTRUCTION OF ENGINEERING STRUCTURES ON CORAL REEFS

4.1 Remoteness of Site

Although the distance of the reef from the mainland is only thirty miles on the average, the distance of a proposed site from an adequately sized part can be quite large. This creates numerous problems in the actual design of a structure on such a site, and also in the planning of the overall operation. The economic aspect of the project is probably the most important factor that arises from this.

The cost involved in transporting men and equipment to the site can be quite staggering when one considers that this cost is incurred before anybody even sets foot on the actual site. The size and number of vessels required obviously depends on the scale of the project and the distance of the nearest suitable port. Conversely, when designing the structure, it is desirable to know the type of vessels that will be available in order to ascertain what sort of equipment can be taken out and to determine the size of any pre-fabricated sections that may have to be used.

If the site of a proposed structure is an appreciable distance from a suitable port, it may be most economical to engage the services of one large boat that takes all the men, materials and equipment out in the initial trip. The boat would then anchor at some safe spot close to the site, and smaller barge-type vessels would be used to transport men and equipment onto the reef itself. These vessels could be carried on the deck, or in the hold of the mother ship.

Of course, this method depends somewhat on the condition of the actual site as discussed in Section 4.2.

An alternative scheme that could be employed if the site was relatively close to a port, would be to have several small boats transporting the equipment and materials out as they are needed. The advantage of this is that only one small vessel has to remain idle at the site to act as a base, while any others are used when they are needed. This means that less money is lost on idle equipment and labour (i.e. the boat and its crew). Two boats would probably be sufficient for the whole operation and, as before, small barge-type vessels would be used to get onto the actual site. If these vessels won't fit on the boats, they could be towed behind, since the distance to the site is small. In an operation such as this one, good planning is essential, because if materials did not arrive on schedule it would mean lost money due to workers and equipment at the site being idle.

Regardless of what method is used, the cost associated with getting men and materials to a reef site can be a vital factor when determining if a proposed structure is economically feasible.

4.2 Condition of Actual Site

Construction on a completely or partly submerged reef can present numerous problems, but it is generally something that cannot be avoided when the proposed structure is on the reef proper. The condition of the reef itself can be a deciding factor in the choice of a site. For example, it may be desirable to have the structure at a certain spot on the reef, but because the reef surface was never dry or was very rough and uneven thus making it extremely difficult to work on, the site may have to be moved to a more favourable position. On the other hand, due to navigational requirements it is not always possible to position a proposed light structure in the area that will give the simplest and most economical foundations.

As mentioned in Section 4.1, barge-type vessels can be used to get men and equipment onto the reef itself, and this usually has to be done at high tide. This is due to the fact that the leeward side of most reefs is scattered with numerous, isolated coral outcrops or bommies that are generally very close to the surface of the water at low tide. In fact, it is not unusual for them to be actually sticking out of the water, thus making movement on the reef at any time other than high tide very difficult and dangerous.

4.3 Dependability on the Weather

Without a doubt, the weather is the most unpredictable of factors controlling work in a coral reef environment. Unfavourable weather conditions mightn't necessarily stop work completely, but it can lead to a decrease in the standard of the work carried out and cause friction among the labour force, due to the uncomfortable working conditions. Bad weather conditions can mean a delay in the supply of materials to the site, and this possibility should be considered when planning the operation. In extreme cases, adverse weather conditions may completely halt work, and if the construction is at a critical stage, the structure may be in jeopardy unless preventative measures are taken.

CHAPTER 5

FOUNDATION DESIGN

5.1 Site Investigation

5.1.1 Introduction

A detailed site investigation is one of the basic requirements for any engineering project, including offshore structures. A proper site investigation involves a determination of the stratigraphy of the area in question, and a determination of the engineering properties of the soil or rock present.

Core drilling is probably the best method for determining the stratigraphy of a site, while pentrometer and seismic tests can be used to obtain an idea of the layers present and their relative strength. These methods are all quite straightforward when carried out on dry land, but when applied to offshore situations, they become much more complex.

To determine the engineering properties, a variety of laboratory tests can be used, either on their own or in conjunction with in-situ tests. Regardless of what sort of laboratory tests are used, core drilling usually has to be carried out beforehand in order to obtain a sample suitable for testing.

The fact that we are dealing with coral rock in a coral reef environment ultimately leads to problems never before encountered, that must be overcome to successfully determine the engineering properties of the rock.

5.1.2 Core Drilling Operations

A core drilling operation is probably the most effective way of determining the stratigraphy of a site and obtaining samples of the rock present. Drilling on a bare reef, whether it is partly submerged or completely submerged, presents many problems that have to be overcome.

Even though in some cases the surface of the reef may be dry at low tide, it is often very rough and uneven thus making setting up of the drill rig very difficult. In the case of a completely submerged reef, drilling would have to be carried out from a platform that could be either floating or semi-permanent. A floating platform would have to have an extremely effective anchoring system to keep the rig stationary thus reducing the difficulties involved in drilling. Varying types of semi-permanent structures such as mobile template platforms or self-elevating platforms can be readily utilized for drilling in coral reef situations. These type of platforms are discussed further in Chapter 7. Such methods are used extensively in the petroleum industry for both exploratory drilling and extraction of the oil. Appendix A indicates relevant papers and journals that can be consulted for a more detailed review of drilling methods employed for offshore situations.

Figure 5.2 shows a core drilling operation in progress on Keeper Reef, using a Minuteman Mobile Drilling Machine, Model M/M and under license by Fox Manufacturing Company. The drill rig is mounted on a raft, made up of a framework of ten 200 litre fuel drums arranged so that an opening is left in the centre for drilling. The complete raft measures 3.5 metres square and is 0.8 metres to deck level.

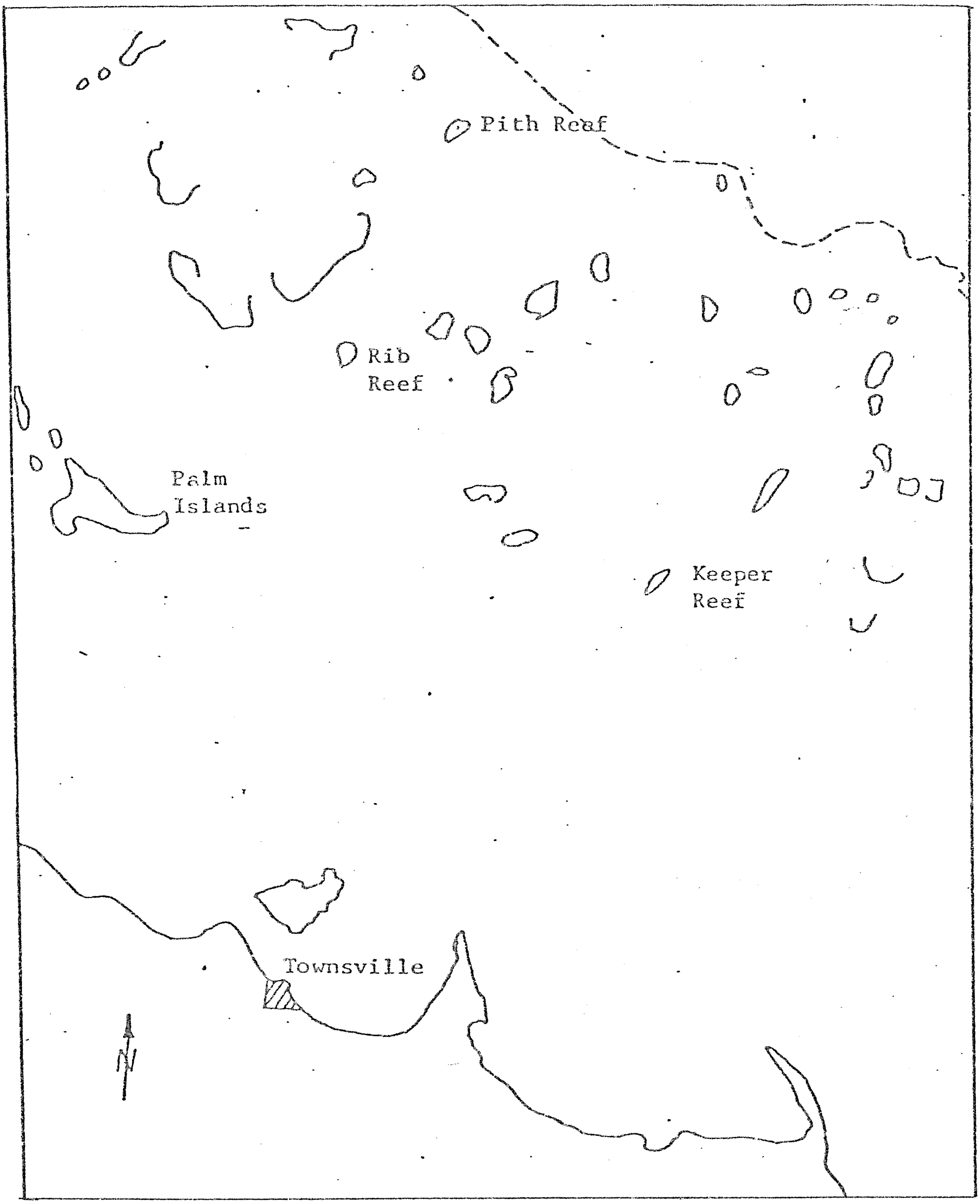


FIGURE 5.1 Location Map

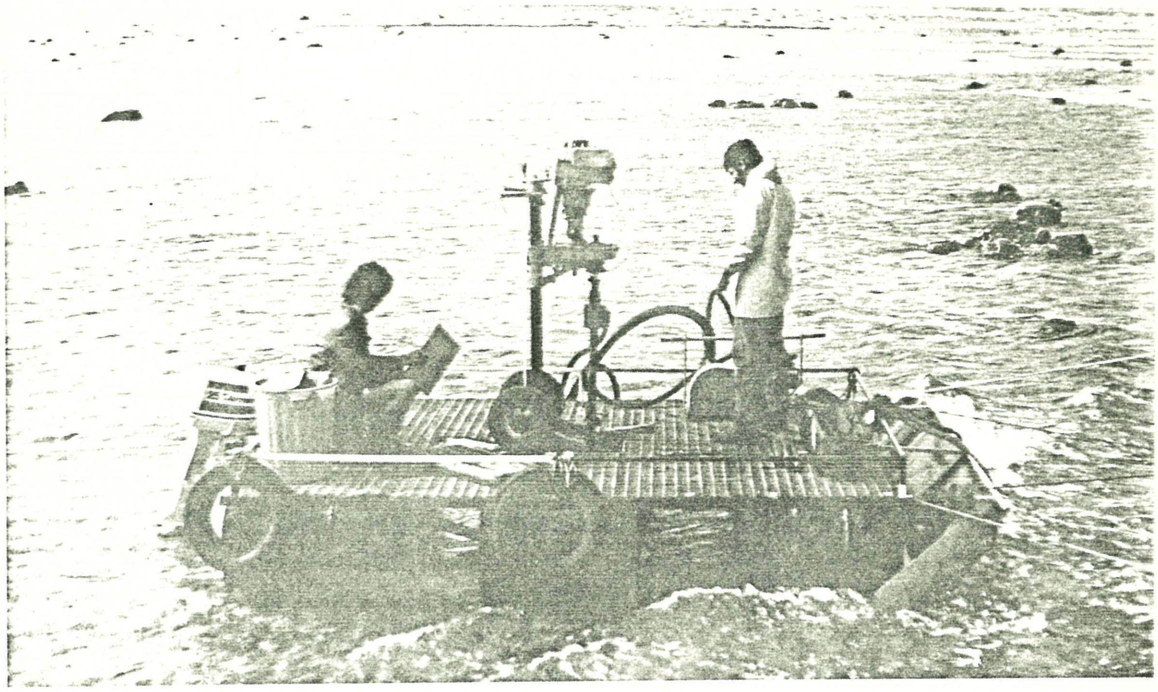


FIGURE 5.2 Core Drilling Operation on Keeper Reef

It is fitted with an outboard motor so that it can transport the rig onto the reef at high tide. The raft is then allowed to settle on the reef surface as the tide goes down so that drilling can be carried out from a stable platform. It would be quite possible to use this raft for drilling in a floating situation, as long as a suitable anchoring system was used.

A common problem encountered when core drilling a reef mass is the poor recovery rate often achieved due to the occurrence of cracks, fissures, cavities and pockets of unconsolidated sand throughout the mass. Moss (1976) describes a detailed core drilling operation conducted on Keeper Reef. He describes the problems encountered and recommends changes to improve the operation. Sand inflow was the main problem and a suggested improvement was the inclusion of a casing procedure to increase recovery. The average core recovery rate he achieved was only 12 per cent and the percentage of core suitable for testing was generally only 5.

In an endeavour to determine the thickness of the reefs and the nature of the foundation on which they are built, the Great Barrier Reef Committee (1942) has sunk two bores through coral reefs. The first one was at Michaelmas Cay, a small sand cay about 35 kilometres NNE of Cairns, in the centre of the Great Barrier Reef. Figure 5.3 shows the results of this bore. The second bore was on Heron Island, a flat, roughly oval, wooded sand cay in the Capricorn group of reefs east of Rockhampton. The results of this bore are shown in Figure 5.4.

The bores sunk by the Great Barrier Reef Committee were done by means of a Victoria Boring Plant of the rotary and percussion type. At both sites, there were unexpected difficulties encountered during boring; the most common being the striking of narrow hard bands

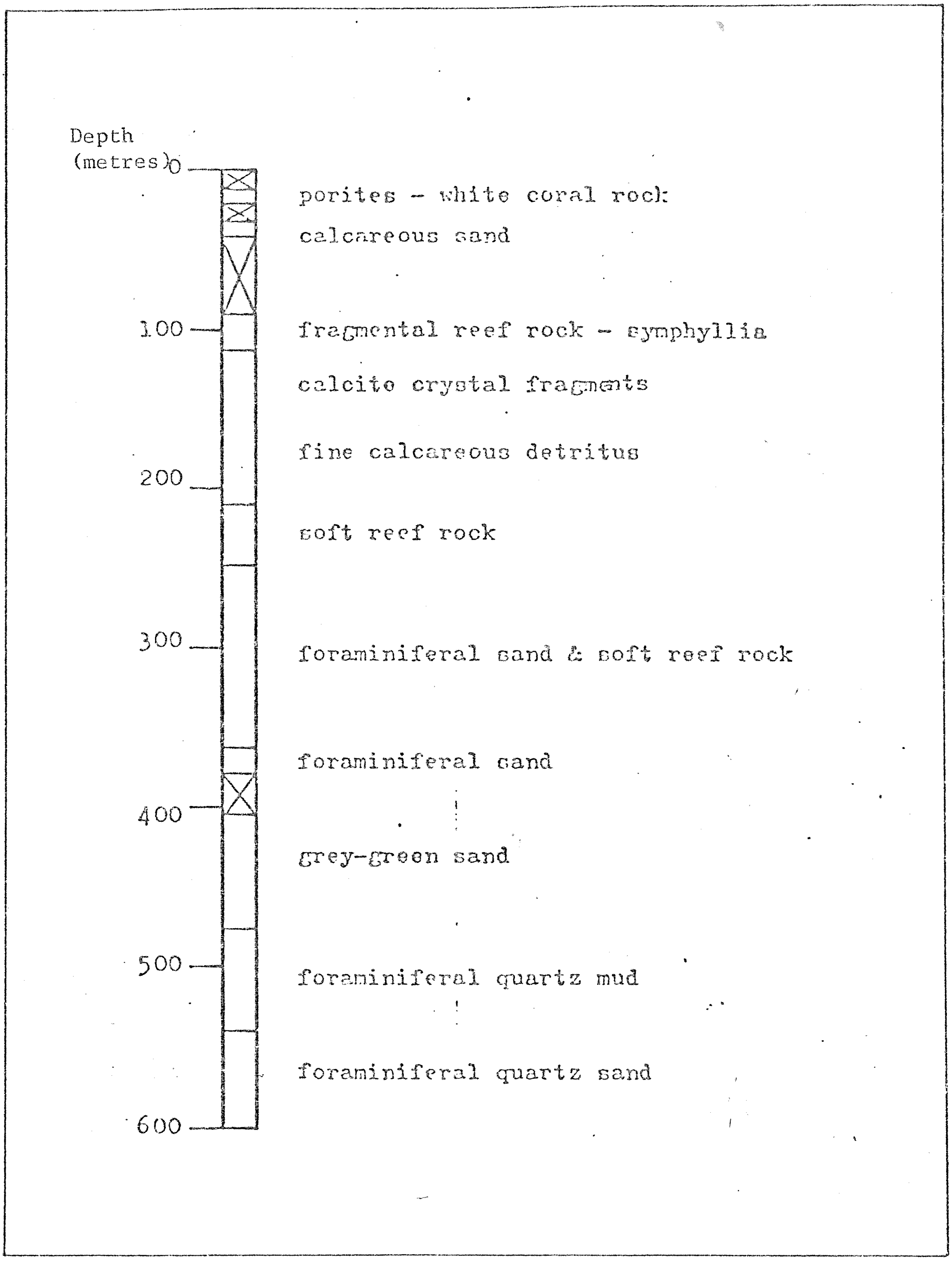


FIGURE 5.3 Sub-surface Profile obtained from drilling at Michaelmas Cay. (Blank sections indicate no core recovered.)

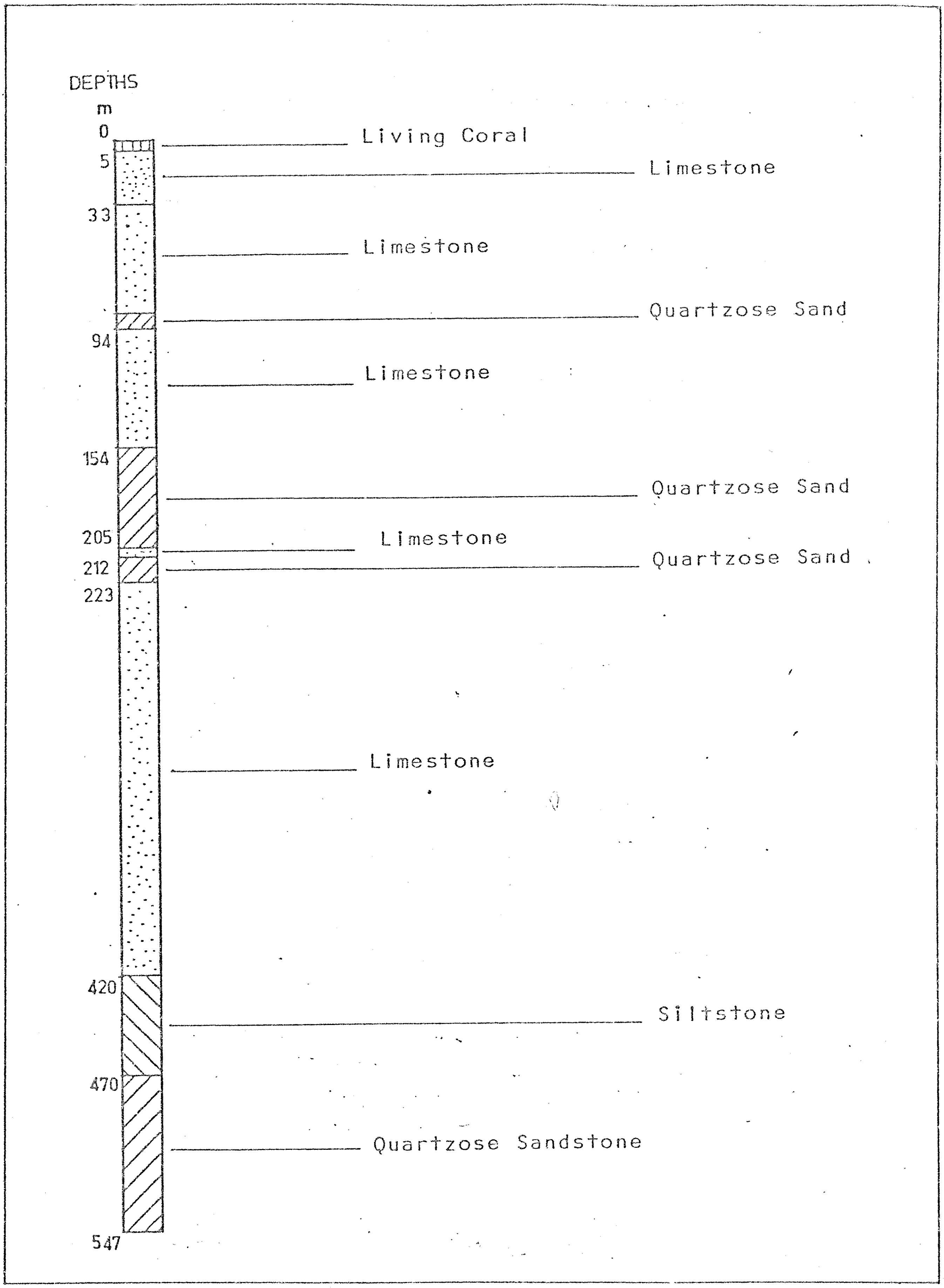


FIGURE 5.4 Core obtained from Heron Island.

through which the casing could not be driven, and reduction in sizes were called for more frequently than was anticipated or desired. The drilling in such a friable material for both sites necessitated the use of several lines of casing.

In 1947 four holes were drilled in Bikini Island by the U.S. Navy in co-operation with the U.S. Geological Survey (Emery, 1954). Also, in 1952, two holes were drilled into opposite sides of Eniwetok Atoll by the Atomic Energy Commission and the Los Alamos Scientific Laboratory in co-operation with U.S. Geological Society (Ladd, 1953; Ladd and Schlanger, 1960). Both of the bores at Eniwetok penetrated to sufficient depth to reach the basement rock at 1267 metres and 1405 metres. The drill used was a Frank Model 137/33, rotary type, trailer-mounted. Rock bits were used for straight drilling, and diamond bits for coring. In both Bikini and Eniwetok drillings, large amounts of soft rock were encountered, much of it being poorly cemented coral sand. Also, large caverns caused difficulty with loss of tools.

5.1.3 Penetrometer Tests

Penetrometer tests are generally not used for offshore site investigations, although tests have been carried out at Keeper Reef, off Townsville, using a Nordmeyer Heavy Dynamic Penetrometer with a drop weight of 50 kilograms and a full height of 50 centimetres.

The basic procedure behind these tests was to record the number of blows required to advance the rods ten centimetres, thus enabling you to get an idea of the relative hardness and strength of the layers. The tests are also very useful for indicating the presence of sub-surface cavities or weak layers consisting of pockets of unconsolidated sand. Figure 5.5 is the result of one such penetrometer test. This test indicates the presence of a very hard layer at a depth of approximately eleven metres with a lesser one occurring at six metres. The test also shows that the first metre of the reef mass is relatively harder than the following five metres, and after a depth of approximately 16 metres, the reef mass appears to be fairly uniform. Figure 5.6 illustrates the capability of the penetrometer tests to indicate the presence of sub-surface cavities. This test shows that there is an extremely large cavity at a depth of about 4.5 metres that could be critical in the foundation design.

The penetrometer tests were carried out with the rig sitting on a barge, the same as that used for the core drilling operation shown in Figure 5.2. The barge is held in position by means of winches at each corner, connected to anchors that are embedded in the coral. The winches are held taut in order to keep the rig stationary, and they have to be adjusted regularly in accordance with tidal fluctuations. Even though the barge is held in position by these winches, wave motion still caused appreciable up and down movements of the barge thus leading to slight inaccuracies in the readings.

Using the procedure described above as many as three tests per day could be carried out, and this rate could be easily increased if a more efficient procedure was formulated. Disposable tips were used in order to speed up the process by making the extraction of the rods much easier. The tests on Keeper Reef were carried out in situations where there was less than two metres of water, and in some cases the barge was actually sitting on the reef surface at low tide. This method could readily be adapted to deeper water with the only modification possibly being in the anchoring system.

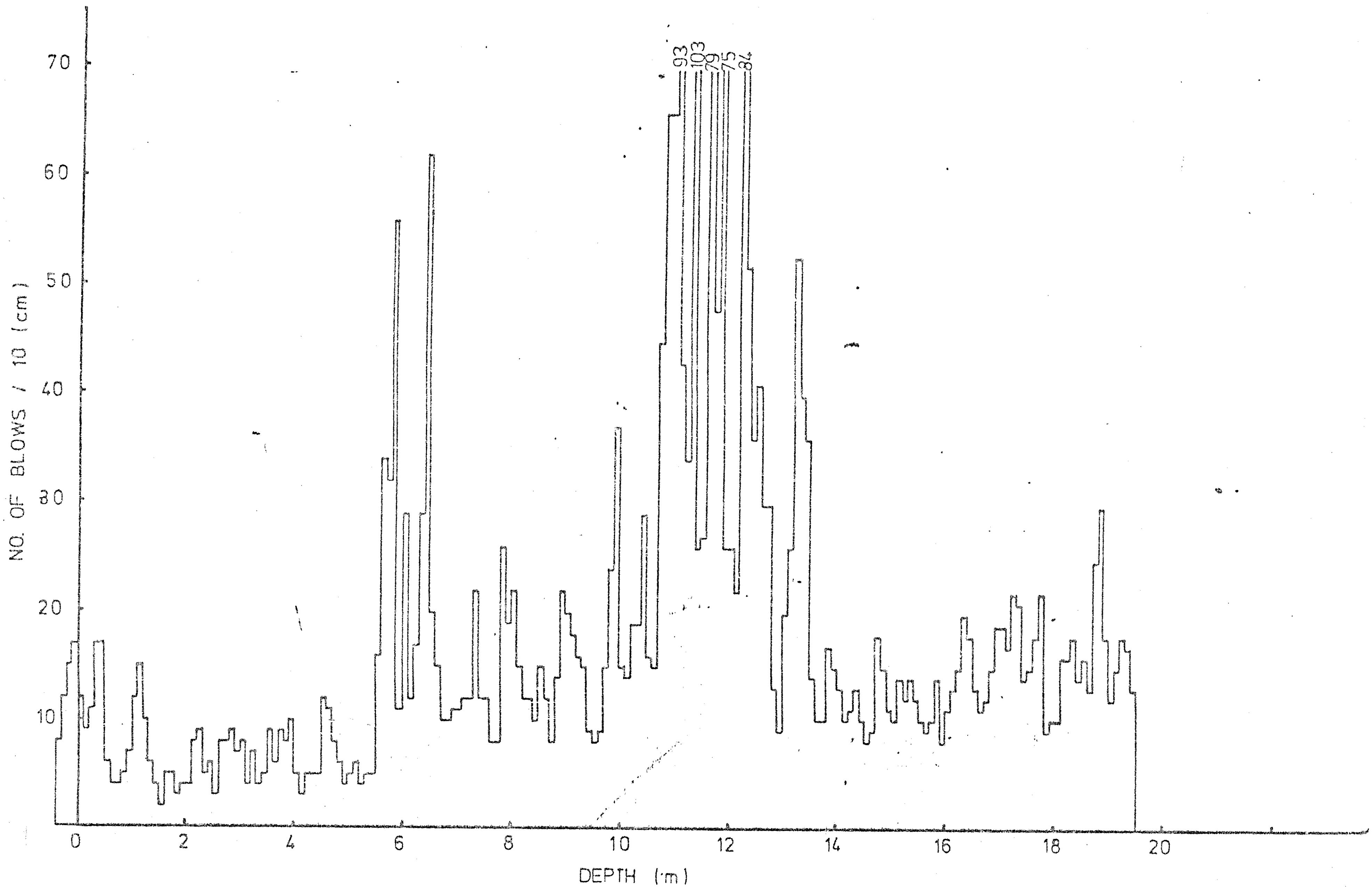


FIGURE 5.5 Penetrometer Test on Keeper Reef

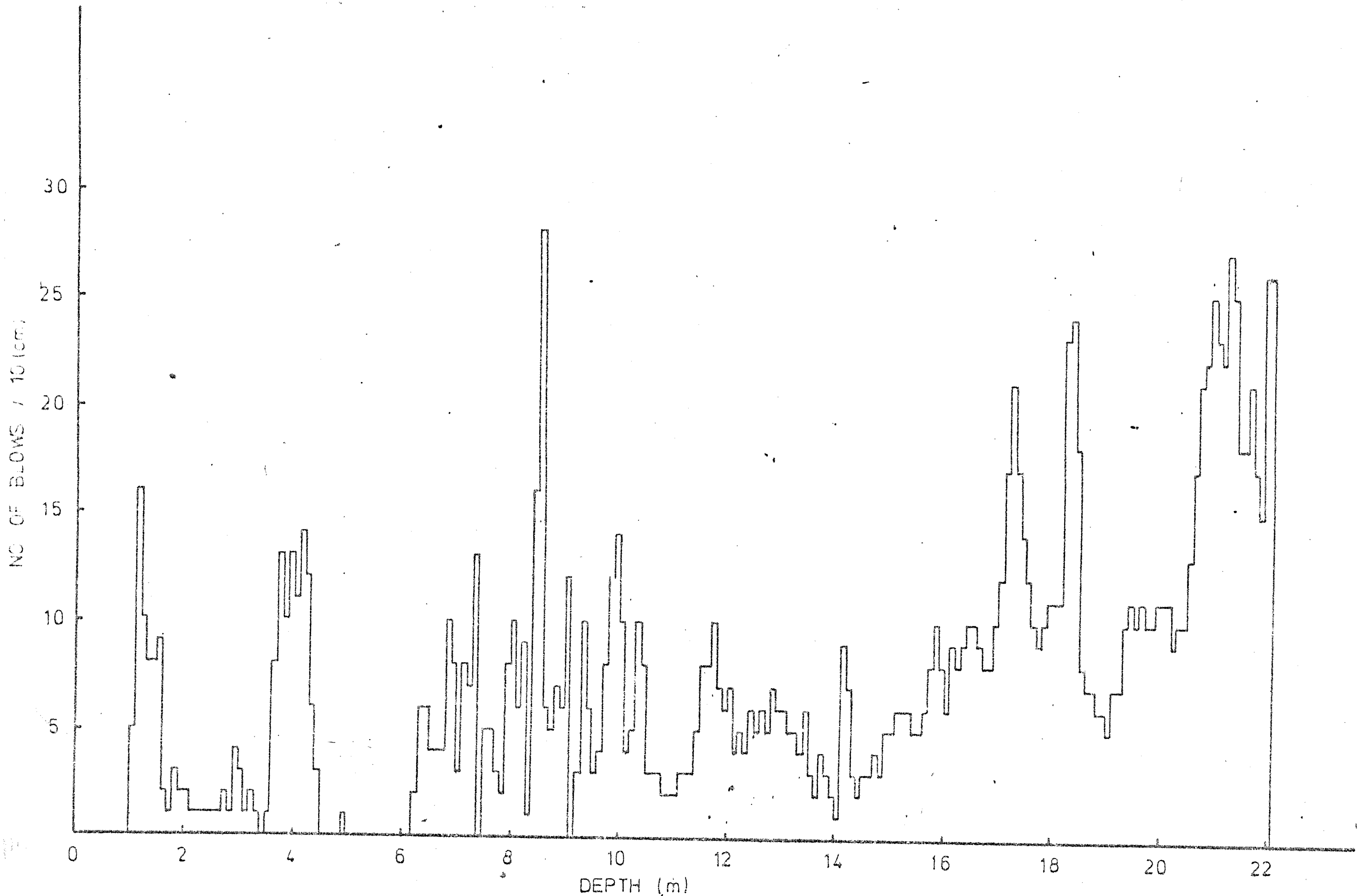


FIGURE 5.6 . Penetrometer Test on Keeper Reef

Penetrometer tests should be considered as giving a qualitative view of what lies under the surface, in order to assist in the design of the foundations and in detecting any problem areas that may exist. It should be noted that these tests can only be used to estimate the engineering properties of the coral rock, not to actually determine them.

5.1.4 Seismic Testing

Seismic testing can be used to determine the depth, thickness, and relative density of different layers present in the reef mass. The basic principle behind the technique involves the analysis of waves produced by a detonator after they have been refracted or reflected off the various layers. The waves are received by a geophone and their velocities can be calculated and related to the relative densities and thicknesses of the layers.

This method is useful for determining the presence of any particularly dense sub-surface stratums that may have to be utilized in the foundations. For example, tests carried out on Keeper Reef show that an uncomformity exists at approximately 20 metres. This unconformity is thought to be the interface between the low stand of the sea level and growth produced since then. This level represents a more lithified and coherent rock mass compared to the overlying layers. That is, the material is more dense when compared to the younger overlying coral deposits. Such a layer could be used as a safe bearing stratum for massive foundations that have very high loads placed on them. Seismic surveying can also be useful for indicating the presence of large sub-surface holes and bodies of unconsolidated sand that would have to be avoided in foundation construction.

Activity in the field of seismic research applied to coral reefs is very limited and much more is needed before the reliability of the technique can be properly evaluated.

5.2 Suitable Foundations for Structures on Coral Reefs

5.2.1 Introduction

Foundations for offshore structures can be broadly divided into two main categories: piles and spread footings. The special form that they take is dependent upon the condition of the site, the methods of construction available, and the loading that they are required to withstand.

The foundations of an offshore structure have to be designed for both horizontal loads produced by wind and wave forces and vertical loads due to wave forces and the weight of the structure. The horizontal loads produce a shear at foundation level as well as an obvious overturning moment. In deep water this overturning moment can be quite large and is due primarily to wave loads, whereas in a shallow water situation, such as a coral reef environment, the overturning moment due to wave loading is greatly reduced.

5.2.2 Piles

Pile foundations are used almost exclusively for the support of permanent offshore structures, including those on coral reefs. The last two decades have seen tremendous advancement in the technology behind the design and construction of fixed offshore platforms, due primarily to increasing offshore oil production. Areas such as the North Sea, the Florida coast, and Bass Strait, are dotted with fixed platforms used for various functions including, storage and loading facilities, primary treatment plants, and scientific research bases. These generally involve water depths of anything between 50 and 300 metres and situations where wind and wave loading can become quite severe.

The design of such structures usually consists of cross-braced hollow steel piles, while hollow cylindrical prestressed concrete piles are sometimes used for unbraced or free standing pile foundations. The steel pile-type structure can be readily applied to a coral reef situation whereas concrete piles in coral presents special problems that will be discussed later. The methods of construction and installation of such structures are described in detail in Chapter 7. A large number of publications that deal with the design of piles in a variety of different situations are documented in many journals, the best being the Offshore Technology Conference (OTC) series.

Myers (1969) suggests that pile types for offshore structures should be selected in terms of the following factors:

Suitability of bearing material

Length of pile required to reach this material

Character of structure to be supported

Character of loading (constantly static, intermittent, lateral impact, buoyant uplift, duration, etc.)

Availability of materials

Means of transport of piles and equipment

Factors causing deterioration

Amount and estimated costs of maintenance

Estimated economic life required

Estimated costs of piles and foundations (if not the same with different pile types)

Availability of funds

Obviously, all of the above factors apply to the situation of a structure on a coral reef, with particular emphasis focused on suitability of bearing material due to the limited experience in this field. Almost without exception, existing structures on coral reefs are supported by driven steel piles. Lighthouses constitute the majority of what can be termed substantial structures, situated on the Great Barrier Reef.

In 1975, the Department of Construction erected lighthouses on Pith Reef and Rib Reef, north of Townsville. (See Figure 5.1). Both sites were always submerged by at least one metre of water and the foundations were situated on bare coral.

The Pith Reef tower consists of a reinforced concrete base, 6.5 metres square and 1.8 metres thick, founded on the coral and anchored to four steel piles. Four concrete columns are cast on the slab and tied with a concrete ring beam at the top, 4.1 metres above the base slab. The actual tower is a 24.5 metre high, standard lattice tower sitting on the ring beam. Below is a brief description of the design values used:

Calculated uplift on piles	=	232 KN
Assumed friction	=	30 KPa (6000 psf)
Permissable bond stress	=	690 KPa (100 psi)

$$\text{Factor of safety} = \frac{\text{moment resisting}}{\text{moment overturning}} = 1.5$$

Required length of embeddment in pile cap = 224 mm

Piles required to be driven to a depth of 8 m.

The Rib Reef tower consists of a 3.5 metre diameter, 1 m thick reinforced concrete base founded on the coral and anchored to four steel piles. In the centre of the base is cast a cylindrical reinforced concrete column, 1.2 metres in diameter which is capped by a 4 metre diameter, 0.2 metre thick concrete platform. The platform is 9 metres above the base, and the actual light just sits upon it. The design values for the foundations of this tower are very similar to those used for the Pith Reef tower.

In both cases, 232 mm x 220 mm x 123 kg/m steel 'H' piles were used, driven by the drop hammer method. The weight of the hammer was 1.5 tonnes and the drop height was 1.8 metres. In both cases, the piles were driven to a depth of approximately 6.5 metres, where virtual refusal was encountered (compared to the design depth of 8m). This left a length of embeddment in the pile cap of approximately 1 metre, since 7.3 metre lengths were being used for driving (compared with required embeddment of 224 mm). The pile driving details from both sites were virtually the same, and they are shown below:

First 1.2 metres, penetration of 40 - 50 mm per blow

Next 3.7 metres, penetration of 230 - 250 mm per blow

Next 0.9 metres, penetration of 75 - 150 mm per blow

Next 0.6 metres, penetration of 25 - 50 mm per blow

Virtual refusal encountered at 6.5 metres

Even though the above design specified a pile length of 8 metres, due to the hardness of the coral rock they could only be driven to 6.5 metres, where it was assumed that sufficient bond had been developed to satisfy the design. This discrepancy illustrates the uncertainty involved in the design of foundations in coral rock; due mainly to the limited knowledge of the engineering properties involved. It is common for the pile depth to be deleted from the design and left up to the judgement of the engineer on site. Figure 5.7 shows a portion of a plan for the foundations of a 30 metre, free standing tubular tower situated on a sand cay at Frederick Reef. The design specifications state that the pile length is to be determined on site and if the coral prevents driving, piles are to be deleted. If the piles are deleted, the design wind velocity and design wave height are increased in order to increase the factor of safety.

Another common method employed by the Department of Construction is to specify a range for the pile driving depths in the design (e.g. 6 to 8 metres). If the piles cannot be driven to a depth in this range, then extra piles have to be added to the design.

Although driven steel piles are the best foundation types for a coral reef, there are still numerous problems that may be encountered. Cavities or pockets of porous coral and unconsolidated sediments may cause sudden slip of the pile, resulting in loss of end bearing and side friction. In cases where the structure is on the slope between the reef and the normal sea bed, driving may cause localised failure in the coral as illustrated in Figure 5.8. To prevent this, the coral would have to be prebored down to a suitable level and then driving started from there.

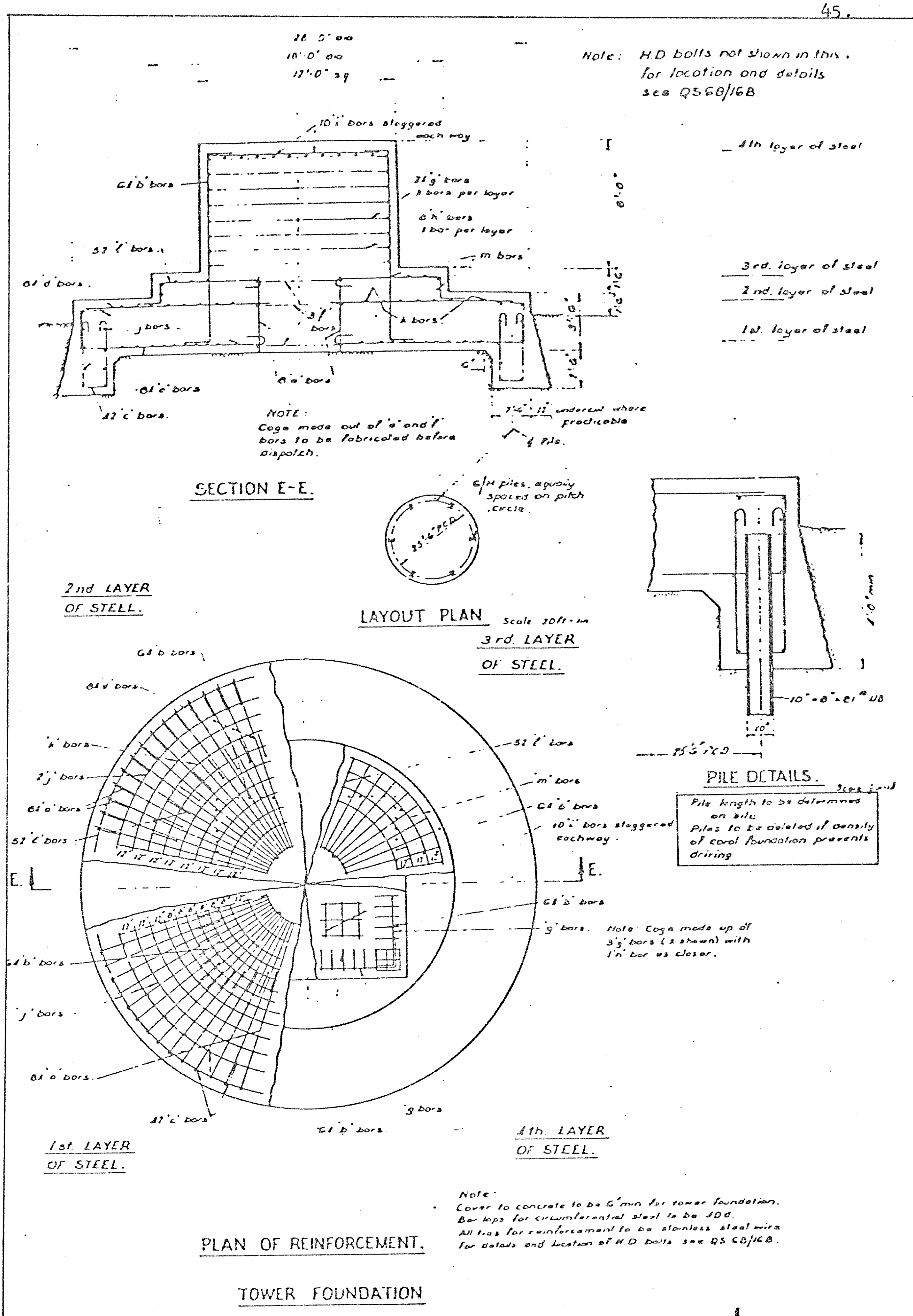


FIGURE 5.7 Foundation Details of Frederick Reef Tower.
(Note Pile Details indicated.)

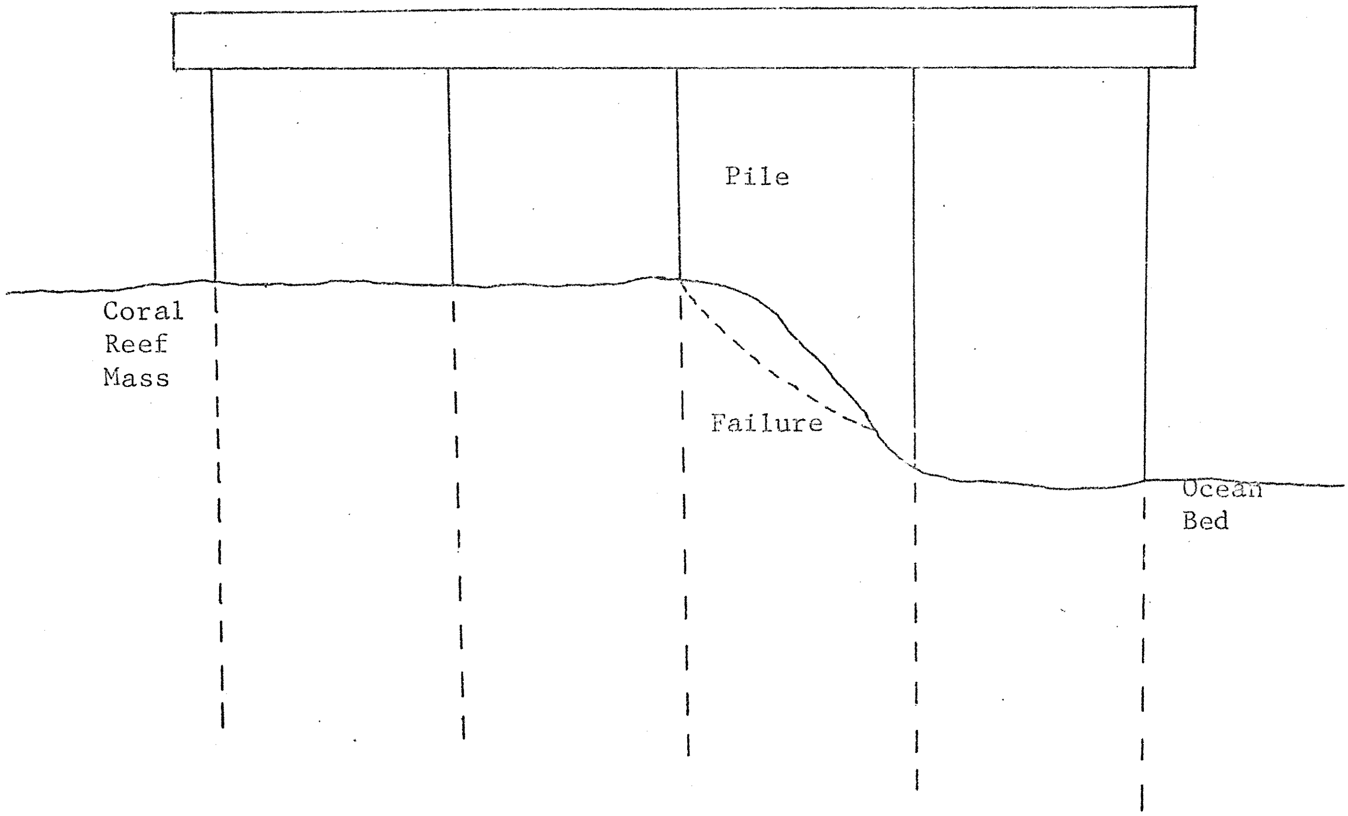


FIGURE 5.8 a. Localised Failure occurs due to Pile Driving

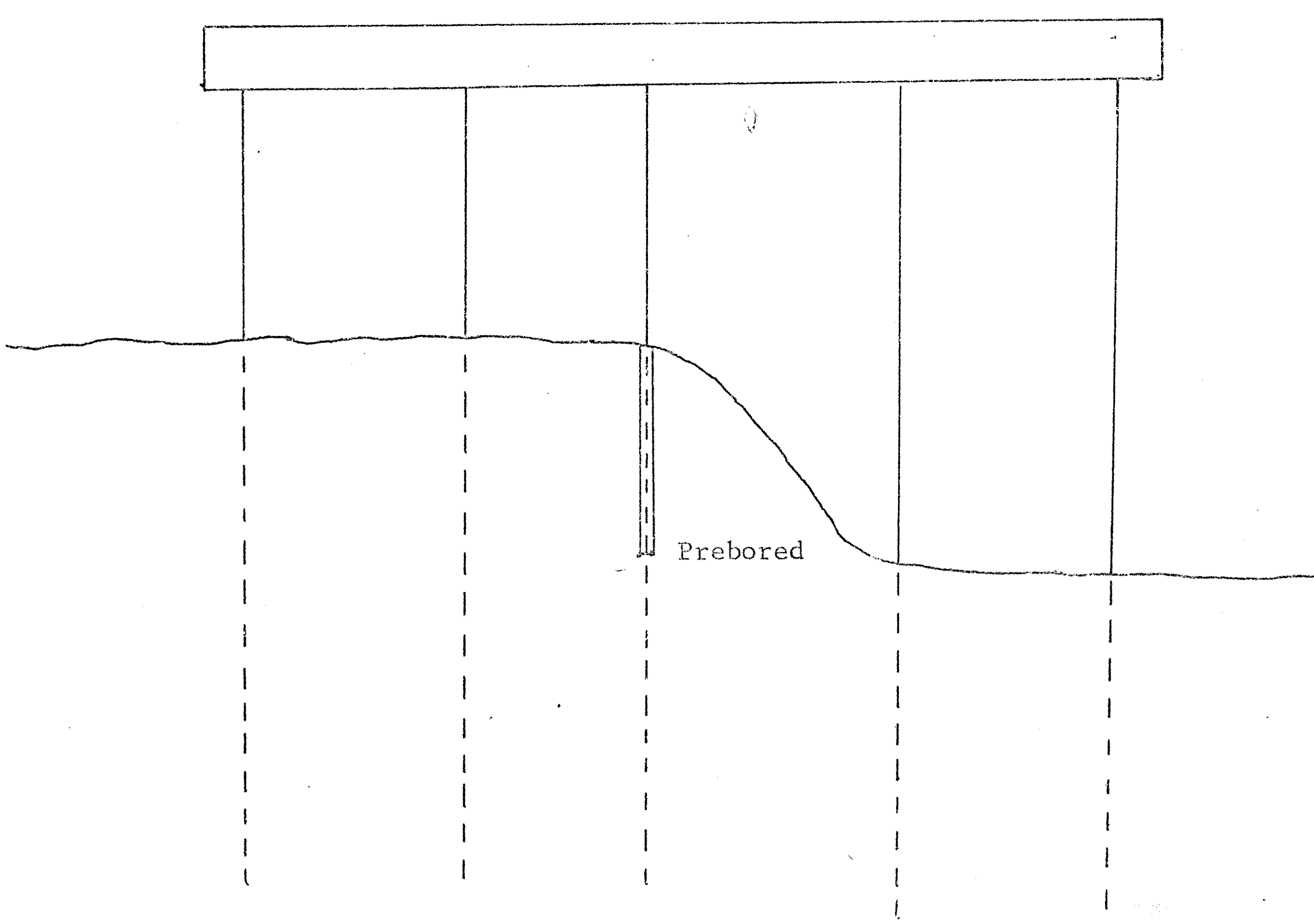


FIGURE 5.8 b. Pile Prebored to prevent Failure.

This situation could often arise where marine platforms are used, since the deep water is utilized for the docking of large vessels.

Concrete is widely used for piles on offshore structures because it resists decay, marine borer and insect attacks, corrosion; and because it can be cast in place or at a nearby location. However, such piles are heavy to handle and require relatively heavy drive hammers. Concrete piles are generally made at or close to the site at which they are used because of their weight. They may be precast before driving or poured in place, although poured-in-placed piles are not well suited to use in exposed marine locations. For heavy marine use they should be precast and prestr ssed to prevent cracking and disintegration.

Cameron (1975) used a form of cast-in-place concrete pile for the foundation of the radar beacon on Keeper Reef. The base of the mast was placed in a 0.5 metre deep cored hole, and then concrete was poured around it. The original design required the hole to be 1.5 metres deep, but due to the extreme hardness of the coral penetration was only to 0.5 metres and because of this it was assumed that 0.5 metres was sufficient.

Cast in place or pressure grouted concrete piles can be totally impractical when there are large cracks, fissures, cavities or pockets of unconsolidated sand, because of the large amounts of concrete that would be wasted in these defects. Generally, where the coral mass is very uniform, any concrete cast into it has good cohesion because of the porous nature of the coral. Probably the main advantage that coral has over other soft unconsolidated soils when used as a foundation is that it is not subject to scour.

5.2.3 Spread Footings

Spread footings include any size or shape of foundation element that develops its supporting power primarily by soil bearing at or near the sea floor. Moveable offshore structures designed for drilling or construction purposes usually employ some form of spread footing because of the equipment simplification afforded by the use of shallow support. Such foundations are not frequently used for permanent construction because they are vulnerable to erosion and have limited resistence to lateral forces.

Mat footings are usually large in area and are planned for sea-floor penetrations that are quite small in comparison to the foundation width. This type of foundation is commonly employed for semi-submersible drilling barges that are used in relatively shallow water; usually less than 10 metres. In contrast to mats, some structures use individual footings designed to penetrate up to one or two times the width of the footing, thereby gaining lateral resistence and, in many cases, support of stronger bearing material. This type of footing is commonly used in connection with the elevating-deck type of moveable structure. This type of structure is in wide use throughout the world for petroleum drilling; some having been designed for operation in water depths up to 95 metres. The utilization and installation of semisubmersible and self-elevating structures is discussed further in Chapter 7.

Cameron (1975) proposed a caisson-type foundation in the preliminary design of a platform on Broadhurst Reef. This form of foundation is basically a mat footing and was chosen in this case because the proposed site was in the lagoon area of the reef where the water is shallow and the bottom is flat and sandy.

These are basic requirements if this type of foundation is to be used. Another factor influencing the choice of a mat footing was the fact that the proposed structure was a relatively small platform with a design life of only five years, meaning that the loads are greatly reduced. A mat footing would be impractical for a very large platform because of the massive dimensions, and quantities of concrete that would be required.

Footings have been used for the foundations of lighthouses and weather stations on sand cays, but with very limited success. It was found that even though the cay was permanent and stable at the time of construction, this is not the case in the long run. Severe weather conditions such as cyclones, and changes in the regional hydrology, lead to drastic alterations in the structure and shape of the cay. This can lead to severe erosion under the footing, thus endangering the stability of the structure.

CHAPTER 6

DESIGN OF THE SUPERSTRUCTURE

6.1 Introduction

The superstructure of an offshore structure is that part of it above the foundation level, and, to simplify the discussion, I will refer to this as the structural part or simply the structure.

The fact that lighthouses constitute 99 per cent of the structures on The Great Barrier Reef makes it very difficult to ascertain what sort of structural design is suitable for reefs, based on past experience. Where the foundations are concerned, it is the special problems encountered with the coral that is important, regardless of what the structure is. So, therefore, one can assume that the foundations of lighthouses can be readily applied to any structure, regardless of its form.

In the case of lighthouses, the structure is very stereotyped; that is, there is a standard structural design that can be applied to any situation. This standard design is basically a tower and it is useful for lighthouses, and nothing else. But, it can be assumed that the structural design principles employed for offshore situations are universal. That is, the materials and structural form may differ, but analysis of the loads will be the same; with their magnitudes depending on the particular environment.

6.2 Materials

A typical coral reef situation is an extremely corrosive environment and this, along with cost, are the major factors controlling choice of a structural material. In this type of situation maintenance costs can be very high, so therefore the structures should be designed so that they require the absolute minimum of maintenance throughout a long life. Reduction of maintenance can be achieved by using corrosive-resistant materials such as aluminium and stainless steel, or by using protective coatings that can either be chemical or metallic. The final decision on what to use will be based on the long term cost and reliability of the respective methods. The actual design or form of the structure can also be used to reduce maintenance. For example, a free standing tubular tower has fewer joints and discontinuities (corners) than a lattice tower, so therefore it requires less maintenance.

The Commonwealth Department of Construction conducted a survey in 1971 into the suitability of four different types of materials for use on lighthouse structures in harsh marine environments; in particular, coral reef situations. The materials tested were: general purpose structural grade mild steel manufactured to Australian Standard Specification A149-1965; high strength low alloy structural steel manufactured to ASTM Standard Specification A242-66 (Austen 50); stainless steel, American Iron and Steel Institute Type 31b; and aluminium of the Aluminium Development Council Specifications 5000 and 6000 series. Each of these materials was examined to determine its suitability with regard to corrosion resistance and strength.

Mild steel, galvanised and/or painted, is the most widely used material on lighthouses to date. Its principal advantage is its low initial cost, while its disadvantages are that it readily corrodes in a marine atmosphere and it requires regular maintenance. An example of the rate of corrosion is at Miles Reef where galvanised mild steel angles positioned 3 metrės above high water on an exposed site were reduced by 3.2 mm in nine years.

High strength low alloy structural steel (Austen 50) shows a tendency to corrode under conditions of wetting and drying, developing a dense oxide layer which becomes denser and more adherent with age and tends to inhibit further corrosion. Austen 50 is not recommended for use unpainted in a marine environment, and laboratory tests show that the paint life on low alloy steels in a marine atmosphere is at least doubled when compared with the paint life on mild steel at the same site.

Stainless steel of AISI Type 361 has been used extensively for lighthouse structures off the Queensland coast, and has proved most successful in service. Stainless steel, due to its oxide coating, is a passive metal which allows it to be exposed to severe corrosion environments without deterioration of the stainless steel surface. However, stainless steel is subject to certain types of corrosion that can cause failure of structures and that must be considered and checked for at regular intervals. These are: intergranular corrosion, stress corrosion, pitting corrosion, service corrosion and galvanic corrosion.

Aluminium has been widely used with considerable success for boat hulls and ship superstructures for nearly fifty years, but the use of aluminium on lighthouses has been very limited. Temporary aluminium tubular scaffolding structures have been erected as light and beacon supports at several points along the Australian coast, and aluminium has also been used for radar reflectors on beacon structures. Due to fatigue behaviour of aluminium whereby it does not have an endurance limit in marine environments, structures of aluminium must not have members subjected to fluctuating stress conditions.

Protective coating can be a very efficient method for reducing corrosion in structural materials, but its efficiency depends on regular maintenance to keep the coating in good condition. Mild steel and low alloy copper steel must be coated with a protective paint to prevent corrosion in marine environments and must receive regular maintenance depending on the type of paint system and the base metal. With aluminium and stainless steel, the prime advantage is that the oxide film for both metals is protective to the extent that painting is not required and hence maintenance is reduced to a minimum. If aluminium and stainless steel are painted, regular maintenance is required as the paint surface will hold water and assist pitting if not maintained in good condition.

In considering any paint system for a marine environment, it is essential that the initial coating be shop applied to sandblasted white metal surface. The requirements of a paint system are that:

(i) It is resistant to abrasion and impact so that sections can be transported and handled with the minimum of damage to the paint surface.

(ii) Repairs and repainting can be readily made to the paint system with the minimum of preparation and equipment.

(iii) The finishing or envelope coats must be resistant to marine and ultra-violet attack for extended periods up to 10 years.

(iv) The priming or sacrificial coats should be anodic to the base metal and will offer protection if the finishing coats fail at isolated points.

Many different paint systems are available that generally satisfy the above requirements. These include vinyls, urethanes, tar epoxys and chlorinated rubber with zinc base primers or with aluminium spray. The tar epoxys give a hard surface, which may be brittle and they are generally difficult to overcoat, without sandblasting beforehand. The other top coats are easier to overcoat.

Figure 6.1 shows the estimated structural maintenance time required for lighthouse towers constructed of the four different materials discussed above. These figures were calculated for a program that involved the construction of 70 towers off the Australian coast, and they indicate the extra maintenance time per year required (i.e. on top of maintenance for existing towers). Approximately one-third of the towers involved in this program are in coral reef environments, or similar situations off the Queensland coast. This diagram illustrates the tremendous saving in maintenance afforded by using stainless steel instead of mild steel or austen.

The values obtained in Figure 6.1 were based on previous maintenance programs for towers of the type shown, and since aluminium is very much an unknown quantity as far as use in lighthouses is concerned, the values obtained for it should be treated with caution. However, stainless steel, mild steel and austel have been used extensively for lighthouses, so therefore a comparison of these three can accurately be obtained from Figure 6.1.

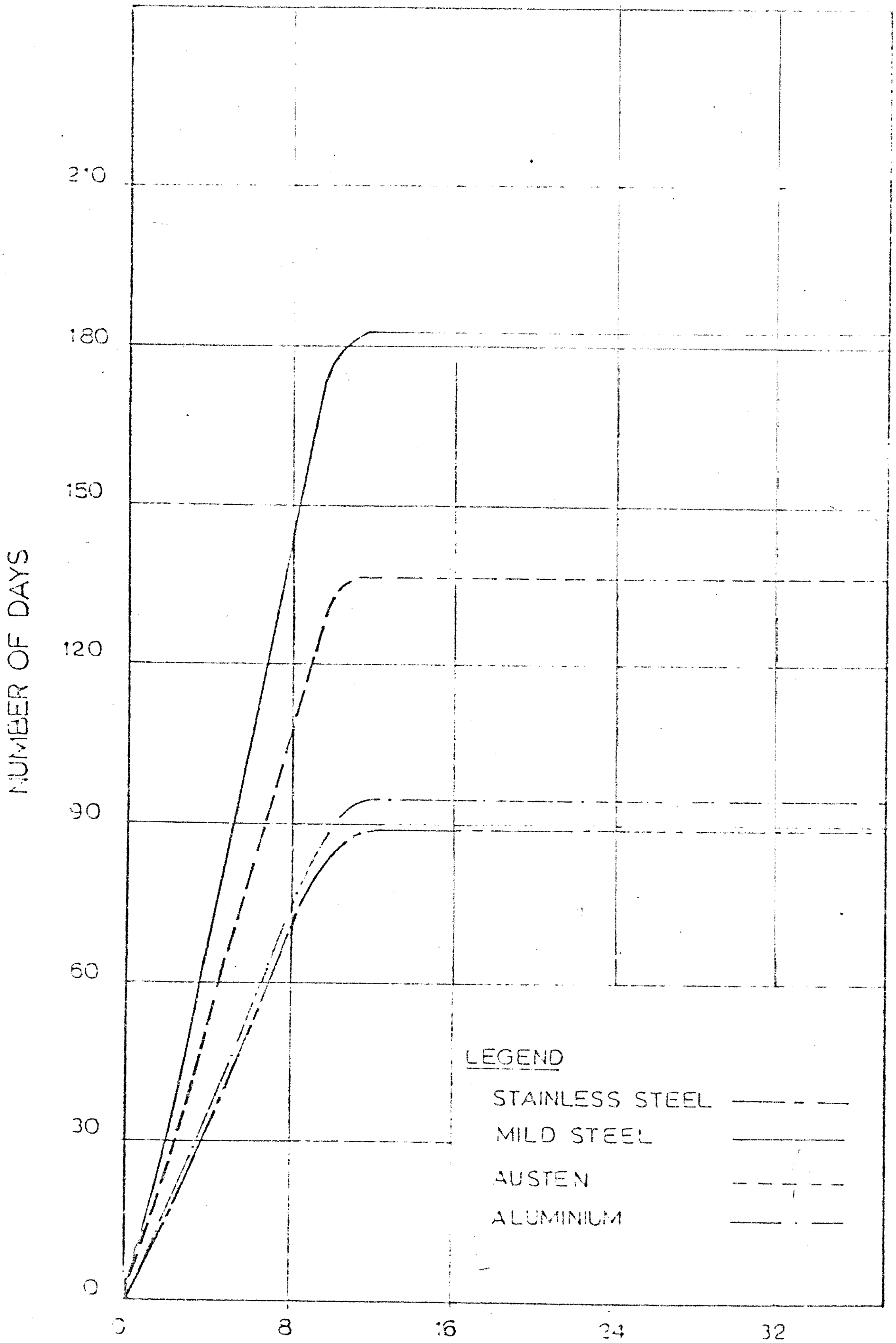


FIGURE 6.1

ESTIMATE OF ALL MAINTENANCE REQUIRED IN DAYS FOR 70 TOWERS BUILT OVER A TEN YEAR PERIOD.

Figure 6.2 shows the variation of the estimated total expenditure on one tower versus age of the tower. Obviously, the initial cost at zero years is the construction cost of the tower. The total expenditure is based on actual cost; that is, the sum of money which invested now will finance initial costs and all other commitments including structural and routine maintenance. From the graph it can be seen that a stainless steel tower is the most expensive to build, while mild steel is the least; but after just nine years the total expenditure on the mild steel tower is greater than that on the stainless steel one. And, after 25 years, the stainless steel tower will prove to be cheaper than any of the others. It must be noted that the actual monetary values shown are not accurate since the figures were formulated in 1970, but the behaviour the lines exhibit could be assumed accurate. The only major changes possibly being increases or decreases in the initial costs due to price fluctuations of materials.

Of course, corrosion isn't the only criteria by which structural materials for offshore structures are chosen, but it is often the deciding factor since other factors can be controlled somewhat by changes in the design. The characteristics of a material that are important for offshore use can be divided into three general categories: Design, Fabrication, and Service. The individual characteristics of each category are:

Design	Fabrication	Service
Yield strength	Joining	Corrosion resistance
Ultimate strength	Forming	Stress - corrosion resistance
Young's modulus	Machining	Corrosion - fatigue resistance

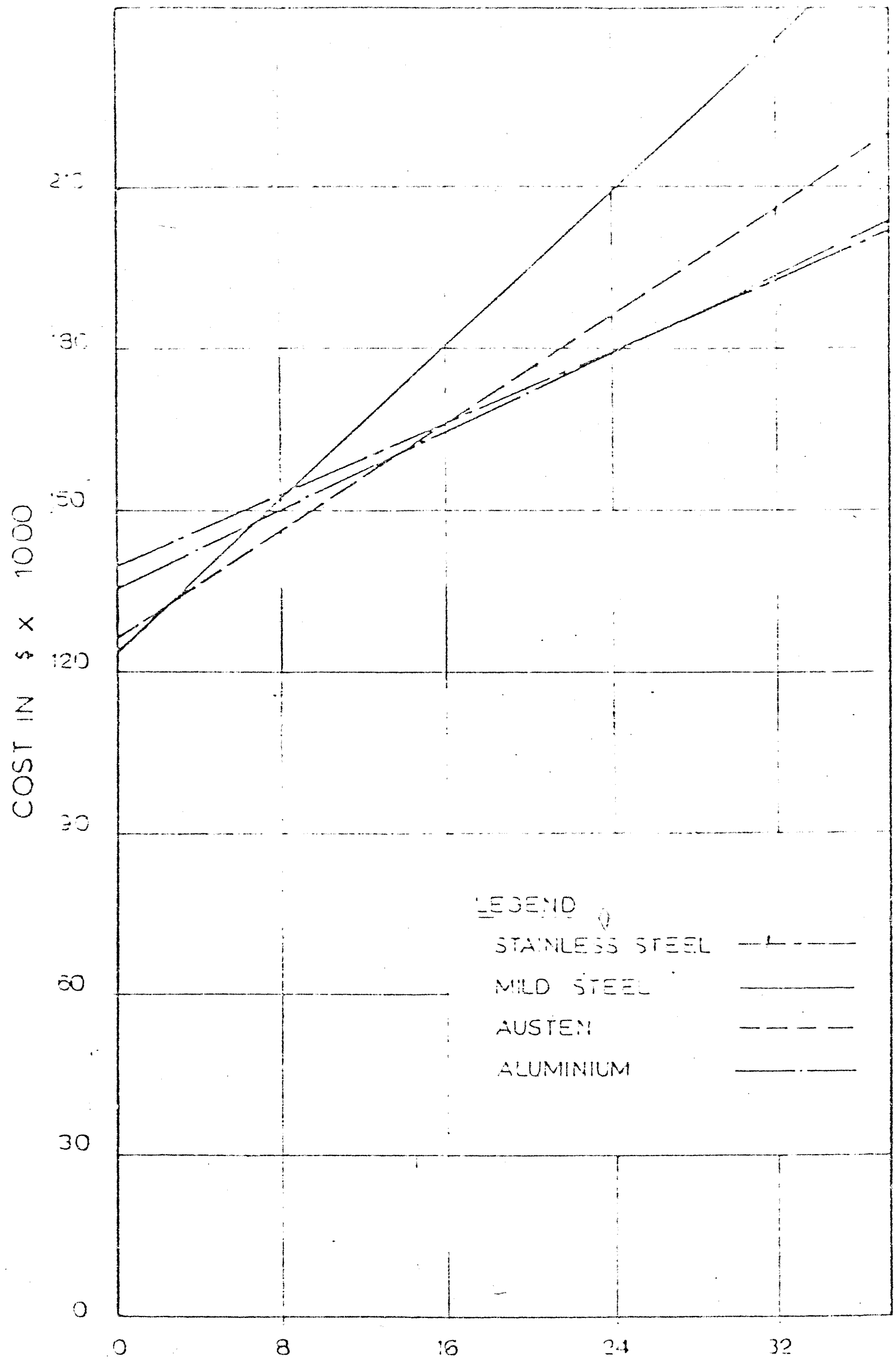


FIGURE 6.2

ESTIMATED TOTAL EXPENDITURE ON ONE TOWER BASED ON ACTUAL COSTS.

Design	Fabrication	Service
Density	Quality control	Repair
Poisson's ratio		Maintenance
Fatigue resistance		Creep resistance
Brittle-fracture resistance		

The importance of the design and service characteristics is obvious, but fabrication is equally important for offshore structures since the materials have to be fabricated and machined before they are transported to the actual site. So therefore it is imperative that the materials form into the required components precisely, since no major machining or fabricating is possible on site.

6.3 Structural Loads

6.3.1 Introduction

The structural design loads of an offshore structure are a function of predicted sea conditions, duration of operations and the acceptable risk for exceeding the predicted load values. This risk will be governed by the actual states of seas encountered during the useful life of a platform and the increased structural cost the operator is willing to accept. In general, design of offshore structures involves probability aspects that are much more important than in the design of onshore structures.

Important considerations in the design of offshore structures include: selection of a structural design life; computations of maximum expected wind-speed, storm-tide, and wave characteristics; and selection of design allowance for any long-term detrimental effects on the structure from the elements (e.g. corrosion). The relative importance of each of these factors for a single application is dependant upon factors such as the particular meteorology and submarine topography of the site.

As has been stated previously: the analysis of the structural member stresses is the same in a coral reef environment as any other offshore situations, and is basically the same as a similar terrestial structure. It is the actual determination of the design loads for the particular environment that is specialized. For this reason, the discussion in this chapter will be strictly limited to the determination of the design loads for a coral reef environment and will not consider the structural analysis.

6.3.2 Wind Loads

Owing to the statistical nature of severe storm or cyclone occurrences, the most probable adverse conditions that a structure will encounter depend on the length of time during which the structure is required to perform its planned function in the environment; that is, its design life. The Commonwealth Department of Construction recently conducted a study into the design of a standard lighthouse tower for remote sites on the Australian coast, particularly those in coral reef environments off Queensland. The design life was chosen so as to comply with the economic and maintenance analysis conducted in the study. A value of thirty years was decided upon, but it should be noted that this does not mean that the towers will cease to be useful after that period. The basic procedure used by the Department of Construction for determining the design wind speed is described below.

The selection of a single design wind speed for standard lighthouse towers would result in all towers being capable of withstanding the most intense wind likely, within the design period, for any point on the Australian coast. Due to the range of wind velocities that can be expected on the Australian coast, such a design philosophy would result in conservative designs for a large proportion of towers.

An analysis of annual maximum wind gusts at various locations on the Australian coast was made using the Gumbel's standard skewed distribution as described in "Extreme Wind Gusts in Australia" by H. E. Wittingham. The design wind for a thirty year period was determined on a constant risk basis, so that there is a 20 per cent chance that the factored design wind will be exceeded one or more times within a thirty year period. For cyclonic areas, this design wind is slightly greater than the return period wind which could be equalled or exceeded on the average, once in thirty years. Together with this analysis, the detailed history of major cyclones between 1957 and 1967 within a number of areas on the Australian coast was examined and an estimation of the likely winds to be expected within the design period was made.

The analysis showed that for a thirty year period a design wind of 240 kilometres per hour is suitable for all areas. This value was used for the Australian coast north of latitude 27 degrees south, while south of this in areas not subject to cyclones a design wind of 176 kilometres per hour was decided on. This can be compared favourably with the design wind speeds calculated according to the Australian Standard Wind Loading Code (AS1170, Part 2 - 1975), as shown below:

Structure in cyclone region

Cyclone factor (multiplier) = 1.15

Structure in terrain category 1

Assume maximum height of tower = 30 metres (100 feet)

Velocity multiplier = 1.16 - From Table 4

Regional basic wind velocity for a 30 year return period

= 51 m/s - From Table 2

Design wind velocity = 1.15 x 1.16 x 51 m/s

= 68.03 m/s

= 245 kilometres/hour

(Compared with design wind speed of 240 km/hour used by Department of Construction.)

Up until recently, all lighthouse structures in Great Barrier Reef waters have been standard steel lattice towers of varying heights. Because of this, the analysis of the member stresses due to wind loading is a relatively simple procedure, with computer programs being used in more recent years. The trend towards the use of free standing tubular towers in some situations has lead to special problems being encountered when considering the wind loading. This is illustrated in the design of the Frederick Reef tower.

The Frederick Reef tower is a 30.48 metre high, free standing tubular tower constructed of stainless steel and situated on a sand cay, east of Mackay, on the outer edge of the reef (see Figure 3.1). The basic shape and dimensions of the tower are shown in Figure 6.3. Experience with similar land-based structures, and wind tunnel investigations indicated that significant stresses could be induced in the tower by vortex shedding. This vortex shedding gives rise to a lateral thrust on the tower and, an oscillatory motion normal to the free stream velocity will develop, especially if the frequency of vortex formation is close to the natural frequency of vibration of the tower. It should be noted that the line of maximum induced stress is perpendicular to the wind direction. The natural periods of the tower were ascertained as:

1st mode - 0.38 seconds

2nd mode - 0.096 seconds

3rd mode - 0.041 seconds

The corresponding critical wind velocities for vortex shedding are:

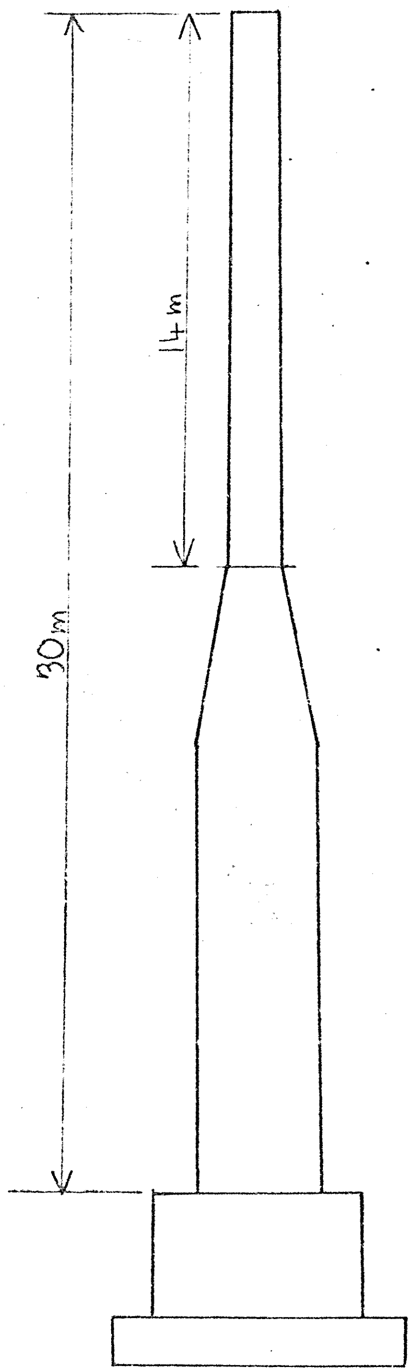
1st mode - 37.7 km/hr.

2nd mode - 149.4 km/hr.

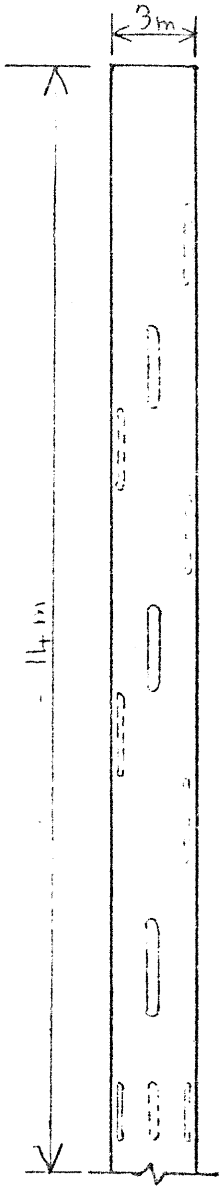
3rd mode - 349 km/hr.

The likelihood of the 3rd mode ever being reached is remote since the design wind speed of the tower is 240 km/hr.

Conservative calculations based on the above values concluded that vortex-induced flexural stresses of approximately 86 MPa may occur. In order to reduce these stresses, it was decided to put in a series of vertical slots as shown in Figure 6.3.



General Shape and Dimensions of Frederick Reef Lighthouse



Slots are 1.22 m long and 152 mm wide

Enlarged View of Top Section of Tower showing arrangement of Vertical Slots used to reduce effect of Vortex Shedding.

FIGURE 6.3

These slots, which are 152.4 mm wide by 1.22 metres long, were found, primarily by wind tunnel testing, to significantly reduce the vortex-induced stresses.

It was stated previously, that among other things, the structural design loads of an offshore structure are a function of the acceptable risk for exceeding the predicted load values. This "acceptable risk" approach to design is especially important in the case of projects that are a commercial venture; that is, the project has to be a viable economic proposition. This concept is a field of research within itself and involves a variety of probabilistic, semi-probabilistic, deterministic, and stochastic analysis methods that are far too complex to be considered in the context of this discussion. Appendix A gives a number of references that could be helpful in pursuing this aspect further.

6.3.3 Wave Loads

Just as the design of a structure to withstand wind loading is based on a design wind speed, the wave loading design is based on a design wave height. As well as being used to determine the wave-induced member stresses, the design wave height determines the elevation of the lowest deck of a platform-type structure, since it is desirable to keep this deck above water at all times. The acceptable risk of exceeding the predicted values that the operator is willing to undertake is once again a major factor in the choice of a design value.

The maximum wave height, or crest elevation, is comprised of three components:

(1) The astronomical or tidal component,

(2) The barometric component, and

(3) The wind-stress component.

The tidal component is simply the change in water level due to the attractive forces of the moon and sun. It is probably the easiest to determine since the tidal range for most points on the Queensland coast is known, thus allowing the range for points just off the coast to be predicted with reasonable accuracy. Alternatively, tidal gauges can be installed at the proposed site in order to obtain accurate measurements.

The barometric and wind-stress components are more commonly referred to as the storm surge; that is, the increase in height that occurs during severe storms or cyclones. Obviously, this is the time when the maximum crest elevation will occur. The barometric component is the general rise in water level in a given area, due to the pressure drop associated with a severe storm. In Great Barrier Reef Waters, a rise in water level of 0.3 metres for every drop of 30 millibars in pressure could be expected.

The wind-stress component is a result of the onshore wind stresses acting on the water surface and is in effect a piling up of a water mass against the coastline. In a reef situation, this component is generally not significant, due to the breakwater effect that the reef has. That is, the distance of exposed water between the structure and the reef (the fetch length) is not sufficient to generate waves of significant size.

According to Myers (1969), there are three basic steps that should be undertaken when determining the actual wave forces acting on a structure. These are:

(1) selection of a design wave amplitude and period,

(2) selection of the appropriate wave theory to compute velocities and accelerations within the wave, and

(3) selection of drag and mass coefficients and a suitable theory for wave-force calculations.

It should be noted that in each of the above steps a selection process is required. That is, from the many alternatives available, the designer must choose the one which he regards as the best for the particular situation.

Cameron (1975) used the Morrison formula to calculate wave forces acting on a marine platform in Broadhurst Reef. The formula states:

$$F = \tfrac{1}{2}\, p\, g\, D\, a^2 \cos^2 \Theta \left(1 + \frac{a \cos \Theta}{d}\right)$$

$$- \, p\, g\, \frac{PID^2}{4}\, a \sin \quad K\,(d + a \cos \ \Theta)$$

where F is the force on the pile;

p, g are the density and gravitational constants respectively;

a is half the wave height;

Θ is the phase angle;

d is the depth of water;

D is the pile diameter; and

K is the wave no. = $\frac{2\ PI}{L_o}$ where L_o is the deep water wavelength of the wave

Using a maximum crest elevation of approximately 4.5 metres, parameter values for different wave periods were substituted into the formula and the worst case was evaluated. A fifteen second wave gave the largest force on the section. The sensitivity of this force to large variations of the wavelength parameter, L_o, which is difficult to evaluate accurately, was tested and found to be acceptable.

In the case of structures on bare coral reefs that are covered by water all or some of the time, the wave forces are relatively small because of the shallow depths involved and the energy-dissipation effect that the reef has. Lighthouses in such a situation are generally designed so that the wave forces present are not critical. The design usually consists of the actual tower sitting on a concrete slab that is connected to the base slab by massive concrete columns. The columns are the part of the structure actually subjected to direct wave forces. In the design of the Pith Reef and Rib Reef towers, the Department of Construction used a design wave of 3.6 metres and a period of 7.4 seconds.

6.3.4 Impact Loads

Impact loads arise in situations where the offshore structure provides mooring facilities for boats. The loads are usually absorbed by some sort of guard system which ensures that excessive loads are not placed on the actual structure. Fenders are the most common form of protection utilized.

Fenders are protective devices located and arranged in such a way that they will absorb a calculated amount of kinetic energy when struck by a moving ship while bringing it to rest. Some energy, if kept below harmful limits, will be absorbed by the deformation of the ship's hull. Piles are used extensively as vertical fenders set out in front of a marine structure. Figure 6.4 shows prestressed cantilever-pile guard fenders.

In most cases, cantilever piles alone absorb too little energy and require too much deflection to develop the required pile strength in bending. Fender piles are generally supported laterally at the deck of the structure to reduce the deflection. However, the piles will absorb too little energy and will probably arrest the approach of the ship with too little movement, thus increasing the reaction. So, therefore, fender piles are often separated from the structure by some sort of energy-absorbing device such as hard rubber pads. The performance and load capacity of the various types of energy-absorbing devices and fender systems available are usually given in the manufacturer's specifications.

Fender piles can be used in various arrangements as guards to prevent ships from accidently running into a marine structure. The type of construction will depend upon water depth, size and speed of the vessel, current, soil characteristics, and economic factors based on expected frequency and degree of damage to the vessels and fender system./ Fender piles would be required in the case of a platform situated on the leeward edge of a coral reef, such that one side is in deep water, to allow for ships to dock alongside.

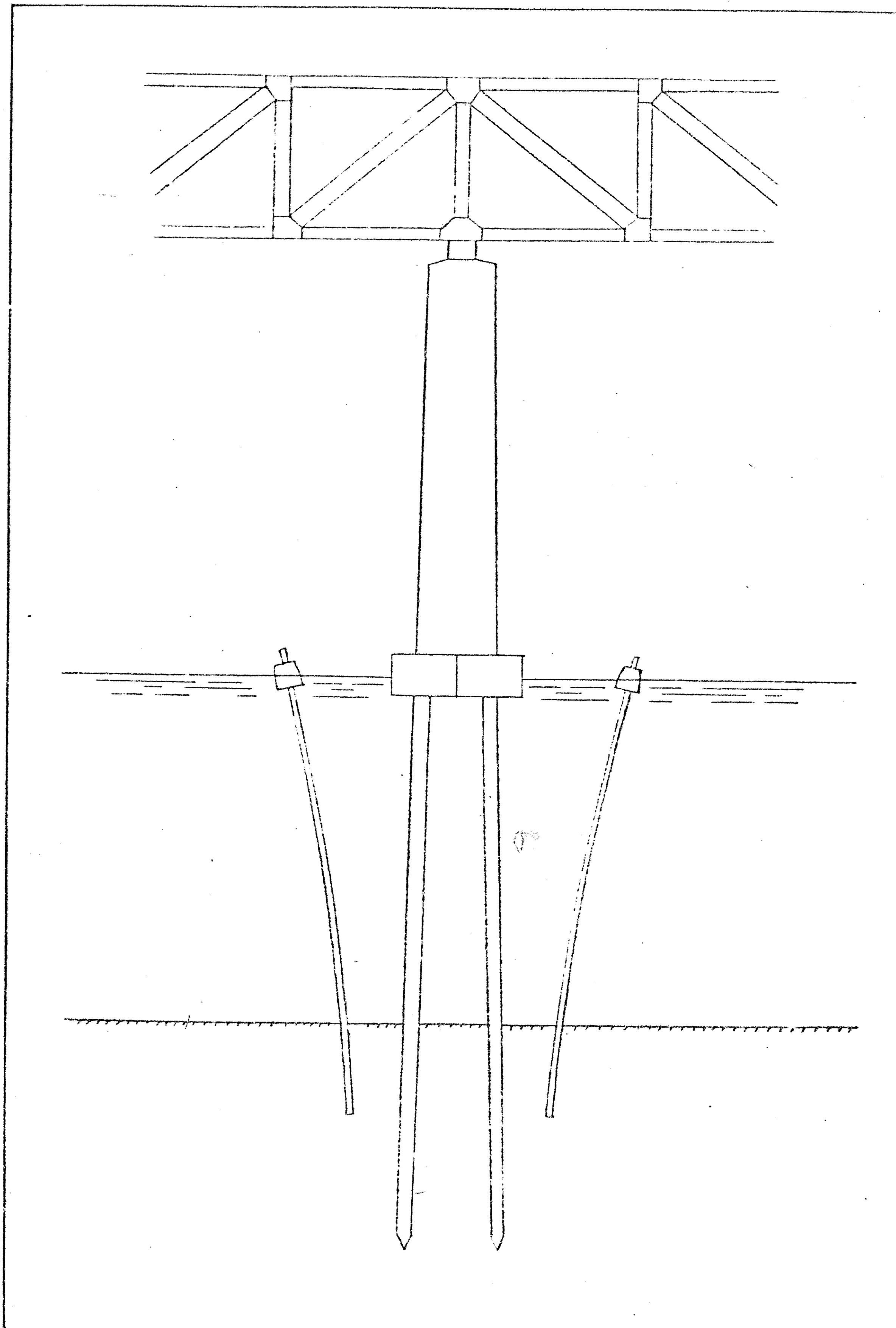


FIGURE 6.4 Prestressed Cantilever Pile Fenders

CHAPTER 7

CONSTRUCTION ON A CORAL REEF

7.1 Introduction

When designing a structure in a coral reef environment, it is essential to take into account the major construction difficulties that will inevitably arise. These difficulties can be kept to a minimum by designing the structure such that the construction is kept as simple as possible, and by efficient planning of the overall operation.

Construction on a coral reef, whether it be on the bare coral itself or in the deeper water areas of the lagoon and the leeward edge, generally involves the use of a working platform from which all operations are carried out. The types of platforms that have been used as well as those that could be used in coral reef situations are discussed in Section 7.2. It should be noted that as well as being used as working platforms, the platforms discussed could be utilized as permanent or semi-permanent structures in their own right. When the proposed structure is situated on a sand cay or a small island, construction is obviously much simpler because of the fact that you have a firm, dry base from which to work. In situations like this, conventional land-based construction techniques can be used to a major extent.

The foundations of a shallow-water offshore structure, particularly those on coral reefs, usually involves the use of considerable quantities of concrete. In situations where the site is submerged, numerous problems arise in the mixing and placing of the concrete, and its strength and durability can be adversely affected. These problems and ways of overcoming them are discussed in Section 7.3.

7.2 Types of Working Platforms

It was stated in Section 5.2 that the most common type of marine platform currently in use consists of a crossed-braced hollow steel pile foundation. The main advantages of this type of platform is that they can be used in virtually any depth of water and they are easy to install. Also, they can vary in size and small scale versions can be utilized as temporary working platforms. Figure 7.1 shows a typical crossed-braced template type structure used for oil drilling in deep water. The installation procedure for this type of structure is illustrated in Figure 7.2, and described below.

The procedure begins with the placement on the sea floor of a prefabricated substructure or template, that is usually constructed at a shore facility and transported in its entirety to the site. The legs of the template are open tubular columns through which bearing piles are driven. The pile driving rig would be on a large floating barge or raft that is securely anchored in position next to the template. Once the pile driving operation is completed, the deck structure can be easily erected since the top of the template is sticking out of the water, thus providing a stable base from which to work.

The template-type structure discussed above is essentially a fixed platform. In some cases, a movable working platform is a much more practical proposition since, as well as being used for the construction, it can be used for the initial site investigation to determine the most favourable position of the proposed structure.

The self-elevating barge is a mobile-type platform that has been used for many years for petroleum explorations in water depths of up to 100 metres. It consists of a steel barge with wells at the corners and along the sides, through which steel caissons or legs are placed. The barge is towed to the site of the proposed structure with the legs fully raised so as to reduce drag. When the required location has been reached, the legs are released to rest on the bottom and jacking devices over each well location are used to drive the legs into the soil until the force exerted is sufficient to raise the barge out of the water to a height above the maximum wave height. The jacks have to be then operated continuously to hold the barge on the legs. Figure 7.3 illustrates the basic procedure behind the installation of a self-elevating platform.

A submersible or bouyant type platform is a movable platform that basically consists of a column-supported deck resting on a bouyant frame. The platform is towed out to the specified location and the frame is then sunk so that it rests on the sea bottom and acts as a mat footing (see Figure 7.4). This type of structure is usually restricted to shallow water situations where there is a reasonably flat bottom. In deep water, the frame can be adjusted so that it floats at an intermediate elevation, with the structure held in place by an anchoring system.

All of the above platform types are suitable for use as working platforms when the proposed structure is very large, thus requiring heavy equipment for both the construction and the site investigation.

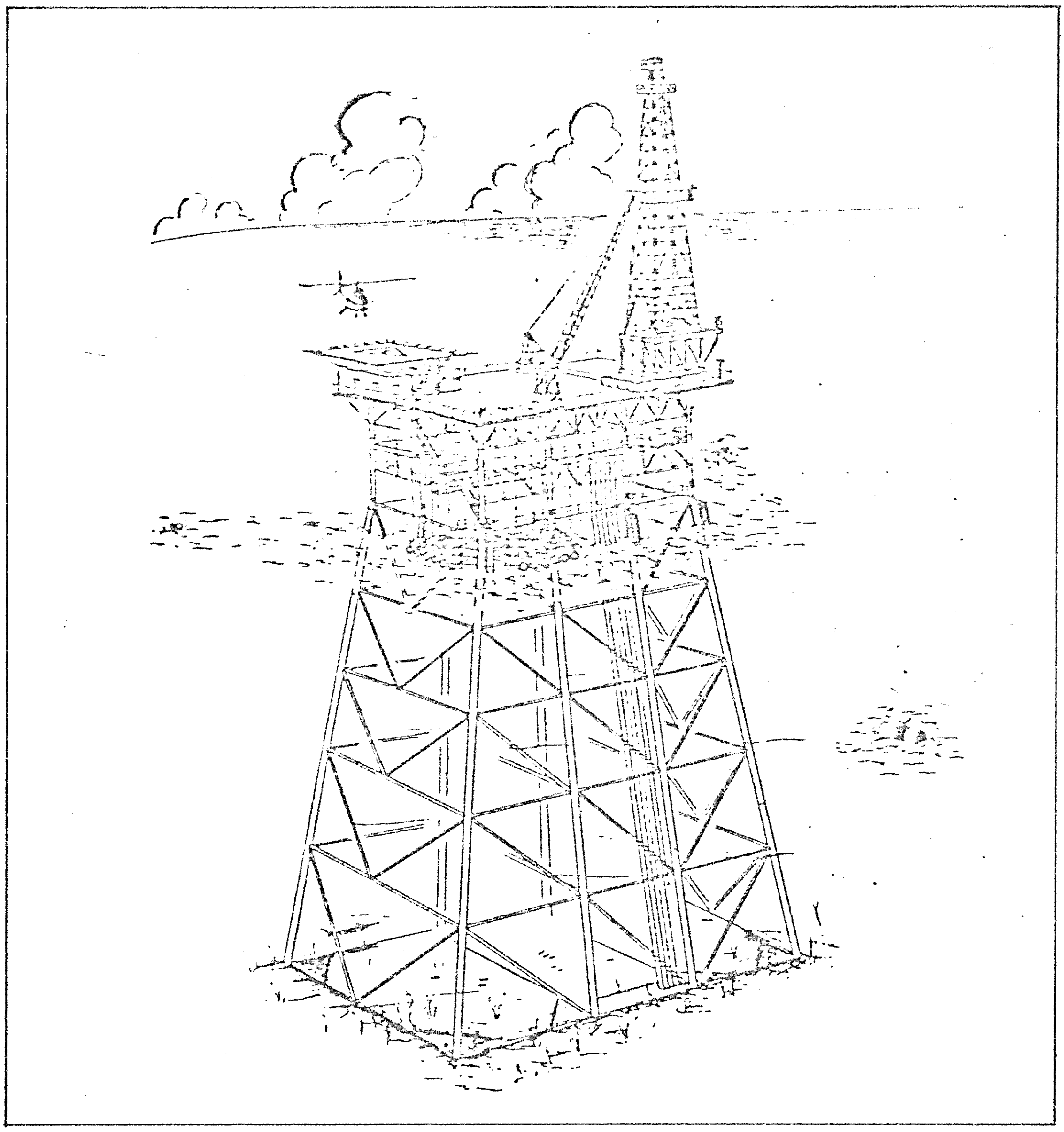


FIGURE 7.1 Typical Template-type Structure used for Oil Drilling (after Civil Engineering in the Oceans, Ref. 7).

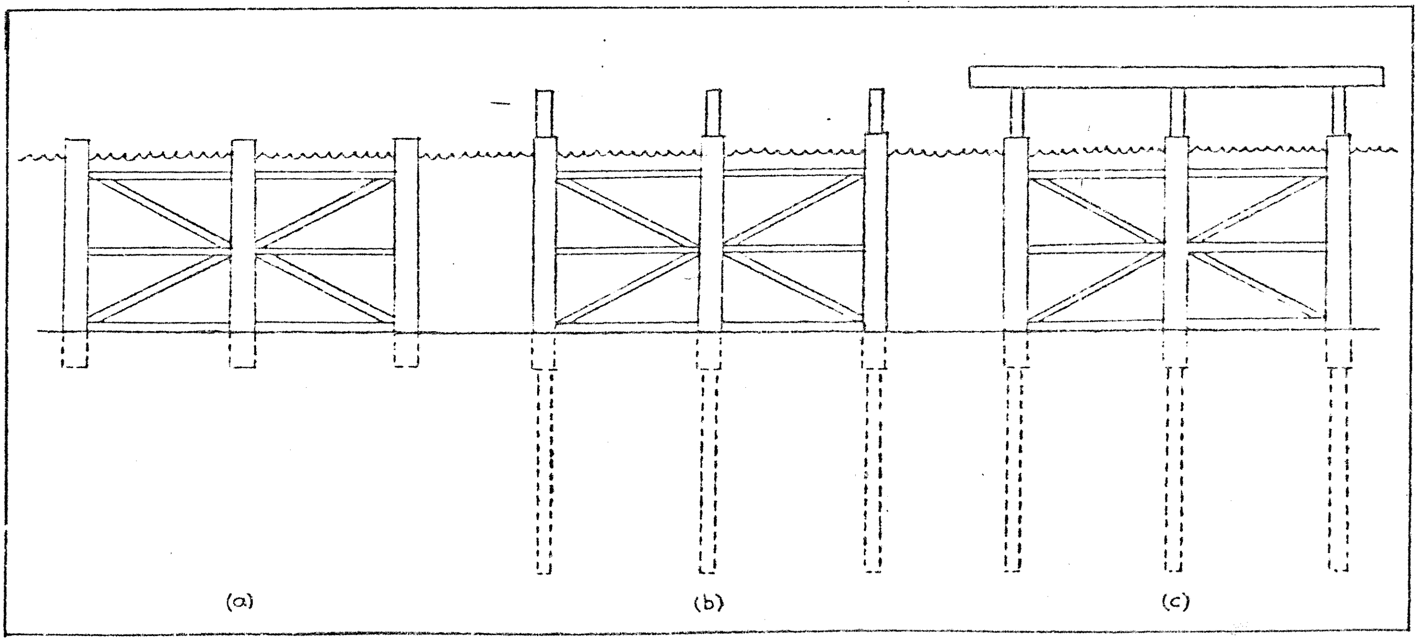


FIGURE 7.2 Installation of Template-type Structure:
(a) Placement of Prefabricated Substructure,
(b) Bearing Piles in Place, (c) Deck Structure Erected.

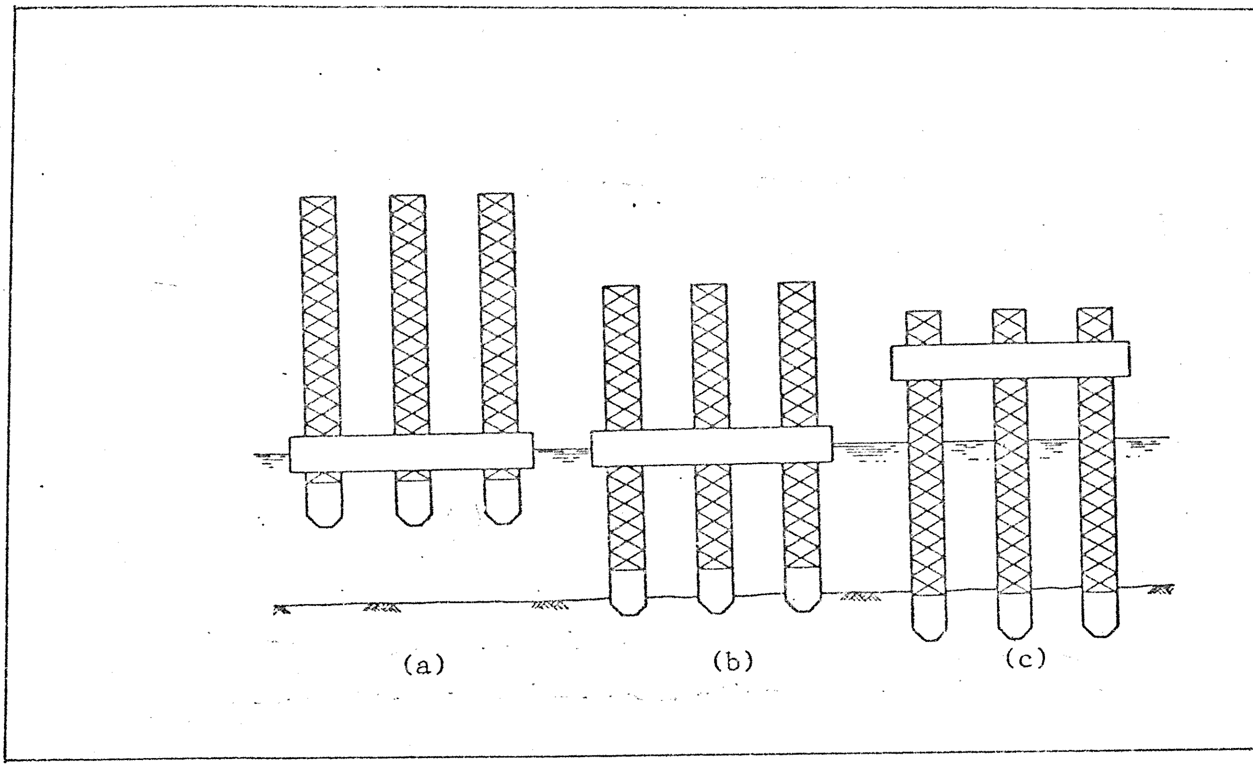


FIGURE 7.3 Installation of Self-elevating Type Platform

(a) Positioned over Site

(b) Legs Released to Rest on Bottom

(c) Deck Jacked up

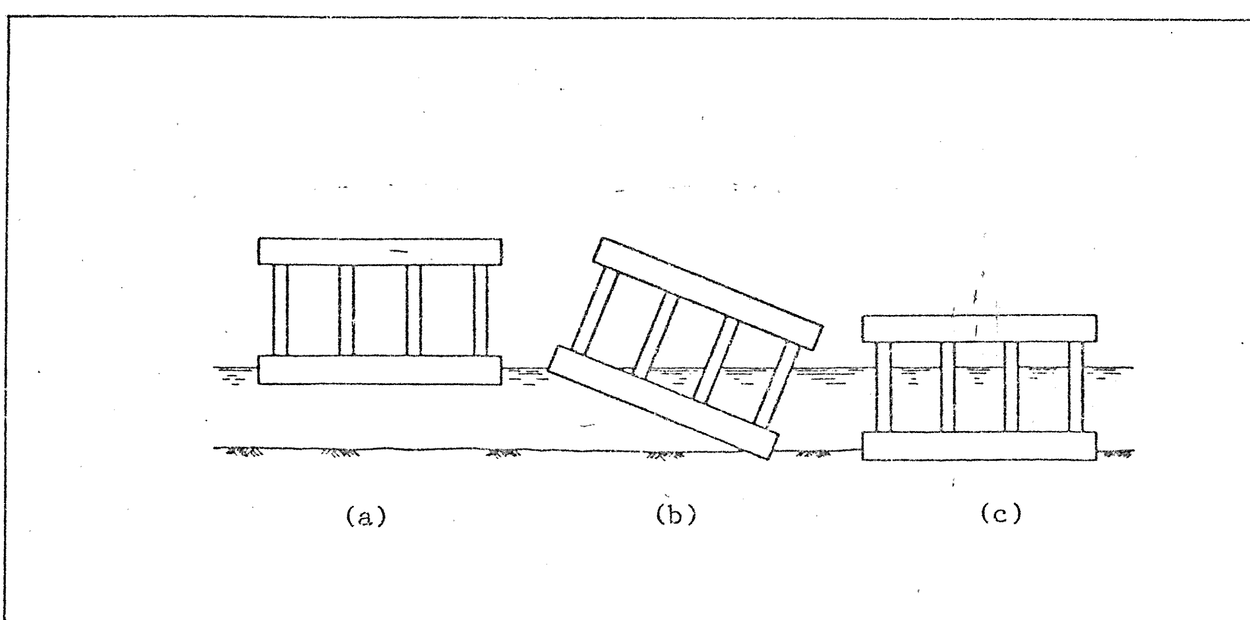


FIGURE 7.4 Installation of Submersible or Bouyant Type Platform

(a) Positioned over Site

(b) Bouyant Frame Submerged

(c) Resting on Bottom.

In a coral reef situation they can be utilized when the proposed structure is in the deep water areas of the lagoon and the leeward edge.

In cases where the proposed structure is relatively small and the water shallow, construction can be carried out from simple, temporary working platforms that are built over the site. An example of this is the Pith Reef and Rib Reef lighthouses, where the site was a bare coral reef always submerged to at least 1 metre. The platform consisted of a standard steel pipe scaffold tube and fittings, similar to that used for formwork. With a labour force of 9 men, the total working time for both towers was approximately 14 weeks, of which 1½ were taken to construct the working platforms.

7.3 Use of Concrete

Concrete is used extensively in the construction of structures in coral reef environments, and for that matter any marine environment. Many structures can be constructed of precast concrete elements that are cast by conventional methods and then placed into position. In some cases, such as the base slab of the Pith Reef and Rib Reef towers, the concrete has to be cast in place under the water.

In addition to the usual requirements for all concrete, concrete in marine environments must be selected with special attention to its durability. A marine environment can severely test the durability of concrete, and if improperly constructed the concrete may suffer rapid and serious deterioration. The main factors causing this deterioration and some common preventative measures are listed as follows:

(a) ABRASION. Abrasion is due to the presence of suspended particles of sand and gravel moving about in the water. It can be resisted by increasing the strength and density of the concrete, while at the same time using abrasion-resistant aggregate and ensuring a good surface finish.

(b) MARINE ORGANISMS. Organisms, such as the boring clam, that attack concrete are especially prevalent in tropical waters. Their attacks are normally effective only on weak, porous and soft concrete, and a hard dense concrete surface will usually provide adequate protection.

(c) CHEMICAL ATTACK. Chemical attack on concrete takes place from the action of sulphates and chlorides in the seawater, and is once again most prevalent in tropical waters. Use of sulphate-resistant cement and the avoidance of corners and sharp edges in the design will significantly reduce chemical attack.

(d) CORROSION OF REINFORCEMENT. This is probably the most serious factor that can lead to a deterioration in the strength and durability of structural concrete. In broad general terms, this corrosion takes place in permeable, porous concrete which is exposed alternately to salt water splash and to air, such as in the tidal and wave-splash zones. Salt is deposited in the concrete, thus setting up electro-chemical action which corrodes the reinforcing and leads to the spalling off of the protective concrete. The corrosion is affected by a number of factors including temperature, type of reinforcing, concrete cover, cracks, and type of cement.

Underwater placement of concrete is generally unavoidable when founding a structure on a bare coral reef. Specialized techniques have been developed to ensure that concrete placed underwater is properly and efficiently placed, and that it will develop its required strength and other characteristics. Some of the more common processes include: the Tremie process; underwater buckets; grout intrusion; hydraulic cements; and bagged and sacked concrete. It should be noted that the general effect of submergence on concrete may often be positive. That is, good quality concrete may increase in strength with the passage of time.

In some places of the world, recrystallized coral (coral sand) has very high strengths with excellent engineering properties. On Guam Island, it is used for coarse as well as fine aggregate in concrete mixes. In Hawaii, it is quarried and used in the manufacture of cement. However, in many places, severe concrete distress has occurred due to the use of improperly washed coral aggregate, or the use of salt water in concrete batching. Moss (1976) conducted a brief investigation into the suitability of unconsolidated coral sediments (sand) for use as a concrete aggregate. Tests were conducted to determine the strength of both the aggregate and the final concrete mix using sand from Keeper Reef. The results of his investigations are detailed in the report.

7.4 Erection of Superstructure

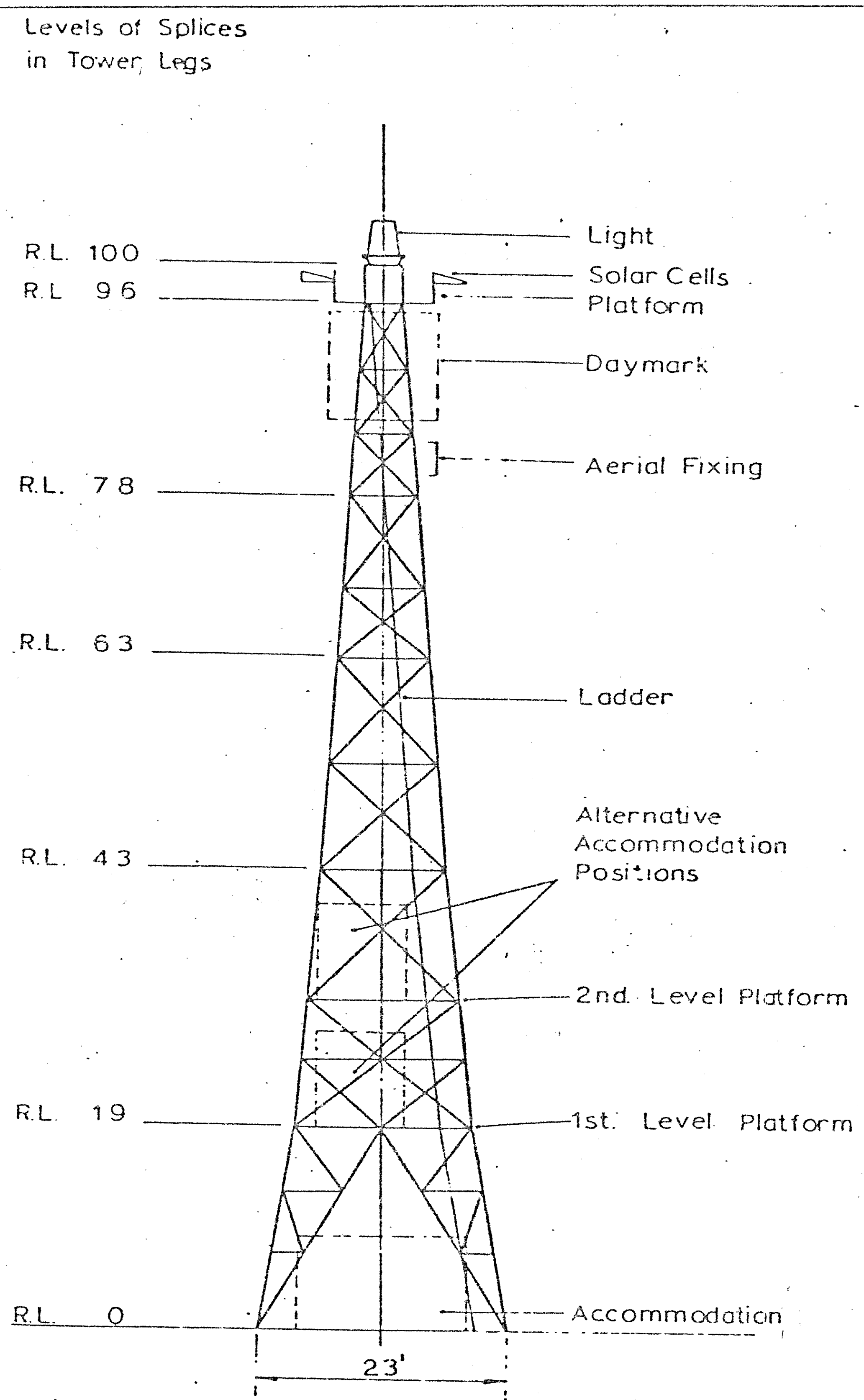
Because of the many difficulties associated with construction in a marine environment, the proposed structure should be designed so as to make the erection of the superstructure as simple and as quick as possible. Probably the most widely used and most efficient method for ensuring this is the utilization of prefabricated sections. Basically, the process involves the joining or splicing together of sections that have been fabricated on the mainland and transported in their entirety to the proposed site.

The number and size of the prefabricated sections depends primarily on the shape or form of the proposed structure, the condition of the site, and the method of transportation to the site. It is fairly obvious that a tower-type structure lends itself to the use of prefabricated sections more readily than a platform-type structure. This is due to the fact that the sections of a tower are all basically the same shape with identical splices, thus greatly simplifying the erection. In the case of a platform-type structure, the sections would be of varying shapes and sizes, thus increasing the complexity of the splices which in turn increases the time of erection.

Because of the limited working space available on bare coral reefs, any prefabricated sections that are to be used for such a situation must be an easily manageable size. This is brought about by the fact that the sections have to be extensively manhandled at all stages of the operation because of the limited amount of auxilliary equipment, such as heavy cranes and hoists, that can be employed in such a situation.

The method by which the sections are transported to the actual site also has to be considered when determining their size. If the sections are relatively small, they can be transported in the hold or on the deck of a ship, whereas large sections may have to be towed to the site. The towing of sections should be avoided wherever possible, because of the many difficulties associated with such an operation.

Figure 7.5 shows the design of a standard, lattice-type lighthouse tower proposed for remote sites off the Queensland coast. The tower consists of six prefabricated sections that can be transported to the site on board the lighthouse tender ship. The sections are spliced at the levels shown in the diagram, and they are transported from the tender to the actual site on the reef by means of amphibious vehicles known as LARCS. The whole design was geared towards a tower that will require the minimum of on-site labour, and time, for its erection.



PROPOSED STANDARD 100 FT.
LIGHTHOUSE TOWER

FIGURE 7.5

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

When planning a major engineering structure, a thorough site investigation is probably the single most important step. Although it can be initially expensive, a site investigation may often save money in the long run by ensuring that the structure is not over-designed, and by detecting problem areas that would have otherwise been discovered during construction, thus leading to an exorbitant increase in cost. When the proposed structure is on a coral reef, the importance of the site investigation is even more prominent because of the extreme variability of a reef mass.

The different site investigation techniques applicable to coral reef situations were discussed in Section 5.1. Comparison between the methods indicates that penetrometer tests are best suited to a coral reef situation. They can be used to accurately indicate the presence of problem areas, and the relative strengths of layers in the reef mass. Where it is proposed to utilize piles for the foundation, tests should be carried out at each individual pile position, since the variability in extent, thickness and competence of coral strata within the overall mass can be extreme.

Without exception, piles are the most suitable foundation for a structure on a coral reef. Existing structures on the Great Barrier Reef are almost exclusively founded on driven steel piles, although prestressed concrete piles could be applicable for larger structures. Cast-in-place or pressure grouted piles could only be used in situations where investigations have proved beyond doubt that the reef mass is extremely uniform and coherent.

In cases where the structure is massive, and very large prestressed concrete piles are required, driving can create special problems. On the main island of Hawaii, where high-rise buildings are supported by prestressed concrete piles, all skin friction capacity is neglected when estimating pile capacities. This is because the brittle nature of coral fragments results in particle crushing rather than densification when compacted or when a pile is driven into it.

The costing involved in construction on a coral reef can lead to great flexibility in the foundation design. The cost of driving the actual piles is insignificant when compared to the cost of getting the men and equipment out to the site. This means that the increase in cost incurred by driving a pile deeper than specified in the design is negligible. So, therefore, if the coral is weaker than was expected and the pile was not achieving the required resistance or set at the depth specified in the design, driving should continue until sufficient resistance is encountered.

A structure in a coral reef environment should be designed so that the utilization of prefabricated sections is taken full advantage of. This is in order to keep the actual on-site construction down to a minimum. Corrosion-resistant materials should be used wherever possible, in order to keep maintenance to a minimum and to increase the long-term strength of the structural members.

In summary, when designing a structure on a coral reef, the engineer should have a thorough knowledge of the proposed site and he should allow for a certain amount of flexibility in the foundation details. Also, wherever possible, the design engineer should be on site when the piles are driven so as to enable him to determine whether any variations in the design are necessary.

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APPENDIX A

LITERATURE REVIEW

A detailed literature review was undertaken in order to obtain any publications concerning structures on coral reefs. This proved extremely difficult, and no publications specifically relating to design and construction on coral reefs were located. There were innumerable articles found concerning general offshore structures in a variety of marine situations. These were mainly based on work carried out by the Petroleum industry in areas such as the North Sea, the Gulf of Mexico, and Bass Strait. Although these articles do not refer to coral reefs whatsoever, some of the basic design and construction principles illustrated in them could be applied to any type of marine environment.

Some of the journals and indexes that were thoroughly investigated include: The Engineering Index; The Geodex Soil Mechanics Information Service; The Geomechanics Abstracts; The Offshore Technology Conference series; and the Civil Engineering in the Oceans Series. The Offshore Technology Conference Series of Publications yielded the best results by far.

Given below is a list of articles that may be of interest when planning a structure in a coral reef environment, and for that matter any type of marine environment.

A.1 ENGINEERING INDEX

1. 1975 Vol. 74, Part II

   No. 044066

   "Offshore Regulations and their Impact on Permanent Offshore Structures"

2. 1978 (Feb.) Vol. 16 No. 2

No. 012175

"Offshore Structure Reliability Engineering".

3. 1978 (Oct.) Vol. 16 No. 10

No. 076178

"State of the Art: Behaviour of Structures and Structural Design".

4. 1976 Vol. 75 Part II

No. 043556

"Feasibility of a Fixed Platform for 1300 feet of Water".

A.2 OFFSHORE TECHNOLOGY CONFERENCE PROCEEDINGS

1. 1977, Vol. I

No. OTC 2802

"Inspection and Monitoring of Concrete Structures for Steel Corrosion".

2. 1977, Vol. I

No. OTC 2746

"Statistical Design Basis for Obtaining Additional Information for the Design of Offshore Platforms".

3. 1977, Vol. I

No. OTC 2794

"Wave Loads on North Sea Gravity Platforms: A Comparison of Theory and Experiment".

4. 1977, Vol. II

No. OTC 2809

"Platform Verification - A View from a Member of the Industry".

5. 1977, Vol. II

No. 2861

"Application of Regulations and Design Codes for Offshore Installations".

6. 1977, Vol. II

No. OTC 2863

"Norweigan Regulations for Design of Offshore Structures".

7. 1977, Vol. III

No. OTC 2944

"Response of Offshore Piles to Cyclic Loading".

8. 1977, Vol. IV

No. OTC 2961

"Mackeral/Tuna Platform Design and Installation".

9. 1977, Vol. IV

No. OTC 3028

"Offshore Platform Risk".

10. 1976, Vol. I

No. OTC 2477

"Controlled Piledriving Above and Under Water with a Hydraulic Hammer".

11. 1976, Vol. I

No. OTC 2503

"Performance of Mat Supported Jack - Up Drilling Rigs".

12. 1976, Vol. II

No. OTC 2553

"Offshore Platforms: Observed Behaviour and Comparisons with Theory".

13. 1976, Vol. II

No. OTC 2604

"Fatigue of Structural Steel for Offshore Platforms".

14. 1976, Vol. II

No. OTC 2607

"Fatigue Design of an Offshore Structure".

15. 1976, Vol. II

No. OTC 2608

"Probabilistic Fatigue Analysis of Fixed Offshore Structures".

16. 1976, Vol. IV

No. OTC 1949

"The Role of Research in the Design of Concrete Offshore Structures".

17. 1976, Vol. IV

No. OTC 1948

"Vortex Excited Structural Oscillations of a Circular Cylinder in Steady Currents".

18. 1976, Vol. IV

No. OTC 1958

"Corrosion in the Offshore Environment".

19. 1976, Vol. IV

No. OTC 1962

"Metal Exposures at Tropical and Marine Sites".

20. 1975, Vol. II

No. OTC 2311

"Pile Load Tests in Calcerous Soils Conducted in 400 feet of Water from a Semi-submersible Exploration Rig".

21. 1975, Vol. II

No. OTC 2333

"Numerical Calculation of Storm Surges: An Evaluation of Techniques".

22. 1971, Vol. II

No. OTC 1405

No. OTC 1406

No. OTC 1407

23. 1970, Vol. I

No. OTC 1180

"Wave-Exciting Forces and Moments on an Ocean Platform".

24. 1970, Vol. II

No. OTC 1311

"Submarine Placing of Concrete by the Tremie Method".

APPENDIX B

RESEARCH ACTIVITIES

The majority of the information on which the discussion in the preceeding text was based came from sources associated with construction on the Great Barrier Reef. Efforts were made to obtain information from overseas sources, and some of the organizations that were contacted, include: Dames and Moore, an American engineering company with experience in the field of construction on coral rock (mainly in Hawaii); the Museum of Natural History in French Polynesia; CEA and ORSTOM, scientific organizations in Tahiti; and the Australian Embassy in the West Indies.

The above contacts yielded no specific information about the design and construction of structures on coral reefs, apart from things that were already known from experience gained on the Great Barrier Reef.

The main avenue of research concerning structures on our own reef was the Commonwealth Department of Construction, whose head office was in Melbourne at the time, but is now in Canberra. During a four day visit to Melbourne at the beginning of July, research was undertaken at the Department's head office and visits were also made to the CSIRO Department of Applied Geomechanics, the Department of Transport, and the Department of Harbours and Marine. Some time was also spent in Brisbane contacting the Department of Mapping and Surveying, and the Queensland Divisions of the Departments of Construction and Transport.

The information obtained from the abovementioned was invaluable in ensuring the progress and value of the project.