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JET PUMP SYSTEMS FOR MAINTAINING TIDAL ENTRANCES

REPORT No. P.W.D. 87051

PUBLIC WORKS DEPARTMENT, N.S.W.
ENGINEERING DIVISION

JET PUMP SYSTEMS FOR MAINTAINING TIDAL ENTRANCES

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Cover Photograph

"The entrance to Tuggerah Lakes closed to tidal flows by accreted beach sands, 27th April 1987."

FOREWORD

Wyong Shire council requested this study into the technical feasibility of utilising jet pumps to maintain a permanent tidal channel at The Entrance. The use of jet pumps in the marine environment is only recent to Australia and could have potential for widespread application. This study evaluates the mechanical and operating features of the equipment as well as site specific considerations concerning implementation at The Entrance.

This report addresses Council's request to investigate the option of using jet pumps to maintain a tidal channel at the entrance to Tuggerah Lakes. It does not examine alternative systems and only provides an overview of the prevailing hydraulic processes.

The use of jet pumps is only one of a number of options available for managing the Tuggerah Lakes entrance. This report provides information to assist Council to evaluate this option with the alternatives so that an appropriate management strategy can be developed for the area.

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SUMMARY

This study is directed at the application of jet pump technology to maintenance of small tidal entrances in general and to the Tuggerah Lakes entrance in particular. It does not set out to analyse in detail the natural processes controlling the closure of the Tuggerah Lakes entrance, nor does it set out to determine the optimum means of achieving permanent opening of the entrance. The study does, however, address in detail how jet pump technology might best be applied in an attempt to maintain a permanent opening in the Lakes system.

A jet pump is a device which transports ambient fluid (eg, a sand/water mix) by the entrainment action of a jet of high velocity water emanating from a nozzle and flowing across an entrainment gap into a mixing chamber. The jet pump has no moving parts or electricity supply, relying for its energy on a high pressure motive water pump located elsewhere, which in the context of tidal inlets would be in a pumphouse on nearby stable ground.

As part of the study, literature relating to the processes operating at tidal inlets and to jet pumps was examined. Advice was also obtained from people and organisations with experience in jet pumps. The only permanent coastal jet pump installation in Australia is the $7.2 million plant at the Nerang River entrance on the Queensland Gold coast. That plant is one component of a $40 million entrance stabilisation scheme designed to bypass littoral drift and thereby prevent the formation of a hazardous bar at the river entrance.

There is no installation known where jet pumps alone have been able to maintain a permanent tidal opening. United States experience with jet pumps is restricted to shoal reduction within trained entrances or transfer of littoral drift across trained entrances, as at Nerang.

This study has found that jet pumps will not maintain the **location** of an untrained tidal inlet channel per se. The jet pumps could be outflanked and rendered ineffective by channel migration. To overcome this, some form of restraining wall or walls would be required to fix the channel location above the jet pumps.

If jet pumps were used, they should be located near the ocean end of the inlet, preferably at the natural point of minimum width. To avoid damage, the pumps and associated pipelines must be laid below flood scour depths.

The jet pumps should be in sufficient number and at an appropriate depth to create a combined crater width that would occupy most of the natural (or design) channel width. This would minimise circumvention of the jet pumps by sand feed along the channel margins. Experience has shown that jet pumps excavate a crater with side slopes of 1:1 to 1:1.5.

At Tuggerah Lakes, two jet pumps located at 10m below mean sea level and having a capacity of 50 tonnes per hour of sand (each), would be adequate to maintain a permanent non-navigable opening to the sea in conjunction with ancillary training works.

The pre design cost of a jet pumping scheme for Tuggerah entrance is estimated at $820,000 capital cost and $54,000 annual cost. This is considerably greater than the cost of the present management approach of ad hoc bulldozing, but much less than entrance training walls. The capital cost includes the cost of a channel restraining wall and contingencies. The latter incorporate an allowance of $150,000 for rock excavation that may be needed to locate the jet pumps 10 m below mean sea level.

The major problem with jet pumps is blockage of the sand feed following accumulation around the jet pumps of neutrally buoyant materials such as plastic, cans, other rubbish and perhaps most significantly, kelp. The entrance does experience heavy kelp build-up at times. It is not possible to accurately predict how often such debris would need to be cleaned away by divers. Each cleaning operation would be difficult and expensive.

The study has found that whilst jet pumps do offer a feasible solution which is intermediate in cost between entrance maintenance by occasional cutting of a channel with a bulldozer and entrance training works, they may be labour intensive to operate and may attract considerable criticism because of their susceptibility to blockage.

The function of jet pumps at Tuggerah Lakes could be performed by standard mobile plant which does not suffer the operational uncertainties of jet pumps and could thereby be guaranteed to perform its designed function.

1. INTRODUCTION

1.1 Background

There are a large number of tidal inlets on the NSW coast which lead to coastal lakes. Typically the lakes have a small tidal range and a relatively small tidal prism. The tidal prism is the volume of water entering and leaving the lake on each tide. In some of these inlets the small tidal prism results in the entrance being dominated by shoaling processes and subject to closure at irregular intervals, with the smaller inlets closing more frequently than the larger. Such closures generally cause a deterioration in water quality and result in higher flood levels than would occur if the inlets were open. This effect is marked for minor floods but less so for major ones.

Local government authorities have responsibility for the management of these coastal lakes and are assisted in this by various Government Departments. The Public Works Department of NSW (PWD) provides technical advice in respect of tidal processes. In addition, under the provisions of the Rivers and Foreshores Improvement Act, approval is required from the Department before excavations can be made from the bed and banks of tidal waterways. Approvals to the opening of coastal lakes and lagoons are generally provided without delay.

Wyong Shire Council is actively engaged in the management of the Tuggerah Lakes System. The Lakes' tidal entrance to the sea is located at The Entrance. Historically, the entrance has been predominantly open to the sea. The report of the Inter-Departmental Committee on Tuggerah Lakes, 1979 (Ref 15) noted that the entrance had closed nine times in the previous 100 years. In recent years closures have been more common and persistent. Wyong Shire Council has suggested this is due to two factors:

* The lack of major rainfall events necessary to create sufficient runoff to purge the entrance channel of sand shoals.

* The need to artificially open the entrance (when it is blocked) after the Lakes have risen only one metre above mean sea level.

A high head would allow a better opening to be scoured with more sand being removed from the immediate vicinity of the entrance. However, nuisance flooding experienced by development around the foreshores necessitates entrance opening at lower lake levels and more sand thereby remains to effect subsequent entrance closure.

Wyong Shire Council would like to have a permanent tidal entrance to Tuggerah Lakes to:

* increase tidal flushing in an effort to improve water quality

* maximise productivity of the fishery

* reduce nuisance flooding

* provide as pleasant an environment as possible for its residents and to attract tourists and tourism related development

* reduce the need for artificial opening with the attendant complaints and criticisms that are raised when the entrance closes

Council has pointed out that as they develop water supply schemes in the catchment, runoff to the lake will be reduced necessitating some form of lake entrance improvement works to prevent exacerbation of the closure problem.

Council has also indicated that it does not favour a trained entrance formed by two training breakwaters because of the cost, aesthetic impact, resulting reduction in sandy foreshores and the inflexibility of such massive works if problems occur after construction such as unanticipated shoaling. A large trained entrance could also increase the Lakes' tidal range and lower mean lake level which is on average super-elevated 0.2 m above mean sea level. The increased tidal range and lower water levels would create foreshore problems around the Lakes by exposing the weed and mud in the shallow lake margins.

Kirrawee Engineering Services Pty Ltd were approached by Council to devise a scheme whereby sand could be pumped away from the entrance channel using submerged jet pumps. Such an approach was thought to have the potential advantages of a relatively low cost, largely unseen and hence of low visual impact and flexible in that the amount of sand removed could be regulated by varying the pumping duration each day.

Council requested the Department to assess the technical viability of the scheme. The study has been carried out at no cost to Council through funds made available by the Minister for Public Works and Ports.

1.2 Aims and Scope of the Study

The objective of the study is to investigate and report on the feasibility of deploying jet pumps to maintain tidal entrances to coastal lakes in general, with specific reference to the Tuggerah Lakes entrance. The scope of the investigation has been limited to the assessment of available data and to engineering judgement. Detailed analysis of coastal processes dominating the closure mechanism were deemed to be unwarranted in a study of this nature due to the time and cost involved in their determination.

Specifically the study is aimed at defining:

* operating principles of the jet pump

* the effectiveness of jet pumping in improving entrance channel stability

* the impact of jet pumping on coastal processes generally
* the effect of the coastal environment on the jet pumping installation
* the practical aspects of the system
* capital and operating cost
* safety of the plant and of the public
* conceptual jet pumping schemes for the entrance to Tuggerah Lakes (if applicable)

The above aspects have been addressed in six distinct parts of the investigation namely:

* literature review of jet pumps and coastal process theories
* definition of jet pump characteristics
* review of previous operational and trial jet pump installations
* review of tidal entrance theories with respect to jet pumping systems
* examination of the characteristics of the Tuggerah Lakes entrance
* the development of conceptual jet pumping schemes for the Tuggerah Lakes entrance

This study principally addresses the potential of jet pump technology, with other matters being treated only in a manner sufficient to make a professional judgement regarding their impact on jet pumps and vice versa. That is, only an overview was obtained of the natural processes causing the closure of the Tuggerah Lakes entrance. In this regard it is important to note there may be other more viable (eg economical/environmental, etc) options available to manage the entrance.

1.3 Literature Review

1.3.1 Sources

The literature survey was commenced by referring to two data bases. The first was the "Delft Hydro" data base of abstracts and citations on hydrodynamics and hydraulic engineering. This was accessed via the Canadian QL data base system. The second source was the "General Investigations Tidal Inlets - Report No 4" (GITI 4) by Barwis of the US Army Corps of Engineers. From these and secondary sources the list of references in Section 8 were selected as relevant to the present study.

GITI 4 is very extensive containing 1069 references most of which are annotated. It refers also to other bibliographies and to research in progress at a few universities as at 1976. The accent is heavily in favour of coastal process description with little reference to jet pumps. Over half of the articles are purely descriptive of particular sites, mostly in the USA.

Most of the references to jet pumps were obtained from the Delft Hydro data base.

1.3.2 Coastal Processes

The coastal process references listed in Section 8.2 comprise the major works of general application to the study of tidal inlets. Inclusion in this list was dependent on the article drawing inferences concerning hydraulics or sediment transport in a way as to be applicable to this investigation.

Blench (Ref 9) and Engelund and Hansen (Ref 10) dealt with sediment behavior in unidirectional flow. O'Brien (Ref 5,6) and Jarrett (Ref 7) addressed the problem of regime channels carrying tidal discharges. Bruun (Ref 12,13) directed his attention principally to the interaction with the wave driven coastal processes. Williams (Ref 16) was valuable as an extensive catalogue of the types of tidal inlets on the NSW coast.

1.3.3 Bypassing Projects

No references were found referring to the use of jet pumping plant to maintain tidal inlets in the manner proposed for Tuggerah which suggests that the proposal is novel. However, the articles quoted in Section 8.3 had much in common with the proposed scheme. Some described particular installations employing a crater sink as the collection point for sand bypassing. There was also reference to fluidisation practice where flow from buried pipes was used to significantly reduce the angle of repose of sand bed material.

The article by Richardson and McNair (Ref 19) is a detailed procedure for design of a jet pump system for use in sand bypassing. This does presume that some form of impoundment will be involved. Furthermore, while it highlights the need for an adequate coastal process studies, it does not address the techniques for such studies in any detail.

1.3.4 Jet Pumps

The references selected in this section were those which dealt principally with the jet pump as a item of plant. Unfortunately, there were no references containing cost parameters for the operation of jet pumps.

2. JET PUMP CHARACTERISTICS

2.1 Basic Principle

The jet pump is a device which uses the energy of a high velocity jet to entrain the fluid (be it liquid or a sand slurry) to be pumped and to develop the pressure head necessary to pump the resulting mixture through the discharge line. The normal mode of operation involves the jet pump immersed in the fluid to be pumped. The jet pump has no moving mechanical parts, the work being done entirely by the fluid pumped through the unit.

The essential physical components are shown in Figure 1. They comprise a nozzle, a mixing chamber and a diffusion section. All these components are circular in section and axisymmetric about the centreline of the pump. The fluid to power the system - generally designated motive fluid - is provided by an auxiliary or motive fluid pump which is external to the jet pump. The delivery line from the motive fluid pump is connected to the nozzle where pressure head is largely converted to velocity head in the resulting jet. The geometry and finish of the converging and parallel sections of the nozzle determine the head losses in the nozzle which can be minimized by using smooth streamlined shapes.

The gap between the nozzle and the mixing chamber provides a flow path for the sand laden water to gain access to the mixing chamber. The mixing chamber and the diffuser form one unit with the discharge line being connected to the diffuser.

2.2 General Layout

The use of jet pumps typically involves the pumping of slurries from bins or ponds. Where the pumps must be portable, a premium is placed on ruggedness and compactness. The Genflo "Sandbug" available from Kirrawee Engineering Services Pty Ltd and shown in Figure 2 meets these requirements. It is one of a family of jet pumps available from an Australian distributor. In the initial stages of the investigation, no other Australian source of jet pumps had been located. It is now understood that Associated Pump Service Co Pty Ltd also provide jet pumping equipment.

The pump illustrated in Figure 2 has three flanged connections. Two of these connect to the water supply line from the motive water pump and one connects to the discharge line. Of the two motive water input connections, one provides water to the pump nozzle and one serves the fluidising nozzles. There is an alternative layout with only one connection to the motive water supply which serves both the jet nozzle and the fluidising nozzles. The proportion of flow to the fluidising nozzles is fixed by the number and diameter of the nozzles.

Water from the motive water pump flows down the left hand pipe (Figure 2) for the length of the unit and turns through a 180 degree bend which is connected to the jet nozzle. After flowing across the intervening space, the flow enters the mixing chamber where it entrains the sand water mixture. From the mixing chamber, the flow passes through the diffuser to the discharge line. Water to the fluidising nozzles flows down the right hand pipe and through a 90 degree bend to the fluidising nozzle array. The flow from the fluidising nozzles is used to break up loosely consolidated sediments.

2.3 Operating Principles

2.3.1 General

The jet pump operates by a transfer of momentum from the motive fluid to the driven fluid within the mixing chamber. In this it differs from the venturi pump which has certain superficial similarities but whose operation basically depends on generating a low pressure region in contact with the driven fluid.

The following description refers to dimensions and quantities shown in Figure 1.

The motive fluid is delivered to the nozzle at a pressure P_n and a flow rate of Q_n. The pressure P_n refers to the pressure in the motive fluid line and not the lower pressure which occurs in the nozzle. In any installation these parameters are virtually unaffected by downstream conditions. They depend on the motive fluid pump characteristic, the head differences and friction losses in the line, the nozzle diameter and environmental pressure P_s. While P_s may vary, the change is generally small particularly when viewed as a percentage of the total pressure in the system.

The high velocity jet from the nozzle passes across the intervening space to the mixing chamber. This space is of the order of one or two nozzle diameters. Although some entrainment occurs in this gap, most of the mixing occurs within the mixing chamber. The sand slurry is drawn into the mixing chamber through the annular space between the high velocity jet and the chamber wall. At the entry to the chamber, the jet is present as a high velocity region surrounded by an annulus of low velocity driven fluid.

At the exit from the mixing chamber, the velocity profile has achieved a uniformity appropriate to turbulent pipe flow. The length necessary to achieve this mixing depends on the area ratio but is typically of the order of six chamber diameters. This momentum transfer results in an increase in the total flow to Q_d, where $Q_d = Q_n + Q_s$, and a rise in pressure. The purpose of the diffuser - where fitted - is to convert velocity head into pressure head while providing an outlet diameter to match the diameter of the discharge pipeline.

2.3.2 Head Discharge Curves

The head discharge curves for jet pumps are frequently expressed in terms of pressure and mixing ratios as defined in Figure 1. As shown, the form of the curve depends only on the area ratio; ie, the area of nozzle divided by the area of the mixing chamber. The operating point on the curve will be determined by the head loss characteristic of the delivery pipe work.

It is instructive to consider the two extreme conditions of zero entrained flow (Q_s = 0) and zero pressure ratio ($P_d = P_s$).

Where the entrained flow is zero, the jet expands to fill the diameter of the mixing chamber. Were this expansion to occur within a well tapered diffuser, the energy losses would be small and the time rate of change of momentum would determine the pressure developed. The fact that the expansion is occuring in a fluid means that there will be momentum exchange between driven and motive fluids even though no nett entrainment occurs. These exchanges give rise to energy losses which prevent the full theoretical pressure being developed. It should be noted that the zero mixing ratio limit does not represent zero flow in the delivery line but rather flow of the motive fluid only. It is of course the lowest condition of any practical usefulness since it represents the zero useful work limit.

At the opposite limit of zero pressure ratio, the degree of entrainment of driven fluid is such that momentum is conserved and the pressure developed in the delivery line equals the pressure in the fluid surrounding the jet. For these conditions to apply there would need to be no head loss in the delivery pipework, a situation not normally encountered in the field.

Between the two limits described, the degree of entrainment will vary until the pressure developed in the delivery line matches the head required to drive the resulting flow through the delivery pipework.

2.3.3 Efficiency

Efficiency is measured as the ratio between the useful work done on the driven fluid and the work done by the motive fluid. The motive fluid will always form part of the flow delivered by the jet pump and the efficiency of energy transfer from the motive fluid to the driven fluid will be limited by the fact that it is by way of turbulent entrainment and mixing. For these reasons the jet pump is inherently a low efficiency device. Peak efficiencies are in the order of 35 per cent and operating efficiencies can be as low as 20 to 25 per cent. This low efficiency is the price paid for having a small mobile pump with no moving parts.

Where delivery lines are long and/or pumping is required for more than a small percentage of time, an in line centrifugal pump is fitted downstream of the jet pump. This means a significant reduction in power costs at the price of some additional capital expenditure. If slurries are being pumped there is an additional virtue in this combination. The jet pump provides a more uniform pulp density to the centrifugal pump than would be achieved by a simple drag suction or even cutter suction unless these were under the constant supervision of an experienced dredge operator.

2.4 Safety

2.4.1 General

The installation of jet pumps in tidal entrances could give rise to several considerations of safety. There is the potential for injury by direct contact with the high velocity jet. There is the consideration of the induced flows being a hazard as well as the risk posed by the steep side slopes to the crater formed by the pumps.

2.4.2 Direct Contact

An effective pumping installation would have the pumps installed some depth below water surface. Typically this will be 7 to 10 m. Furthermore, in entrances subject to a periodic influx of kelp or other trash, a screen would need to be fitted some distance above the pumps. For a pump 10 m below water surface (as shown in Figure 12), the screen would be set 6 m below water surface so as to provide a large screen area to prevent blockage. This screen, while being deep enough to be beyond the reach of most swimmers, would also provide an effective barrier to direct contact with the pumps.

There remains the possibility of fishing lines being entrained in the pump. While this would not prove a problem to the pumps it would be an annoyance to fishermen until such time as the local lore made them aware of the risk.

2.4.3 Induced Velocities

The flow of water induced by the pumps would be insignificant compared with the tidal flow naturally present. A typical water flow induced by the pumps would be of the order of 0.02 m^3/s compared with typical tidal flows of at least 10 and in most cases 100 m^3/s. Hence, the induced flows would not be a hazard.

2.4.4 Crater Side Walls

The presence of the steep side walls to the craters could pose a potential threat to non-swimmers. The threat could arise in circumstances where the entrance channel generally was shallow enough to permit wading at slack water. At times other than slack water, the velocities in tidal entrances would be such as to make wading a hazardous undertaking for non-swimmers, even without the presence of craters. However, in these circumstances the crater would pose a hazard to popular night time recreational prawning operations which, amongst other locations, concentrate on the entrance channel on the ebb side.

The risk to a non-swimmer would be that he would walk on to the steep side slope of the crater and suddenly be out of his depth in an otherwise safe environment. This risk would be reduced by the fact that at slack water, the deeper water in the crater would be apparent. However, the risk although small, would still warrant consideration and it would be prudent to provide warning signs alerting people to the hazard.

Swimmers should not have difficulty since even if they were to swim across the crater, it is only of the order of 20 m diameter.

3. OPERATIONAL AND TRIAL JET PUMP INSTALLATIONS

3.1 Introduction

Information was gathered regarding jet pump installations in Australia and overseas. A major installation in Australia is located at Nerang (Queensland) where it is the basis of a sand bypassing plant which is part of the Nerang River Entrance Improvement Works. The only permanent installation found in NSW is at Bayswater Colliery where the application is different from that proposed for tidal inlets.

In fact, no scheme could be found that was similar to that proposed at Tuggerah Lakes entrance. The main difference is that the proposal at Tuggerah, as put forward by Kirrawee Engineering Services Pty Ltd, is intended to maintain an open entrance and to stabilise the channel location.

United States applications of jet pumps along with the Nerang installation are associated with entrances permanently located by training works. The purpose of the pumps is to bypass sand or to remove unwanted shoals. However, a requirement that jet pumps alone fix the location of a channel is a more demanding one than that of shoal removal.

In NSW, various tests and small projects using jet pumps have been undertaken in estuaries. They all encountered operational difficulties. A few of these were due to operator inexperience but most were due to blocking of the jet pump intake with coarse or elongated material in the sand being pumped. These tests clearly demonstrate the importance in a permanent installation of careful design in the pipework system and the need to incorporate features that can cope with material capable of blocking the jet pump.

Details of some of the jet pump applications to date are given below.

3.2 Nerang

3.2.1 General

The jet pumping installation at Nerang on Queensland's Gold Coast was designed to bypass the northerly littoral drift across the tidal inlet of the Broadwater. It is similar to the type of installation recommended by the US Army Corps of Engineers for this purpose.

The entrance to the Nerang River has been migrating northward since at least the time of white settlement in the area. In 1979, the Gold Coast Waterway Authority (GCWA) was formed and since then has been involved in improving and stabilising the Nerang entrance.

The channel improvements have included the construction of two breakwaters resulting in the impoundment of sand in the area immediately to the south of the southern breakwater. The jet pump installation has been constructed within this impounding area and comprises a pier supporting ten jet pumps set 10 m below mean sea level (MSL). This is an example of the total impoundment and by-passing of sand.

The general layout is shown in Figure 3. The motive water pump supplies high pressure water to the jet pumps whose vertical risers discharge into a pipe flume, thence to a hopper supplying the slurry pump. This transfers the sand via a submerged pipeline to the beach to the north of the river entrance, thus maintaining the longshore drift without shoaling the entrance channel or forming an offshore bypassing bar. The cost of the installation was $7.2 million.

The installation was inspected on the 26th November 1986 after being operational for six months. During this time it had bypassed 360,000 m^3 of sand. The performance was satisfactory except for a few problem areas which are described in Section 3.2.2.

The plant is extensively instrumented and is in part operated by a computer based control system. The operating procedure comprises selecting a group of four jet pumps. Of these, three are started and continue pumping until the pulp density in the delivery line falls below a specified level. At that stage one of the three pumps is shut down and the forth started. The cycle continues until all four of the pumps are delivering low pulp densities at which time a 30 minute shutdown cycle commences to purge the entire system of sand.

At the time of inspection, the sand level adjacent to the operating pumps was at approximately MSL making the crater developed at the end of pumping 10 m deep. Measurements on two of the craters gave diameters at sand surface of 18 and 22 m implying a side slope on the craters of around 1 in 1. The wave climate at the time was mild. A more energetic wave climate is likely to have the effect of accelerating the infilling of the crater rather than to produce more gentle side slopes.

Measurements at other times by the GCWA indicate that infilling occurs progressively over a period of three or four days during which time the craters become approximately half full.

3.2.2 Operational Problems

The problems encountered to date involve pump blockage and jet wear. On a few occasions, pumps have become blocked with sand even though they were shut down on water only. The common feature of all these events was that the blockage followed a shut down in excess of three days. Several hypotheses have been advanced but it is most likely that pressure fluctuations due to wave action have forced sand through the mixing chamber and diffuser section into the risers. This would be consistent with the fact that the fluidising nozzles were not effective in cleaning the blockage which had to be removed by rodding and jetting from within the delivery riser.

Another problem was wear in the pump and fluidising nozzles. Whether of grey iron or stainless steel, these were worn to the stage of requiring replacement after 200 hours of duty. This contrasts with the manufacturers expectation of from 600 to 1000 hours life.

In the pump inspected, the wear in both the main jet and fluidising nozzles was of the same type and was consistent with a wear pattern which began at the nozzle face and progressed upstream. It took the form of an almost axisymmetric polished wear surface. In section, it was approximately a small radius circular arc. The possible causes included cavitation, erosion and electrochemical corrosion.

Cavitation would be confined to the metal in contact with the high velocity jet and would not explain the wear across the full width of the nozzle face. If corrosion between dissimilar metals was the cause, one would expect to see it exhibited preferentially at their junction rather than at one extremity.

The most likely mode of failure is the high velocity jet (in excess of 30 m/s) creating a toroidal vortex against the face of the nozzle. This would entrain sand to produce a wear pattern very similar to that observed.

In relating this problem to other proposed installations it must be remembered that this particular installation uses large jet pumps with very high velocities. The problem would be reduced or even non-existent in small sized pumps due to lower velocities.

In the pumps available for inspection, there was also evidence of minor wear in the mixing chamber, appearing as slight longitudinal depressions. These would not have had a noticeable effect on efficiency at their stage of development.

3.3 Bayswater Colliery

The jet pump at Bayswater Colliery in the Hunter Valley was mounted on a barge and used to reclaim coal from a tailings pond. This material comprised coal, shale and sand with 10 per cent being larger than 3 mm.

The jet pump was mounted on a dredge ladder behind a log washer used to break up the deposit. The design pulp density was 20 per cent by weight.

The pump nozzle (stainless steel) and mixing chamber were inspected and they showed absolutely no sign of wear after the 100 hours of operation. While the material being pumped was probably less abrasive than that at Nerang, the most likely reason for the absence of wear was the lower operational velocities.

3.4 Mobile Jet Pump, Canada

The Department of Public Works, Canada has developed a mobile jet pump system for channel maintenance of small fishing harbours in the maritime provinces. The unit is entirely self contained and is mounted on a semi-trailer. Its all up weight is 38 tonnes and it comprises:

* motive water pump
* 6 inch x 10 inch Pekor jet pump
* Hiab hydraulic jib
* centrifugal booster pump
* 50 foot lengths of flexible discharge hose that can be handled by two men

The majority of fishing harbours serviced have small tidal prisms so that the channels are significantly over regime and there is probably no tide driven sediment movement. Problems arise only after storm events and the unit is deployed on demand as a result of notification of a loss in channel depth. Being entirely self contained, the unit can be quickly deployed and it will generally finish any one job in three to four days. Dredging rates are in the range of 85 to 100 m^3/hr. Typical entrance channels are 500 ft long and 40 to 60 ft wide.

The Hiab jib which supports the jet pump has a reach of 40 ft which is adequate for the harbours in question but would limit its usefulness in larger channels. The weight and semi-trailer mounting of the rig requires an adequate hardstand area adjacent to the channel to be dredged. The use of the rig is thus restricted to sites with nearby wharves or similar structures and it could not be used for operations on sandy ground.

Operational experience to date has been good after an estimated 600 hours of pumping. There has been no trouble with wear. Considering that a 6 inch x 10 inch jet pump is used to deliver 100 m^3/hr compared with 50 t/hr from a significantly smaller Genflo "Sandbug" being considered for the Tuggerah installation, it is likely that the Canadian pump is operating at lower velocities. This could account for the lack of wear.

The Pekor pump has been blocked by kelp on a number of occasions. The remedy has been to lift the pump above water surface and to close the discharge valve. This effectively backflushes the system and removes the kelp. From this experience the Canadians would be reluctant to use a fixed installation because of the likelihood of blockage with kelp and other marine detritus. Since the entrance to Tuggerah Lakes has been observed to experience heavy kelp accumulation, this could be a significant problem for the proposal.

3.5 Narrabeen Lakes Entrance

Warringah Shire Council tested a jet pump in and near the entrance channel leading to Narrabeen Lakes. These tests were undertaken on the 5th, 6th, 7th and 19th February 1986 and were subsequently reported by Mr M B Gerstle of Warringah Shire Council.

Three test sites were used. The first two were on a sand bank adjacent to the inlet channel and the third was inside the channel although close to the bank.

The Genflo "Sandbug" used was supplied by Kirrawee Engineering Services Pty Ltd and had a nominal discharge capacity of 50 tons of sand/hr. It was supported from the jib of a crawler crane and supplied with motive water from a separate pump pack. The pump was fitted with a set of fluidising nozzles for the purpose of aiding penetration into cohesive or compact granular material and to promote the suspension of sand as feed to the jet pump.

The first two tests confirmed the pumps ability to penetrate sand unaided to a depth of 2 m and indicated that there was no reason why greater penetration could not be achieved. After being shut down for 20 minutes the pump was able to restart without blockage. In one instance the pump ceased to deliver sand while still pumping water. This was due to coarse material packing around the inlet and acting as a reverse filter to prevent the passage of sand.

After two trials on the banks of the channel a site was selected inside the channel for the purpose of testing the ability of the pump to extend its sphere of influence. During this test, the pump established a cone approximately 2 m deep and 4 m in diameter delivering sand at an estimated rate of 40 m^3/hr. This rate was maintained as long as wave action delivered sand to the hole. However, when the waves abated, the solids content in the delivery line fell. The hole remained essentially the same dimensions and its geometry appeared to be unaffected by the tidal currents present during the test.

In summary, the tests indicated that the pump was able to develop a conical pit with side slopes of 1 in 1 and while material was available it delivered sand at a rate approximating that claimed by the manufacturer. The pump did not by its own action extend the area from which sand was pumped and depended on external factors to deliver sand to the cone. After an admittedly temporary shutdown, starting was achieved without any sign of blockage of the pump or lines with sand. However, after some time, coarse material in the deposit did tend to pack around the intake zone, thereby preventing the ingress of sand.

3.6 Tuggerah Lakes Entrance

Wyong Shire Council conducted a test of a "Sandbug" jet pump in the entrance channel leading to Tuggerah Lake on 3rd February 1987. The test site was located in 1 m of water, approximately 100 m upstream from the ocean and 5 m from the southern channel bank at low tide. The pump was suspended from a flat bottom punt and supplied with motive water from a land based pump pack.

The test ran from 7 am to 3 pm covering both an ebb and flood tide with tidal ranges of 1.1 and 1.2 m respectively. By 8.40 am the jet pump had established a crater 3.5 m in diameter and 1.5 m deep, resulting in side slopes of around 1 to 1. There was no sediment in suspension during the ebb tide although the bed formed small ripples about 200 mm long and 10 mm high. Tidal velocities were approximately 0.8 m/s. Deeper penetration of the pump was extremely difficult and when the bed was probed there were indications of clay layer 2 m below the sand bed level.

On reversal of the tide, the punt moved slowly upstream and the pump was able to elongate the crater to give a final top length of 8 m. Velocities in the lower part of this hole were virtually zero even when the channel velocity reached 1 m/s. The pump was shut down at 10.30 am, having pumped only a small quantity of sand due to a lack of feed to the crater.

Observations of the crater were continued to observe its mode of infilling. Under flood tide velocities of 1 m/s the downstream face maintained its 1 in 1 slope and advanced upstream at 0.023 m/min. This gave a measure of sediment flux of 0.015 m^3/m width/min. There was a small quantity of sand in suspension. The upstream face of the crater showed only minor erosion which suggested that sediment transport was being satisfied in part by lateral diffusion from regions not in the lee of the crater.

At 12.30 pm a second test was commenced in the same general area. The pump again developed a conical crater, this time 1.5 m deep and 4.0 m diameter. This crater was again axisymmetric and the calculated side slopes were 1:1.3. Penetration deeper than 1.5 m was not possible due to a firmer layer being encountered. Operations were also made more difficult by flood tidal currents of 1.2 m/s. As well as bed movement, sand was being transported as suspended load in the lower 0.25 m of the water column. Sampling showed a suspended sediment concentration in this layer of 1 gm/l.

The delivery line blocked with sand on several occasions during the second test. This was most likely due to the method of operation in which water could be directed either to the jet pump or to the fluidising nozzles. Blockage occurred when the proportion of flow being directed to the fluidising nozzles rose to about 50 per cent. This is not a condition to be expected in normal operation.

However, once blocking occurred, the pump was unable to clear the delivery line even when operating with the full water flow.

The test was terminated at 3 pm.

From these tests the following conclusions were drawn.

* The jet pump will easily develop a conical crater in a sand deposit.

* Firm material such as clay or compact shell layers may limit its depth of penetration.

* The side slopes of the crater developed are within the range of 1:1 to 1:1.5 and do not appear to be influenced by tidal velocity.

* When the delivery line was blocked the jet pump was not able to clear it unaided.

3.7 Jerrys Plains Intake

Elcom NSW operates a pumping station on the Hunter River at Jerrys Plains to supply make-up water to Lake Liddell. As a result of prolonged low flows in the early 1980's, an accummulation of sand was restricting flow into the intake structure. Water depth at the intake was 1 to 1.5 m, maintained by a weir downstream.

Because of access difficulty, the sand from around the intake was cleared using a Genflo "Odd Job 35" jet pump with a 102 mm discharge line. The motive water pump was located on top of the pump station about 2 m above water level. The discharge line from the jet pump ran up to the pump station roof and thence horizontally along that structure, finally running down the river bank so as to discharge the sand into the river downstream of the weir. The length of the discharge line was between 20 and 40 m and the maximum rise was 3 m.

The jet pump moved very little sand in the first 15 minutes and then for the next 15 minutes it delivered a thick sand slurry. After this it pumped no more sand possibly because it encountered a gravel layer. A crane was subsequently used on a number of occasions to move the jet pump to pick up more sand. The final result was a crater 2 to 3 m in diameter and 1 to 1.5 m deep. The total volume of sand removed was 10 to 20 m^3. The discharge from the jet pump ranged from clear water to a thick sand slurry.

There were a number of operational problems. The intake to the motive water pumps was in the river and it picked up some fine gravel which blocked the fluidising nozzles. The discharge line became blocked on occasions when the motive water pump was shut down while the jet pump was delivering a slurry of sand. These blockages seemed to occur in low points in the discharge line and necessitated disconnection of the discharge line for clearing. This problem was subsequently overcome by shutting down on clear water only. On one occasion a large flat stone was jammed in the mixing chamber. Smaller gravel packed around this stopped the pump which then had to be cleared manually.

3.8 Gymea Baths

In order to remove sand from Gymea Baths, the Sutherland Shire Council hired a 50 mm (nozzle diameter) jet pump since it was not possible to gain access with conventional earth moving plant.

The system used floating motive water and discharge lines. Sand discharge was slow but eventually 150 m^3 of sand was removed, increasing water depth in the baths by about 1 m.

The main difficulty experienced was the blocking of the jet pump by twigs which were present in the sand. This necessitated stopping the pump every few minutes to clear the jet intake. The provision of a practically sized screen would not have solved this problem since the screen would have needed frequent cleaning and its presence would have made working the pump more difficult.

3.9 The American Experience

During the course of this investigation, the US Army Corps of Engineers was contacted for their experience and opinion on the matter of using jet pumps to maintain tidal entrances. A long telephone conversation resulted with Mr T W Richardson, Chief, Engineering Development Division, Coastal Engineering Research Center together with J Clausner and J Pope of his division. As a follow up to the telephone conversation, Mr Richardson sent a letter to the Department (Appendix A) enclosing reference material covering the items discussed.

The Corps of Engineers has no experience with the specific type of installation proposed for the entrance to Tuggerah Lakes. However, in the opinion of Mr Richardson and his staff, there is no apparent reason why the installation would not be successful with careful design and the willingness to make minor changes on the basis of operational experience.

They referred to their experience of the wide variety of material that finds its way into jet pump craters. This includes kelp, seashells, beer cans and construction plastics. Some means of clearing this trash is needed in the American context. While the larger American population using the coastal zone could doubtless cause a greater problem with rubbish than in Australia, it was Richardson's opinion that even in the Australian environment the problem would need specific treatment.

A further point discussed was the need for a sediment free water supply to the motive water pumps. It was agreed that drawing water from Tuggerah Lake upstream of the entrance throat would satisfy this requirement.

Mr Richardson also agreed that there was a need to provide a short stabilising wall to the southern bank of the entrance channel in the vicinity of the jet pump craters. He also advised monitoring the beach to the south of the entrance for signs of recession as a result of the jet pump installation. This he saw as a remote possibility but one nevertheless that should be kept in mind.

In the matter of entrance channel stability, Richardson offered the opinion that the removal of sand infeed to the entrance would likely result in an increase in channel depth. This could increase the stability of the channel cross section which may result in a reduced requirement for pumping of sand in order to maintain that stability.

Mr Richardson referred to the Oceanside Harbour experimental sand bypassing plant in California, development of which began in 1985. It is to incorporate two alternative sand impoundment areas serviced by jet pumps on a jack-up barge delivering sand to the southern downdrift beach. It will be similar in concept and scale to the installation at the Nerang River entrance in Queensland (see Section 3.2). One of the main aims of the work is to monitor the performance of the system generally and the jet pumps in particular over a five year period. If the system performs as expected during this time, the experience gained and data collected will be used to design plants for other harbours.

The only American jet pump is one manufactured by the Pekor company. It differs from the Genflo pump in that an enlarged extension of the mixing chamber encloses the jet nozzle. The ingress of the sand slurry to the pump is then at an angle of 45 degrees through the annular space between the nozzle and the mixing chamber extension. The advantage of this layout is an increased pumping efficiency. The disadvantage is that it is more easily choked by material being jammed in the contracting flow passage. Pumps of this type manufactured from Nihard alloy have shown no wear. Some plastics have been used in Pekor pumps but these suffer from wear when used to pump sand.

Mr Richardson referred to the American experience with fluidisers which are perforated pipes laid beneath the sand surface. Water is pumped through them to reduce the stability of the overlying sand and hence reduce its angle of repose. The aim of this is to extend the sphere of influence of pumping plant beyond that given by the 1 in 1 to 1.5 side slopes normally obtained. The success of these plants was variable. The main problem encountered was the blocking of the pipeline with sand on some occasions when the plant was shut down.

Reference was also made to a design by the Sloan Pump Company of a mechanical crater sink pump. This employs an auger feed to deliver sand to a central pump and so has a wider sphere of influence than a single pump. One of Mr Richardson's engineers has viewed an installation using this unit and it operated effectively although there was concern about the wear of the mechanical parts and the likelihood of the need for frequent maintenance.

4. TIDAL ENTRANCES AND JET PUMPS

4.1 Introduction

This section describes the current state of knowledge of the hydraulics and stability of tidal entrances and the effect of jet pumping on that stability. The lack of data presently prohibits the application of some of these concepts to detailed descriptions and predictions of performance of entrances. However, the information provides a good background for understanding the processes operating at tidal entrances which can be supplemented with site specific data. Hence, the material is included notwithstanding that it covers a wider range than is warranted from the perspective of the Tuggerah Lakes entrance.

4.2 Dynamics of Tidal Entrances

The tidal entrances of interest in this assessment are those which connect a coastal lake or lagoon to the sea or to a large embayment. Williams (Ref 16) has catalogued all such lakes with a water surface area in excess of 0.15 km^2. These were divided into 28 lakes which by definition are generally open and 30 lagoons which are frequently closed. They were also categorised in terms of entrance type (whether channel or delta).

4.3 Tidal Behaviour

The following section makes some implicit assumptions regarding the proportions of a lake system, principally that the water surface area of the lake does not change significantly with lake tide level and that the tidal prism of the entrance channel is small compared with that of the lake. It is also assumed that tidal gradients within the lake proper are vanishingly small; ie, the lake surface in windless conditions is horizontal.

The tidal, as opposed to sediment, behaviour of a lake is most easily characterised in terms of tidal range ratio. This is the ratio of tidal range in the lake to ocean tidal range. This does not address the question of why a particular entrance geometry exists but is a sound basis for a description of the hydraulics of a particular geometry.

The values of tidal range ratio for inlets on the NSW coast range from nearly unity to near zero. The former are frequently associated with deep stable channels and the latter with entrances subject to intermittent closure although this is not a rigid association.

Figure 4 shows the water levels and flows for the two types over one tidal cycle. When the range ratio is near unity the lake tide lags the ocean tide only slightly and the head loss in the entrance channel is a small percentage of the tidal semi range (a measure of the maximum head available to the entrance). In this circumstance the flow in the channel may be considered from a viewpoint of kinematics rather than dynamics. Flow will be given by the time differential of lake storage which is the product of water surface area and water level. Slack water will occur at high and low lake tides which lag slightly behind ocean high and low tides. Maximum channel flows will occur at half tide rising and falling and the range of cross sections available for flood flow will be similar to the range of cross sections available for ebb flow.

For very small range ratios the lake water level is, compared with the ocean tide level, nearly constant. Since accelerative terms are small, the flow from ocean to lake is governed by the ocean tide level, channel geometry and friction. As shown in Figure 4 flood flows occur whenever the ocean tide is above MSL and ebb flows when it is below MSL. Continuity considerations then lead to the fact that the maximum and minimum lake levels occur at the same time as ocean mean sea levels. Thus, lake level and channel flow for range ratios approaching zero lag by 90 degrees behind the same variables for a range ratio approaching unity.

For most tidal entrances with low range ratios, the simplified description of tidal flow needs to be modified in order to take account of the generally shallow channel sections. The modification to flow, which can be significant, is a result of the significantly different cross sectional areas available on flood and ebb. By consideration of the phasing of the tidal flow in Figure 4b with respect to the ocean tide levels in Figure 4a, it can be seen that the water level is always above and below MSL for flood and ebb tides respectively.

The larger cross sections during flood tide (other things being equal) would result in a nett water flow to the lake over a tidal cycle. To achieve equilibrium the elevation of mean lake level is raised above MSL so as to change the duration of ebb and flood flows to values that result in zero nett flow to the lake. For the shallower entrances, these durations are typically four hours flood and eight hours ebb, the curve being of the type shown in Figure 4c. Elevation of mean lake levels are typically in the range 0.05 m to 0.20 m.

4.4 Channel Stability

4.4.1 General

The preceeding section dealt with the hydraulics of a lake entrance of a prescribed geometry. This section deals with factors that may determine that geometry.

Independent parameters which have an influence on channel geometry are:

* lake water surface area

* ocean tidal range

Both of these are readily measured and in an engineering time scale are not subject to variation as a result of tidal processes.

Also of influence on entrance channel size or stability are:

* longshore sand transport
* onshore/offshore sand movement
* flood runoff
* sediment properties
* entrance modification by storm waves

many of which are not readily measured.

4.4.2 Regime Relationships

Regime theory assumes that a channel in an extensive granular material will adjust its geometry according to the discharge carried. Blench (Ref 9) developed such relationships largely on the basis of field measurements while Engelund and Hansen (Ref 10) used a similarity analysis to arrive at essentially the same result. The Engelund and Hansen relationships are:

$$W = 0.78\ d^{-0.316}\ Q^{0.525}$$

$$D = 0.108\ (Q_t/Q)^{-2/7}\ d^{0.21}\ Q^{0.317}$$

where
- W = width of channel (m)
- D = depth of channel (m)
- d = mean fall diameter of the bed material (m)
- Q = water discharge (m^3/s)
- Q_t = total sediment discharge (m^3/s)

hence

$$A = K\ d^{-0.106}\ Q^{0.842} \text{ for a constant } Q_t/Q$$

where A = cross-sectional area of channel (m^2)

While these relationships were developed for unidirectional flow it seems likely that similar relationships will apply to tidal channels. The Department (Ref 11) developed a relationship between cross sectional area and peak spring tidal discharge for the tidal channel of the Hunter River. It is of the form:

$$A = 1.935\ Q^{0.863} \quad (Q \text{ in } ft^3/s,\ A \text{ in } ft^2)$$

The cross sectional areas varied from 8000 to 37000 ft^2. The sediment type varied from medium sand to sandy silt which demonstrated the insensitivity of the relationship to sediment size. As such it was in agreement with Engelund and Hansen where the exponent of the grain size was -0.106.

More extensive work on tidal inlets was undertaken by O'Brien (Ref 5) and re-analysed by Jarrett (Ref 7). The relationship derived by O'Brien was:

$$A = 4.69 \times 10^{-4} P^{0.85} \quad (ft^2)$$

where P = tidal prism (ft^3)
A = cross-sectional area of channel (ft^2)

On the basis of a later analysis he concluded that the original expression applied to entrances with training walls (jetties) and that untrained entrances were better described by:

$$A = 2 \times 10^{-5} P$$

Jarrett made separate analyses for the Atlantic, Gulf and Pacific coasts for trained and untrained entrances arriving at values for a, b and c in the equation:

$$A = a\, 10^{b} P^{c}$$

Training Walls		**Atlantic Coast****	**Gulf Coast***	**Pacific Coast**	**All Sites**
None or One	a	5.37	3.51	1.91	1.04
	b	-6	-4	-6	-5
	c	1.07	0.86	1.10	1.03
Two	a	5.77		5.28	3.76
	b	-5		-4	-4
	c	0.95		0.85	0.86
All Data	a	7.75	5.02	1.19	5.74
	b	-6	-4	-4	-5
	c	1.05	0.84	0.91	0.95

* - Diurnal Tides
** - Semi-Diurnal Tides

The significant weakness with both O'Brien's and Jarrett's work was that the majority of the data was for entrances with a tidal range ratio of near unity and the tidal prism was calculated from the lake surface area and ocean tidal range near the entrance.

In lakes or lagoons where the tidal range ratio is significantly less than unity, none of the regime relationships can be used explicitly to calculate entrance channel area from lake surface area and ocean tidal range. However, in conjunction with head loss calculations in a particular channel, these relationships have potential use in deriving potentially stable combinations of area and tidal prism.

4.4.3 Stability Analysis

Escoffier (Ref 8) considered the behaviour of tidal channels and their tendency to closure or to the establishment of a stable equilibrium. The two variables considered by him were the channel cross sectional area and the mean velocity. He postulated that there was a critical velocity which, if exceeded, resulted in scour of the channel. On the other hand, if the channel velocity were less than critical, accretion would occur.

A similar conclusion of a critical velocity for a regime channel can be approximated using the relationships of Engelund or O'Brien as a starting point. The approximation required in the Engelund expression is that the exponent of grain size be zero and the exponent of discharge be unity. In O'Brien's work the approximation needed is that the exponent of tidal prism be unity.

Escoffier then plots velocity as a function of channel area as shown in Figure 5. For large values of channel area in the region shown as D, the tidal range ratio will approach unity and the channel flow will be the product of lake area and ocean tidal range and practically constant. The velocity will then vary as the inverse of the area and, should sediment be available, accretion will occur until the critical velocity is established.

On the other hand, in the region shown as A the velocity is determined by frictional resistance in the channel and this resistance increases with decreasing channel geometry leading to eventual closure. Hence, conditions to the left of B on the diagram lead to closure and those to the right of B to a stable section at point C. The rate at which changes occur will depend on the sediment supply to the channel from external sources.

4.4.4 The Influence of Littoral Drift

From a survey of lake entrances, mainly on the Florida coast, Bruun (Ref 12) concluded that whether they were to be subject to closure or not depended on the ratio of tidal prism to gross littoral drift along the adjacent beaches. Gross littoral drift is defined as the total annual sediment flux towards the entrance from both sides.

Three domains were defined based on the parameter P/M where:

P = tidal prism in m^3
M = gross littoral drift in m^3/year

Domain 1: P/M < 100

Entrances subject to major shoaling leading to the risk of closure.

Domain 2: 100 < P/M < 300

Entrances subject to mild to moderate shoaling.

Domain 3: P/M > 300

Stable entrances with little or no formation of shoals.

4.5 The Effects of Jet Pumps

To date, the use of jet pumps to maintain channels has been in association with breakwaters which serve to provide sediment impounding areas as in the Nerang installation. The impounded material is then bypassed using the jet pumps so that the total system acts as a sediment sink for littoral drift.

In line with the reasoning of Bruun, the reduction in sediment feed to the entrance increases the ratio of tidal prism to sediment feed and hence the entrance channel stability. The tidal discharge is then the motive force forming the channel.

A similar effect could be obtained by intercepting the sediment feed at the channel entrance using jet pumps. Trapping would occur in the conical craters formed by the pumps whose function would be to remove the sand and maintain the craters as a sediment trap. Sand pumping would not need to be continuous but only frequently enough to prevent the virtual filling of the crater sink. Provided these sinks were created near the seaward end of the entrance channel, the effect would be to increase the ratio of tidal prism to gross sediment transport and hence improve the channel stability.

While even a series of small craters would intercept bed load, sediment suspended in the lower levels of the water column would be carried across a crater whose dimensions were of the order of magnitude of the water depth. To be effective, the crater diameter would need to be an order of magnitude greater than the water depth so that there could be a reduction in velocity and turbulence levels as the flow expanded passing through the crater.

A depth of immersion of 5 m or more would be required with side slopes of 1 in 1. It would be desirable to have the largest cone possible to ensure trapping and hence an immersion depth of the order of 10 m would be preferable. A practical benefit of deep immersion would be that fewer pumps would be needed to cover the width of the entrance.

A major problem in attempting to intercept littoral infeed to a tidal entrance without twin training breakwaters is that inevitably sand will still enter the inlet via the channel margins adjacent to the jet pump craters. At many entrances some of this infeeding sand manifests itself as an arc-shaped extension of the beach spit or as shoals welded to the beach berm or back beach. At Tuggerah Entrance, these are generally present on the northern side of the entrance.

While the percentage of sand that enters an inlet via the sub-tidal and inter-tidal margins compared to the deeper "ebb channel" is not known, field observations suggest that it is significant and in some cases may be the dominant pathway. As an entrance gradually closes and its throat is constricted, the magnitude of margin infeed will decrease, but it could remain important in contributing sand to the growing shoals upstream of the throat.

In the smaller lakes and lagoons that are inherently unstable and rapidly close following flood breakout scour, it is more than likely that jet pumps located in the entrance close to the ocean beachline would be outflanked by shoaling upstream of them and the entrance would close. In the more robust entrances, jet pumps could probably maintain a permanent entrance, slow the rate of shoal formation upstream of them and reduce the maximum extent of shoaling if they performed ideally. This task would be facilitated if either a training wall or natural training reef or headland acts to stabilize the beach location on one side of the jet pump craters, thus reducing landward fluctuations in the beach-line that would encourage channel margin sand infeed around the crater flanks.

It is probable that some shoaling would continue to occur upstream of the craters and that high tidal energy losses across this shoaling may hamper significant improvement in tidal flushing of the lake. If improved tidal flushing is a management goal, then maintenance dredging of shoals upstream of the jet pump craters would still be necessary using either a conventional dredge or a mobile jet pump. Alternatively, or additionally, twin training breakwaters could be considered to further intercept the littoral supply of sand moved to the inlet entrance by nearshore wave induced currents.

The jet pumps would have no effect on sediment transport during floods when there is a unidirectional flow from the lake while ever water level remains above high tide level. This condition may prevail for many days depending on the nature of the inflow hydrograph to the lake and the degree of lake storage. However, the flood could have an effect on the security and efficiency of the jet pumps.

The first way a flood could impact on the system would be by scouring the shoals and removing the pumps and pipe work. This risk would be reduced and possibly eliminated by setting both pumps and the pipe work well below channel bed level. This is consistent with the previous requirement of deep immersion so as to develop wider pump craters.

The second risk is concerned with the location of the tidal channels after the flood is passed. Sediment movement and consequent scour during a flood is significantly more extensive and rapid than under tidal flows. The configuration of tidal inlets vary greatly, but it is common for the ebb channel under tidal conditions to meander on small radius bends tortuously through the entrance shoals. Under flood scour, the channel tends to shift against any natural rock training with the flow taking the largest radius curve to the ocean that can be accommodated. At Tuggerah this generally sees the main flood scoured channel move southwards until restrained by the reef. Subsequently the resulting tidal channel is perched on the reef and experiences heavy tidal energy losses. This contributes to more rapid shoaling upstream where the tidal gradients are slight.

If jet pumps were installed at Tuggerah there is no mechanism that would prevent this behaviour continuing - at the end of flood scour the craters would be largely infilled and the deepest part of the entrance would most likely be further south. It would therefore be essential to provide a low restraining wall on the south side of the craters, with this wall being tied into the mainland. The wall would have a low top level to provide for flood overtopping, but would ensure the post-flood channel was located over the jet pumps. It would also assist to train tidal flows, thereby increasing the craters' sand interception ability.

5. CHARACTERISTICS OF THE TUGGERAH ENTRANCE

5.1 General

Tuggerah Beach extends 9 km from Pelican Point to The Entrance (Figure 6). The tidal channel leading to the Tuggerah Lakes system lies immediately to the north of The Entrance adjacent to a minor rock outcrop. The three interconnected lakes in the system and their water surface areas are:

* Tuggerah Lake - 58 km^2
* Lake Budgewoi - 11 km^2
* Lake Munmorah - 8 km^2

77 km^2

The tidal range in Tuggerah Lake varies with the condition of the entrance channel but is typically of the order of 20 mm. Budgewoi and Munmorah Lakes are virtually non-tidal. Because of the asymmetry of tidal flows in the entrance channel, the mean water level in Tuggerah Lake is tidally super-elevated 0.2 m above mean sea level. The total area of the catchments draining to the lake system is 660 km^2.

5.2 The Tidal Inlet

The inlet to Tuggerah Lake takes the form of a delta extending 2.6 km from the beach line to the lake. It may be considered in two regions. The downstream 800 m from the beach to the road bridge is the entrance area and consists of unstabilised and frequently mobile sand shoals with one or more tidal channels. The upstream 1.8 km from the bridge to the lake proper takes the form of a fan delta, the sand of which is largely stabilised by weed growth.

The delta margin at the lake is not as stabilised and in an aerial photograph shows signs of bed features formed by wave action. This is particularly so on the north west sector which faces the longer reaches of the lake. The western sector shows many fewer wave derived features and those that are present have an orientation consistent with waves approaching from the west-north-west.

With the exception of the two islands, the upstream delta area is below lake water level. It is dissected by a number of natural channels and two narrow dredged channels, one to the north-east and one to the south-west.

5.3 The Entrance

5.3.1 General

The Entrance proper from the bridge to the beach is a region 800 m long with a maximum width of 350 m. It consists of mobile entrance shoals with one principal channel and as many as three minor channels. On the southern shore, the mobile area extends some 200 m upstream of the bridge while on the northern shore it stops short of the bridge.

The downstream section of the northern shoreline comprises the sand spit known as Karagi Point.

5.3.2 Entrance Hydraulics

The entrance to Tuggerah Lakes acts as a channel with a tidal range ratio approaching zero. The tidal range in the lake is of the order of 20 mm based on tidal prism/surface area calculations. Two field measurements were made on 8th October 1975 and 26th May 1976.

Figures 7 and 8 show the tidal gradients obtained in the 1975 measurement. For an ocean tidal range of 1.6 m, the ebb and flood tidal prisms were 10^6 m^3 with a channel cross sectional area of of 30.7 m^2 at MSL.

For operational reasons, the 1976 measurement was made at a location upstream of the entrance channel and hence did not yield a measure of the channel cross section. The ebb and flood prisms were markedly different with an average value around 10^6 m^3.

The 1975 data showed the short peaked flood discharge and extended ebb discharge curves typical of shallow entrances and small tidal range ratios. The super elevation of the lake water surface was 210 mm above AHD.

5.3.3 Entrance Channel Stability

Tidal Prism and Cross Sectional Area

With only one data point it was not possible to establish an O'Brien type relationship between tidal prism and channel cross section. However, based on the assumption of an exponent of unity, this single observation yielded:

$$A = 9 \times 10^{-6} P \text{ (in imperial units)}$$

The co-efficient of 9×10^{-6} was larger than but of the same order of magnitude as the 5.37×10^{-6} given by Jarrett for semi-diurnal tides on the Atlantic Coast for entrances with none or one training wall.

Peak Tidal Velocities

The 1975 data gave peak tidal velocities of 2 m/s on both the ebb and flood tides. Again there was insufficient data to observe variations of the type suggested by Escoffier. However, given that he suggests a critical velocity of the order of 3 ft/s (0.9 m/s), the data may indicate that in 1975, the channel was attempting to scour towards a stable cross section which would be of the order of twice that measured. A far more likely explanation is that although the velocities were above the critical, a large sediment feed in the entrance prohibited the full development of the section.

Littoral Drift Effect

The work of Bruun related the stability of entrances to the ratio between tidal prism (P) and annual littoral drift (M). This was done by computing the P/M ratio for a number of entrances and comparing the result with the observed stability and shoal development of each entrance.

On this basis, Tuggerah would be classed as an entrance which forms shoals and is subject to closure and would be expected to have a P/M ratio of less than 100. Since it has a measured tidal prism of 10^6 m^3, Tuggerah would therefore appear to be subject to a gross littoral drift of 10,000 m^3/yr or more.

Limitations in Per Bruun's criteria must be considered when applying it to determine the capacity of sediment interceptors. These are:

* The ratio of P/M, being "less than 100" is unbounded on its lower side. For small inlets such as at Tuggerah Lakes entrance, the data in Bruun (Ref 12) suggests a ratio of about 30 may be appropriate; ie, M = 30,000 m^3 pa.

* The average annual gross littoral drift may impinge upon an inlet in slugs, rather than uniformly through time, requiring a sand pumping capacity greater than the average sand supply.

* The criteria does not explicitly incorporate onshore-offshore exchange of sand. After flood scour of the entrance shoals, heavy onshore supply occurs at a rate which may exceed the alongshore rate for periods lasting some months.

A comparison can be made between the value of at least 10,000 m^3/yr obtained above and the very approximate measurement of sand in movement during the flood tide made on 3rd February 1987. The relevant data used was:

* effective channel width	20 m
* peak tidal velocity	1.2 m/s
* bed sediment transport	0.015 m^3/m/min
* suspended sediment concentration	1 gm/l
* suspended sediment layer thickness	0.25 m
* effective duration of movement	2 hours

The calculated sediment transport volumes were then:

* bed load	36 m^3 per tidal cycle
* suspended load	43 m^3 per tidal cycle

Hence a total of 79 m^3 of sediment per tide or approximately 60,000 m^3/yr is in motion in the entrance channel. Hence, the chosen range of 10,000 to 60,000 m^3/yr is likely to be a reasonable indication of the supply to the entrance throat. It is not necessary to intercept all this sediment to maintain a permanent opening. The jet pumps have been sized to each pump 50 t/hr. This would allow adequate provision to cope with higher than average sand infeed.

5.4 The History of the Entrance

5.4.1 Aerial Photographic History

Fifteen aerial photographs of The Entrance area were obtained covering the period from 1954 to 1979. These are sketched in Figures 9A to 9L. Qualitative assessments were made of some of the characteristics of the area between the beach line and the road bridge.

The width of the channel at the throat area was scaled and its location in the north-south direction was noted. Also noted was the direction taken by the beach runout channel. An estimate was made of the degree of shoaling of the entrance in terms of the percentage of the area visible in the photographs as shoal. The results of this assessment are summarised in the following table.

Date	Channel Width (m)	Location	Runout Channel	Percent Shoaled
-03-54	20	S	NE	70
23-09-61	20	S	E	80
18-08-65	40	S	NNE	60
8-03-66	12	S	NE	70
30-07-67	50	S	ENE	70
6-07-69	30	S	NNE	60
16-05-70	30	S	NE	70
22-09-71	45	S	E	90
1-07-72	30	S	NE	90
6-02-73	30	S	E	100
19-06-74	40	almost S	NE	80
19-08-76	50	almost S	NNE	80
19-11-76	30	almost S	NE	80
19-08-77	80	S	ENE	70
7-07-79	30	S	NE	70

In addition, on more than half of the photographs, the beach line for the southern quarter of Karagi Point was displaced to the west by some 50 m while the remainder of the point maintained its alignment with Tuggerah Beach.

From this it was concluded that channel widths at the throat varied from 10 to 80 m with the majority being in the range of 20 to 45 m. Two were 50 m wide and one 80 m. The entrance was generally heavily shoaled. The defined channel was always located at or close to the southern limit of the entrance area and the beach runout channel was predominately directed to the northeast.

The preferred geometry of the entrance with the channel located as far south as possible would indicate sediment feed from the beach to the north building Karagi Point. The north-easterly direction of the beach runout channel would be due not to sediment processes but to the rock shelf. This would also explain the western displacement of the tip of Karagi Point seen on many of the photographs.

In all the photographs examined the entrance was open to tidal flows. The Inderdepartmental Committee Report (Ref 15) found that on nine occasions in the 100 years prior to 1979, the channel had completely closed up and for some tidal cycles there was no flow of sea water into the lake system. Since 1979 to the time of writing, the entrance has closed three times:

* For a period up to October 1985 when it was opened by bulldozers. The entrance remained open for six months then deteriorated over two months, closing in June 1986.
* From June 1986 to the 11th August 1986. Several attempts were made in early July and early August to open the entrance but it quickly closed since there was insufficient lake elevation to create the necessary scour to overcome beach accretion across the cut channel. A successful cut made on the 11th August is described in detail in Section 5.4.2 below.
* In April 1987.

5.4.2 Opening Event of 1986

After the successful cut on the 11th August 1986, the entrance remained open until the 5th April 1987 when it again closed. This event was studied for the information it provided on the scouring of the entrance shoal area and the subsequent re-establishment of a tidally dominated regime.

The basis for the study was a set of small format aerial photographs taken by the Department each covering the area from the beach to the road bridge. In addition, lake water levels were available via an automatic water level recorder at Killarney Vale.

11th Aug 86: Lake Level 0.93 m AHD: Fig 10A

The entrance was closed and the beach berm well developed. The high Lake Level prevented the clear definition of the entrance shoals. The pilot cut was visible.

18th Aug 86: Lake Level 0.40 m AHD: Fig 10B

Flow from the lake had been continuing for the six days since the evening of the 11th August. A 300 m wide channel had developed at the beach outlet and the bed of this channel extended offshore in the form of attached shoals in the near offshore. The channel at the beach outlet appeared to be less than 1 m deep.

A single thread channel developed between Karagi Point and the road bridge. The precursor to this may have been present but not visible on the photographs of the 11th August. The channel depth was estimated at 2 to 3 m and the major shoal to the south and west of this channel was unaffected. The rate of change of Lake level was constant until the 16th August and corresponded to a nett outflow of 90 m^3/sec.

4th Sept 86: Lake Level 0.14 m AHD: Fig 10C

By this time wave action had caused the offshore shoals to rise above the bed level of the beach outlet channel and they were crossed by two separate channels. Water depth over the shoals was virtually zero. The main channel had its greatest depth in the constricted region adjacent to Karagi Point. There was growth of the spit at Karagi, most probably as the result of onshore sand movement rather than from littoral drift.

11th Sept 86: Lake Level 0.15 m AHD: Fig 10D

The principal change was to the offshore shoals which, while still detached from the beach, had adjusted to give three outlet channels. The middle of these was the largest.

24th Sept 86: Lake Level 0.18 m AHD: Fig 10E

The shoals were beginning to re-attach to the beach at the southern end leaving the northern outlet channel as the most distinct one remaining. The entrance channel was developing a flood tide shoal near Karagi Spit on which sand growth continued.

14th Oct 86: Lake Level 0.17 m AHD: Fig 10F

The beach outlet area was continuing to change with the main channel in the near offshore now being the southerly one. Karagi Spit continued to develop and the nearby flood shoal reduced in importance. The main channel from the road bridge to Karagi Spit remained well defined.

3rd Nov 86: Lake Level 0.22 m AHD: Fig 10G

By this time, the main entrance channel had extended through the shoals in the offshore region. It was once again a well defined single thread system with peripheral flood tide shoals. The shoals in the main offshore region were virtually re-attached to the beach.

26th Nov 86: Lake Level 0.22 m AHD: Fig 10H

A single thread main channel was fully developed, trending to the northeast in the offshore region. Coloration of the water made it appear wider than before, although it probably was not. A distinct ebb tide shoal had formed just downstream of the road bridge and there was a major development of Karagi Spit.

12th Dec 86: Lake Level 0.12 m AHD: Fig 10I

The main channel by this time flowed directly east of the near offshore and minor adjustments had occurred to it at the road bridge. These resulted in an ebb tide shoal intruding into the main channel.

14th Jan 87: Lake Level 0.09 m AHD: Fig 10J

The main channel had developed significant ebb and flood tide shoals, losing its distinctive character in the area between the road bridge and Karagi Spit. The channel at the beach outlet had been constricted by a seaward spit to the north and accretion of the southern bank.

30th Jan 87: Lake Level 0.18 m AHD: Fig 10K

The main channel remained well defined at the beach but between there and the road bridge it had widened and shoaled significantly.

12th Feb 87: Lake Level 0.16 m AHD: Fig 10L

The channel at the beach outlet had narrowed and was directed slightly more to the north. Shoaling was extensive upstream of this and no clear main channel remained.

18th Mar 87: Lake level 0.25 m AHD: Fig 10M

A possible further narrowing of the channel at the beach outlet occurred.

6th Apr 87: Lake level 0.26 m AHD: Fig 10N

The outlet channel at the beach line had closed after apparently having narrowed prior to this closing.

Summation

From the time of opening of the channel with the lake level at 0.93 m AHD, the dynamics of the entrance could be divided into a number of reasonably distinct phases.

The first phase lasted from the 11th to the 16th August and during this time the nett outflow was of the order of 90 m^3/s and the flow would have been predominantly unidirectional. This scoured a well defined channel as far as the beach line where it became a 300 m wide shallow exit channel.

This was followed by a period of development of multiple offshore channels and their decay until by 3rd November, a single well defined beach outlet channel had stabilised. This was at the site of the original cut suggesting that the lateral extension was shallower, although no clear evidence of this can be seen from the photography. Alternatively, the location may have been determined by lateral constraints in the throat area.

From the 12th December the entrance channel began to degenerate from a well defined single thread to a series of ebb and flood tide shoals. The indications are that this was the result of sand carried in from the entrance area particularly given the rapid development of Kangi Spit in the proceeding period.

Finally, the outlet channel across the beach began to narrow and was closed at the beach line. The occurrence of the closure coincided with three days of moderate wave activity.

To the extent that conclusions on sediment dynamics can be drawn from a single event, it seems that the following is likely:

* A flood event will significantly widen the beach outlet channel.

* A well defined single thread channel will be established through the entrance shoals.

* Sediment influx from the beach and near offshore will build ebb and flood tide entrance shoals and these will decrease the efficiency of the entrance channel.

* Further sediment infeed will narrow the outlet across the beach until a single event will cause it to close.

* The effect of jet pumping on these mechanisms would be two fold. By intercepting sediment transport into the entrance channel, it would inhibit the development of the shoals and would also remove sand which would otherwise result in a closure event.

5.5 Coastal Processes Involved

5.5.1 Definition of the System

Figure 11 defines the possible components of the coastal processes at the Entrance. These involve the movement of sand between various storage areas. The landward boundary of the compartment is set at the road bridge since there is no evidence of other than very minor sand movement past this point. The seaward boundary is the sand shoals in the near offshore region.

The beach boundary conditions are the longshore sediment fluxes q_1 and q_2 together with the beach sand stores s_1 and s_2. The internal sand store s_4 is taken to comprise the sand stored in the shoals and the sand above water level to the north (Karagi Point) and the south of the entrance channel. The two channel sediment fluxes (q_3, q_4 - flood and q_5, q_6 - ebb) were specified for convenience when discussing the effect of the jet pumping operation.

5.5.2 Flood Conditions

The mode of sediment transport which occurs during a flood is clearly different from that which takes place under tidal conditions.

The first or scour phase lasts several days, the actual duration depending on the magnitude of the flood. During this time, the lake acts as a large detention basin, is super elevated and the flow in the entrance is in the ebb tide direction only. The principal sediment fluxes are q_5 and q_6 transporting sand from store s_4 to store s_3. These losses are broadly distributed both in the shoals and in the areas to the north and south of the channel. The result is to leave a broad shallow beach runout channel, reduced internal shoals and a more clearly defined entrance channel through the shoal area.

After the passage of a flood a second phase ensues where there is a relatively rapid re-adjustment of the sand stores. Under wave and tidal action, the sediment flux q_3 becomes dominant and q_4 is also significant. At this time there may be minor contributions from q_1 and to a lesser extent from q_2. It is considered that q_2 and the variations in s_2 are never significant in the coastal process. The effect of this phase is to re-establish the downstream part of store s_4 and some of the upstream part. While this phase is relatively rapid, it is still the dominant process over a period of weeks to months.

5.5.3 Tidal Conditions

In the absence of floods and post flood adjustment, the sediment movements proceed at a much diminished rate. Flux q_2 is virtually zero and q_1 is a function of the height and direction of the breaking waves, being also small since the site is adjacent to a headland. The tidally driven sediment fluxes q_3 and q_6 are of greater magnitude and hence:

$$q_3 - q_6 = q_1 - \Delta s_3$$

where $$\Delta s_3 = s_{3_{t_1}} - s_{3_{t_0}}$$

This difference between flood and ebb seaward transport is responsible for building the shoals in the entrance area.

5.5.4 Jet Pumping of Sand

The installation of a jet pump in the throat area of the entrance channel would generate a crater which would intercept both q_3 and q_6 provided it reliably performed its pumping function. At the beginning of operations, q_4 and q_5 would be unaffected but as channel development proceeded, both would reduce, q_4 more rapidly than q_5.

The beneficial effect of the jet pumps would be that they reduce nett landward movement of sediment at the throat and hence prevent a reduction in channel area. Being located near the seaward end of the channel, they would be in a position to cope with episodic increases in q_3 which might otherwise cause the entrance to close.

The deposition of sand to the northern beach would increase store s_1 and cause a local seaward movement of the beach line. The long term effect of this would be a slow nett sediment feed northward to Tuggerah Beach.

5.6 Present Maintenance Methods

The present maintenance procedures are not comparable in their effect with a jet pumping system in that they do not produce a permanently open entrance.

The technique used is one of opening a pilot cut through the beach when the entrance is closed and the water level in the lake is of the order of 1 m above AHD. The resulting flow then serves to develop a channel extending from the beach line a considerable distance upstream into the entrance shoals.

The effect depends on the flow existing for a sufficient length of time for seaward sediment transport to develop this channel. The duration of the flow is directly related to the super elevation of the lake at the time of making the cut. Experience has shown that minor super elevations (those somewhat less than 1 m) do not produce the desired result and the entrance so made often closes soon afterwards.

A further drawback to this technique is that a lake at RL 1 m AHD would, for a given inflow hydrograph, result in peak flood levels higher than those which would result if the lake were not elevated and the entrance were open.

The costs involved, being the plant charge for a bulldozer for about one day, are insignificant compared with the costs associated with a jet pump installation.

An alternative approach could involve more intensive excavation using an hydraulic excavator to create a deeper cut. During dry periods the plant could be kept on site to maintain a channel large enough to provide tidal exchange of the entrance area. During wet periods the plant could be used to prevent water levels in the lake from reaching nuisance levels.

In some years it could be necessary to work the equipment for up to 100 days while in normal years (during which the entrance maintains its own tidal opening) no excavation would be required. Allowing for an average of 30 days /yr at $1,000 per day, the average annual cost would be around $30,000. This may provide an optimal solution, overcoming the limitations of occasional bulldozing without incurring the higher cost of jet pumps.

6. CONCEPTUAL JET PUMP SCHEMES FOR TUGGERAH LAKES

6.1 The Kirrawee Proposal (Figure 13)

Wyong Shire Council had arranged for Kirrawee Engineering Services Pty Ltd (KES) to develop and cost a proposal using jet pumps to maintain a permanent tidal opening to the lakes' entrance. KES who are the Australian agents for the United Kingdom "Genflo" jet pumps suggested three jet pumps equally spaced along a 100 m long entrance channel driven from a single motive water pumpset located on the northern shore. The discharge lines from the pumps were planned to deliver to a point about 150 m north of the entrance channel. The proposal also included a development/maintenance dredge using a jet pump which drew its motive water from the shore based pump set.

The principal drawback to the proposal arises from some assumptions which were made before the field tests reported herein were undertaken. The first of these was that the jet pumps would develop craters with side slopes of 15 degrees and in so doing gain access to sand along the full length of the 100 m channel. Field tests undertaken in the tidal channels of both Narrabeen Lagoon and Tuggerah Lakes showed that side slopes would be of the order of 45 degrees which would mean that the three pumps would create three isolated craters.

At the depth of immersion proposed these craters would have an effective lateral width of influence of the order of 10 m. When this width is compared with the widths of naturally occurring Tuggerah channels it is clear that a considerable amount of sand could be transported past all three pumps to the area upstream. While the proposal did not presume to capture most of this sand but only to maintain a channel, the fact of this sand transport would mean that the development of internal shoals would proceed at a pace only slightly less than at present.

The second assumption which was implicit in the proposal was that the creation of jet pump craters would locate the entrance channel. There is no evidence to support this assumption and a very real risk would exist of some or all of the jet pumps being outflanked at which stage they would become ineffective and only a dredging operation could again establish the channel in the specified location.

6.2 Possible Scheme Arrangement

6.2.1 Basic Approach

The fundamental concept of a possibly workable scheme is to intercept sediment as it is about to be transported into the entrance channel. This interception would occur in two or three craters created by jet pumps set below the bed of the channel. The pumps would be operated intermittently for a sufficient length of time to always have the craters at full width even if not at full depth.

Since the jet pumps and the craters they produce have no appreciable effect on the plan form stability of the channel, this needs to be achieved by some other method. The limited available evidence points to a tendency for the channel to migrate to the south. A low restraining wall as discussed in Section 4.5 should prevent this migration.

Sand removed from the channel should be deposited on the beach to the north of the entrance. Failure to do so could cause beach recession.

6.2.2 The Crater Area

This comprises the jet pumps and the craters they create together with the scour protection to the southern bank of the channel. A width of 40 m has been adopted for the crater system. This is based on the assessment of twelve aerial photographs from 1954 to 1979 (Section 5.4.1). The widths of the principal channel in these photographs varied from 20 m to 80 m with most values being 45 m or less. Based on considerations of Section 5.3.3, craters of 40 m combined width would capture sufficient sediment to provide for a permanent tidal opening.

As illustrated in Figure 12, the number of jet pumps needed to achieve a 40 m width depends on the depth at which they can be set, assuming a crater side slope of 1 in 1 or slightly flatter. If it is taken that the 40 m width is needed at RL 0.0 m AHD (this may result in a small space between the two cones) then two pumps would need to be set at RL -10 m AHD and three pumps at RL -6.7 m AHD.

Some rock levels in the general area of the craters are reported in the Inter Departmental Committee Report of the Tuggerah Lakes (Ref 15). These range from RL -9.4 m to RL -6.0 m AHD which suggests that three pumps would be required if rock excavation is to be avoided. However, for the detailed design of the system, better data on rock levels would be required bearing in mind that the pumps could be located in local depressions in the rock and still be effective.

A depth of 10 m is much preferred since the greater crater width in the direction of tidal flow would be more effective in trapping suspended sand.

For estimating purposes, two jet pumps located at RL -10 m AHD have been assumed. A contingency has been provided for rock excavation between RL -5 m and RL -10 m AHD.

Figure 12 shows the jet pumps installed vertically in the craters with pumps and pipes supported by piles. This would require structural checking at the detailed design stage. The pumps would be mounted on carriages that would be raised and lowered on guides attached to the piles using temporarily installed lifting gear.

The motive water line and the discharge line would be laid at RL -5 m AHD to achieve protection against flood scour. The pipelines would be laid carefully to a horizontal grade to minimise the chance of sand blockages if the pumps were shut down while the pipelines were carrying a slurry of high pulp density.

The deep immersion would make routine cleaning of the pump screens more difficult and this would require the service of divers. For costing purposes, it is assumed that screen cleaning would be required six times each year.

The restraining wall would extend from high water mark (determined as an average location under full beach accretion conditions, ie, not after storm beach erosion) westwards to the fixed shore. The length of wall required would depend on the location of the craters, to be determined after detailed rock level investigation.

For estimating purposes, a 75 m long sheet pile wall would be located south of the craters. The wall would be driven to rock or 8 m and have a stiff reinforcing cap of steel beam or precast concrete units. The seaward head of the wall would be strengthened to resist storm attack, either with raked piles or heavy rock/concrete armour units. No quarry run material or small armour would be used because of the problems that would occur if storm waves deposited such material into the craters.

The sheet pile wall would overlap a "Sand Sausage" wall some 2 to 3 m high from RL -3 m AHD to RL 0 to -1 m AHD, depending on final design. This "Sand Sausage" is a resilient liner filled with sand. For most of the time it would be largely buried over its 140 m length. The Sand Sausage could run into a vegetated formed dune adjacent to the hard shore.

6.2.3 The Discharge Point

In the operation of the scheme one prime consideration will be to avoid any erosive impact on the beach to the north of the entrance channel. This will require that sand pumped from the craters is returned to the beach system. This will have the effect in time of increasing the sand in store on the beach and reducing the sand in store in the entrance shoals while maintaining the wave induced sediment flux to the entrance at approximately its present value.

The location of the discharge point will depend on the rates at which the deposited sand is moved to the south and to the north. Should the deposition of sand induce a slow northerly movement as well as the expected southerly movement, the discharge point could be relatively close to the entrance. On the other hand, discharge near the entrance may result in increased pump operation times. In this case it may be more cost effective to invest the additional capital for a booster pump and a longer pipeline. The need for a booster slurry pump could be assessed after experience was gained operating the scheme in its more basic form.

6.2.4 The Pump House

The pump house would accommodate the motive water pump supplying clear water to power the jet pump and if required, the booster pump to supply a longer discharge pipeline.

It would need to be located in an area where sand free water is obtainable. This can be on either the southern or northern shore in an area where its foundations will not be subject to erosion during floods or storm waves.

6.3 Layout Options

6.3.1 Introduction

The variations considered amount to whether the pumphouse is located on the southern or northern shore and whether a near or a far discharge point is used. All options assume two Genflo 50 mm jet pumps each rated at 50 tonnes/hour of sand. The jet pumps would be set at RL -10 m AHD.

6.3.2 Option 1 (Figure 14)

The pumphouse is located on the southern foreshore of the entrance area providing motive water though a 200 m long, 150 mm diameter pipeline to the jet pumps. Near the jet pumps, this line will branch to provide motive water to both pumps at the same time. Although this requires a larger motive water pumpset, it means that only one pipeline is required and that there is no valving. Besides a saving of the cost of valves, this strategy reduces operator costs and the possible costs associated with mechanical malfunction of valves in the difficult maintenance area near the jet pumps.

The discharge from the jet pumps will be via a common 150 mm polypipe to a location near the beach berm 200 m north of the northern bank of the entrance channel. Discharge near the beach berm will enable the creation of a minor store of sand and thereafter, the slope of the sand spoil (of the order of 1 in 20) will generate a feed of sand to the wave zone while increasing the sand in store from the berm to the wave zone.

This is a compromise between a too rapid return of sand to the active zone and the creation of a large sand deposit which would need to be spread mechanically. The distance of 200 m was chosen as a result of operator opinions as to a practical discharge distance for a jet pump to deliver sand slurry.

Locating the pump house on the southern as opposed to the northern shore has the advantage of reducing the length of the motive water pipeline from 300 m to 200 m. Against this must be set the fact that excavation and possible maintenance of a clear water channel will probably be required.

6.3.3 Option 2 (Figure 15)

This option is an extension of Option 1 to enable the deposition of sand 600 m to the north of the jet pumps. While the extension of the delivery line to 600 m is beyond what is normally considered the economical limit for the discharge from a jet pump, the location of the pump house on the southern shore makes the provision of a booster pump impractical unless a second pump house were built on the northern shore to accommodate it with the additional capital cost and operational complication involved.

6.3.4 Option 3 (Figure 16)

In this layout the pump house would be located on the northern shore drawing water from an adjacent existing channel. The motive water pipeline would be 300 m long and the discharge pipeline would be placed with it in the same trench, branching near the pump station to the beach disposal site, a total length of 350 m.

Two advantages of this arrangement are significant savings in pipelaying costs and the ability to regularly backflush the discharge line before shutdown. This facility would permit the use of a fixed time duty cycle even if the sand in the craters was not totally excavated on each cycle. Hence, by occasional adjustment of the duty cycle it could be arranged that the required quantity of sand were pumped and that no power was wasted pumping water only through the jet pumps.

The longer pipelines experience slightly higher pump heads and require a slightly more expensive motive water pump.

The other main advantage is that this option can be readily upgraded to Option 4.

6.3.5 Option 4 (Figure 17)

This option is an extension on Option 3 to permit discharge of sand at a point approximately 1 km north of the jetpumps. It involves provision of a centrifugal slurry pump in the pumphouse to boost the pressure to the 1000 m of discharge line. As in all the options, polypipe will be used for all pipework outside the pump house.

This layout can be implemented as an upgrading of Option 3 should the need for remote disposal of sand become evident after an initial period of disposal at a point 300 m to the north of the jetpumps. In this case, the modification would involve the addition of the slurry pump, modification of pumphouse pipework and the longer discharge pipeline.

6.4 Mobile Barge Mounted Jet Pump System

Some of the work in the initial stages of construction may require a dredge to establish new channels. Once the sytem is operating, there may be occassional need to dredge shoals which form as a result of sand bypassing the entrance sand trap or to pump empty the sand crater of a non-operational blocked jet pump to allow its removal and re-installation. A barge mounted jet pump could perform these tasks.

The options on motive water supply are either to draw on the motive water pump associated with the fixed jet pumps or to provide diesel driven pumpset on the barge. The advantages of the latter approach are that there would be two self contained systems with a reduction in complexity and potential operational problems, and an unrestricted range of operation of the dredge. These seem to outweigh the disadvantages of additional weight of the barge system and the higher capital cost.

The dredge would then comprise:

* barge
* diesel powered motive water pumpset
* jet pump on a rotatable pipe system
* winch for raising jet pump
* winches for barge manoeuvering
* 200 m of floating delivery line.

6.5 Option Costs

6.5.1 Capital Costs

Pre design costing of the options is given in Appendix B.

The capital cost estimates were based on a scheme employing two Genflo Sandbug (50 mm) jet pumps with the associated plant and pipelines. Option 4 incurred an additional cost for a booster slurry pump. Table B1 shows the capital cost components for each option.

In summary they are:

Option 1:	**\$795,000**
Option 2:	**\$833,000**
Option 3:	**\$821,000**
Option 4:	**\$1,005,000**

6.5.2 Operating Costs

The operating costs will depend on the number of hours operation per year. This will be a function of the annual quantity of sand moved which could range from 10,000 to 60,000 m^3/yr. Estimates were based on 1000 hrs/yr to pump 60000 m^3. Allowance has been made for a part time operator, five pump screen clearance operations and an item for spreading the discharged sand on the beach. As shown in Table B2 these totalled:

Option 1:	\$38,000 + (\$6000 contingency)
Option 2:	\$38,000 + (\$6000 contingency)
Option 3:	\$38,000 + (\$6000 contingency)
Option 4:	\$50,000 + (\$12,000 contingency)

6.5.3 Maintenance Costs

It is considered that an annual system maintenance will be required. This will involve the removal and overhaul of the jet pumps together with an overhaul of the motive water pump and, for Option 4, the slurry pump. The annual maintenance costs so determined from Table B3 are:

Option 1:	\$10,000
Option 2:	\$10,000
Option 3:	\$10,000
Option 4:	\$15,000

6.5.4 Equivalent Annual Costs

By transmuting capital costs to an equivalent series of annual costs and adding to these the operating costs (including contingencies) and maintenance costs, the equivalent total annual costs become:

Option 1:	\$165,000
Option 2:	\$172,000
Option 3:	\$168,000
Option 4:	\$218,000

6.6 Discussion

The capital costs of the four options and the total annual running costs show that for options 1, 2 and 3, the cost variations are so small as to make selection on the basis of cost spurious.

The similar costs reflect the lower pipelaying costs of Option 3 resulting from the shared trench for motive water and discharge piplines to compensate for the increased length of those lines. Option 3 discharges sand a further 100 m away from the entrance than Option 1. While Option 2 pumps even further, the pumping is less efficient and the discharge line could be more prone to blockage.

The clear water intake for Options 1 and 2 is likely to need greater maintenance in terms of dredging shoals than for Options 3 or 4 for which a lesser amount of dredging of the intake area is anticipated.

The southern location of the pumphouse in Options 1 and 2 may result in greater noise nuisance than would the northern location of Options 3 and 4.

Option 3 has significant advantages in the facility to use the motive water pump for backflushing the discharge line and in its ease of upgrading to Option 4 if required. The pumphouse size and pipeline strength should be designed to be compatible with Option 4 arrangements.

There is no need to contemplate the construction of Option 4 at the outset.

Based on all of the above considerations Option 3 is superior. Such a scheme would have a capital cost of say $820,000, which when transmuted to an equivalent annual cost and added to operating and maintenance costs results in a total annual cost of $168,000.

7. CONCLUSIONS

(1) The overall stability of a tidal entrance may be determined by use of the ratio of tidal prism to gross sediment feed from the ocean side. Improvement of entrance stability can be achieved by the reduction of gross sediment feed. This is often achieved, on a finite time scale, by the construction of entrance breakwaters. A commonly used alternative method involves the removal of some sediment feed by dredging. The two methods are also often combined in the maintenance of large commercial ports.

(2) A fixed jet pump system could theoretically remove sufficient of the sediment feed to Tuggerah Lakes entrance to provide a permanent tidal opening. However, no record has been found of the use of jet pumps in the manner proposed at Tuggerah Lakes.

In particular the US Army Corps of Engineers (which is nationally responsible for estuary entrance improvements) has been reluctant to install any such scheme even though it has been testing jet pumps within existing trained entrances for more than a decade. Despite this reluctance, officers of the Corps are of the opinion that such an application should be effective if carefully designed and regularly maintained.

(3) Jet pumps have the advantage of requiring no electrical power and having no moving parts. They are therefore suitable for installation in locations where conventional dredges would have difficulty working. A penalty incurred by the nature of the design is a relatively low efficiency, but this is not critical if discharge distances are small.

(4) Jet pumps intercept sediment by forming a crater trap. A present day Australian example of such trapping exists adjacent to the Nerang River entrance in Queensland. Such craters do not initiate sand movement in areas remote from their immediate sphere of influence and hence need to be located in tidal inlets near the downstream end of the entrance channel.

Within the context of application to Tuggerah Lakes, they should be located far enough into the entrance to be away from the beach swash zone where high alongshore and nearshore circulatory transport occurs. The jet pumps should also be far enough into the entrance to be at its 'throat' (natural point of minimum width) to limit sand bypasing of the craters via the channel margins.

(5) There is no obvious mechanism by which jet pump craters would restrain the movement of a tidal entrance alongshore. Hence, to prevent this undesired channel migration at Tuggerah Lakes entrance, it would be necessary to construct a restraining wall on the southern side of the craters, extending landwards to the rock back-shore.

(6) The entrance at Tuggerah Lakes is typical of a tidal entrance with a small tidal range ratio. It is subject to a sediment feed estimated at between 30,000 and 60,000 m^3/year. This sand infeed would not be uniform through time. During storms, when wave energy is high and ocean levels are elevated, slugs of infeed would occur. Similarly, during periods of north-easterly swell, supply to the entrance will exceed the annual daily average. Therefore, the jet pumps would need a greater capacity than the average infeed. Two 50 mm jet pumps, each rated at 50 tonnes/hour, would appear adequate to address this problem.

(7) The initial cost of establishing a jet pump system at the Tuggerah Lakes entrance would be $820,000. Annual running costs and average annual maintenance costs are estimated at $54,000 on a similar preliminary basis.

The capital and operating costs of a jet pump installation are significantly higher than the cost of the present entrance management approach of ad hoc bulldozing. In terms of an equivalent annual cost, the jet pump costs are estimated at $160,000 to $170,000 per annum.

(8) The major uncertainty remaining about the viability of a jet pumping scheme is the susceptibility of the pumps to envelopment by material bigger than a 50 mm sphere (for the pump types under consideration). The craters that trap moving sand will also collect neutrally buoyant materials such as plastic, cans and kelp. These could pack around the pumpscreens and act as a filter to prevent sand feed to the pumps.

Problems of this nature have been experienced in the USA where it has been found that firstly, the sand that fills the crater of the inoperative pump must be excavated, and then the "unpumpables" have to be removed by divers. Such operations are difficult in a hydraulically energetic zone and would accordingly be expensive. Each clearance operation has been estimated at $3,000. Such operations may be required many times per annum and hence the ultimate efficiency of the scheme remains open to question.

(9) The function of jet pumps at Tuggerah Lakes could be performed by standard mobile plant which does not suffer the operational uncertainties of jet pumps and could thereby be guaranteed to perform its designed function.

(10) In view of performance problems experienced by jet pumps in Australia and the USA, successful function at Tuggerah could not be guaranteed. However, if operational problems were surmounted, such a use of jet pumps would provide a valuable tool for the management of estuary entrances.

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Mr G C Williams, Engineer, Estuary Management, PWD

Mr P Stone, Senior Consultant, CMC

Mr C A Miller, Managing Director, CMC

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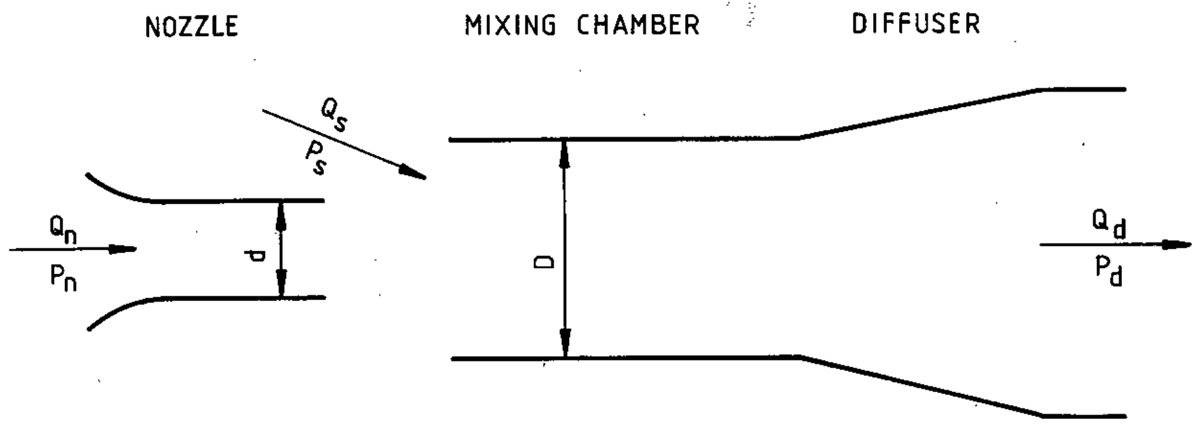
Mr C Panciuk, Marine Directorate, Public Works, Canada

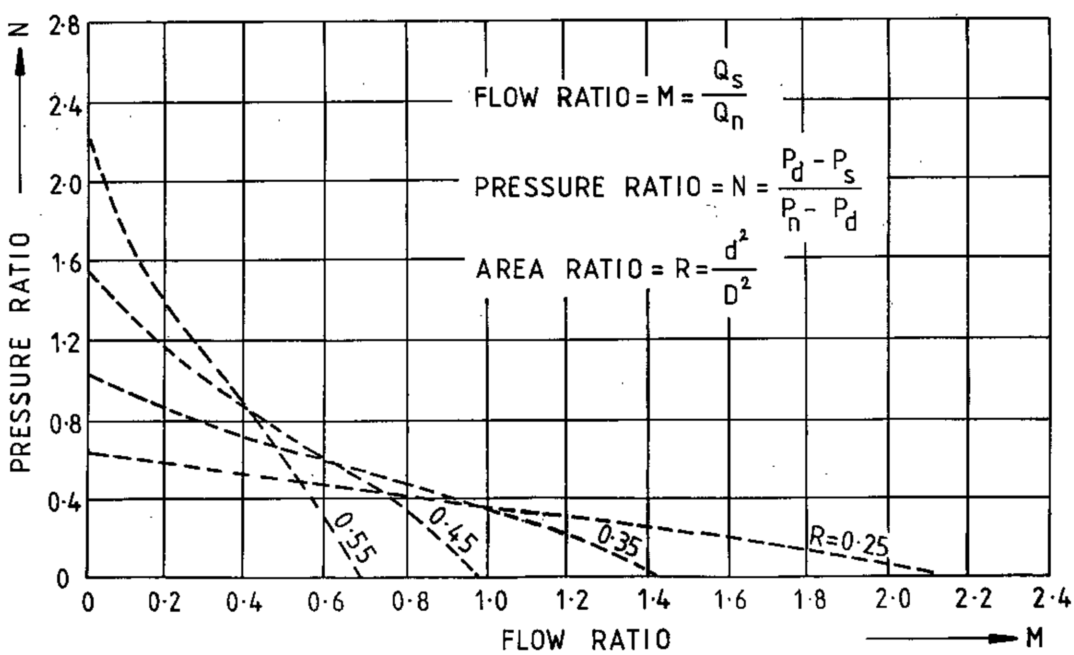
Mr M B Gerstle, Special Projects Officer, Warringah Shire Council

Messrs D Robinson and H McDonald, Department of Harbours and Marine, Queensland

Mr P Venton, Slurry Systems Pty Ltd

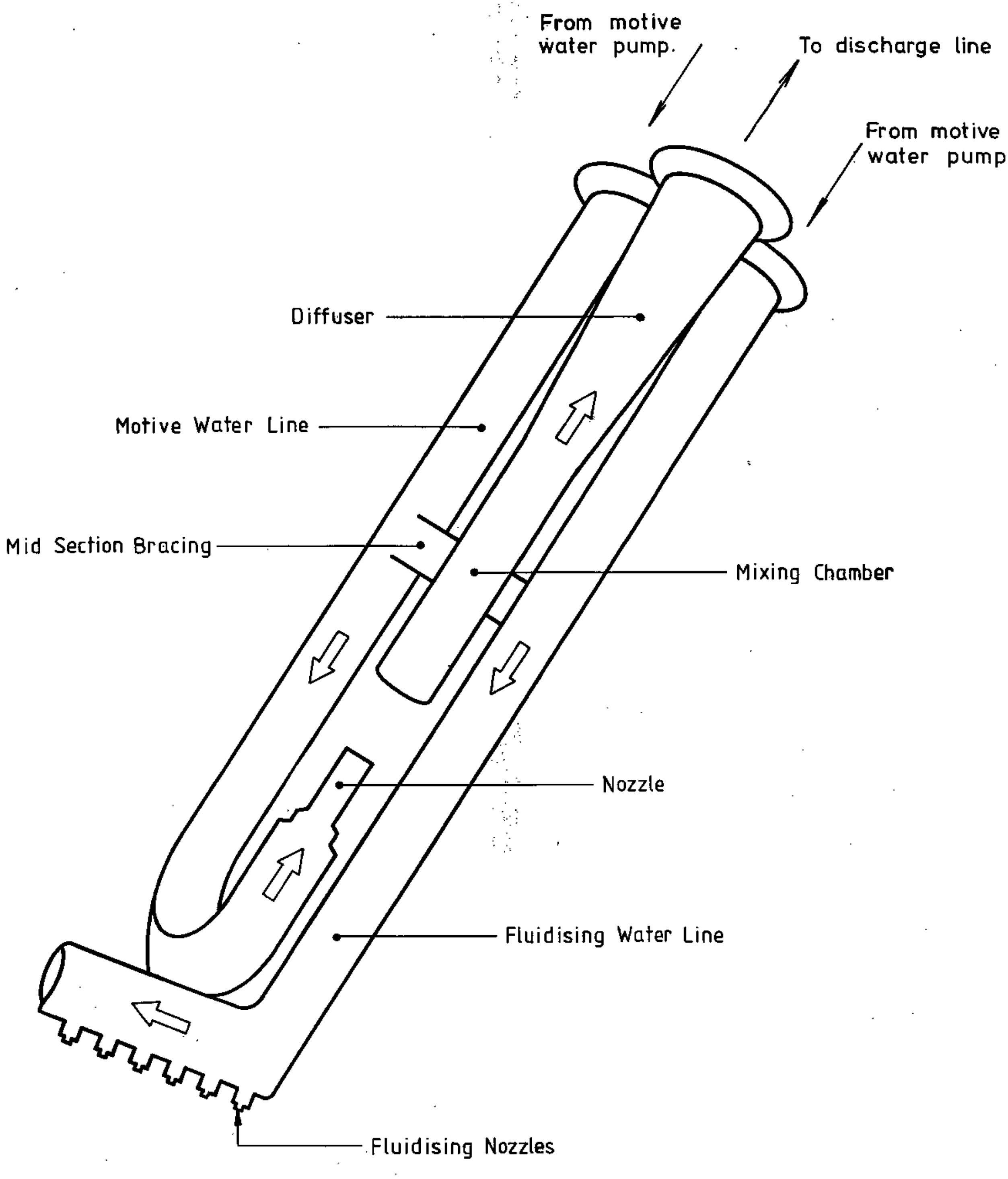


TYPICAL JET PUMP SCHEMATIC

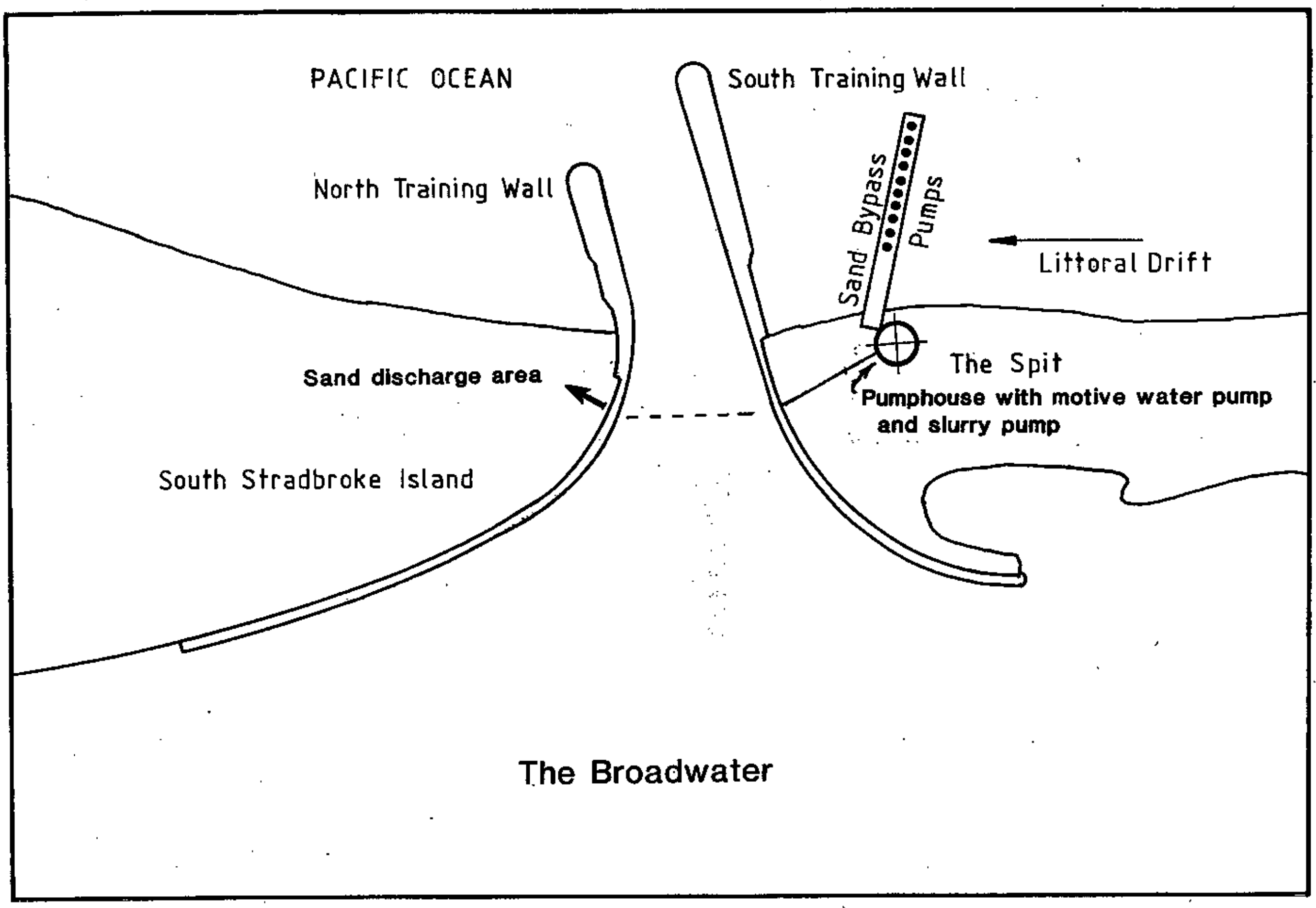


TYPICAL JET PUMP CHARACTERISTICS

TYPICAL JET PUMP LAYOUT & CHARACTERISTICS FIGURE 1

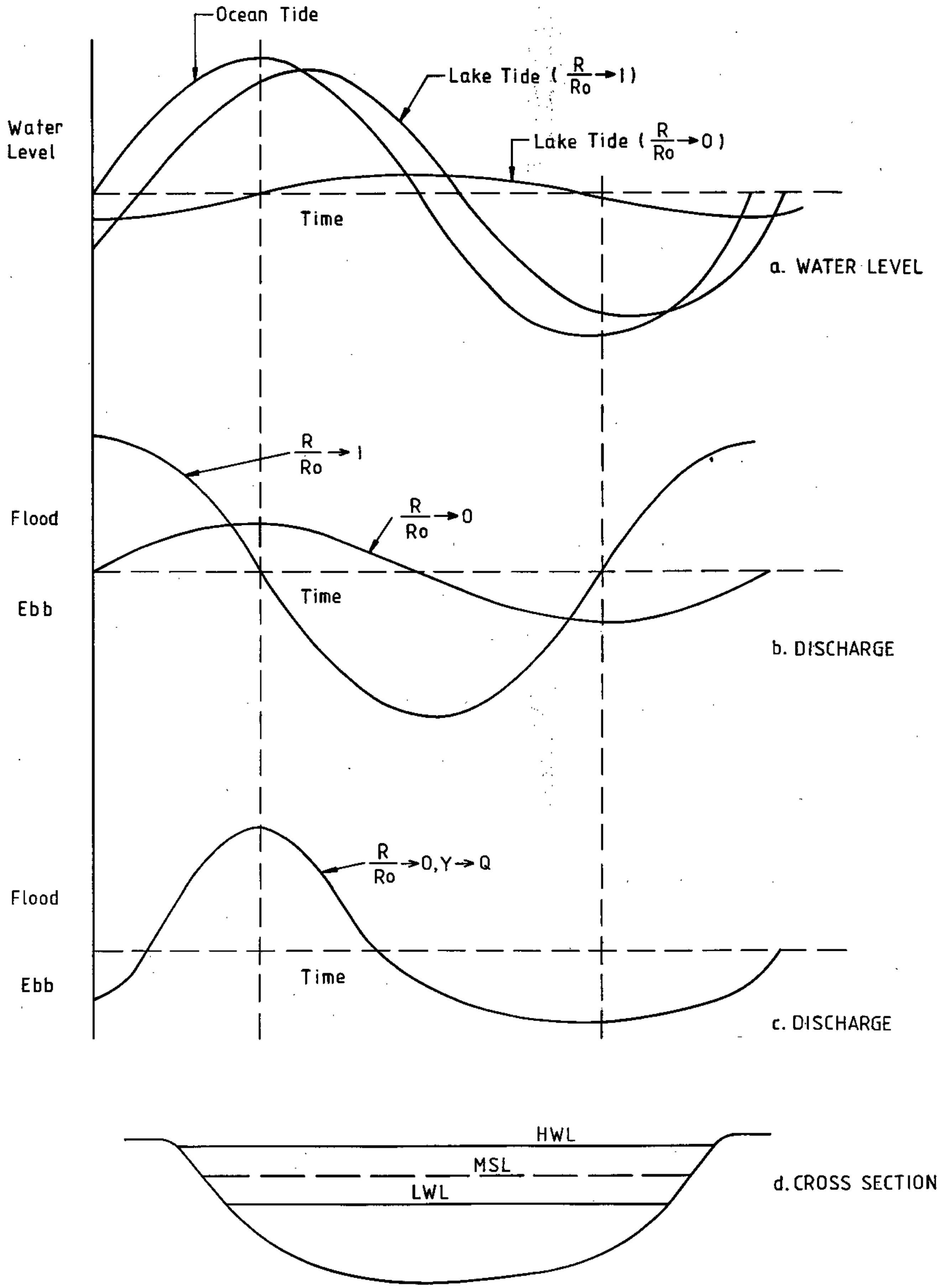


THE GENFLO SANDBUG FIGURE 2



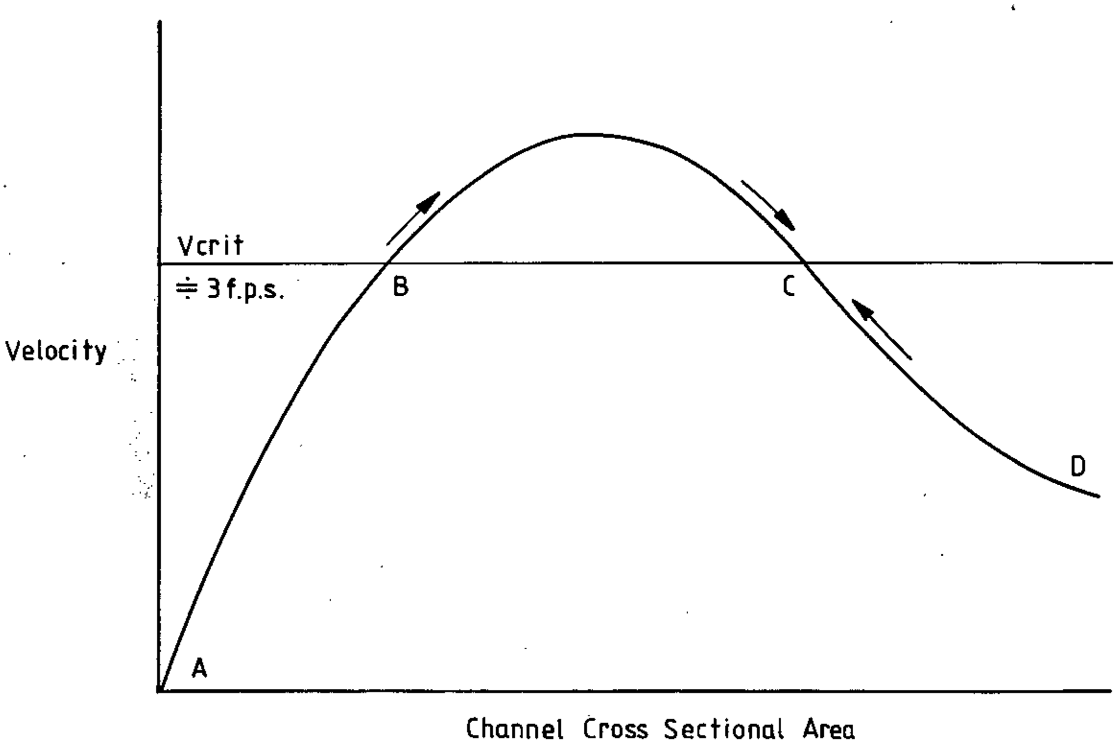
NERANG BYPASSING PLANT

FIGURE 3



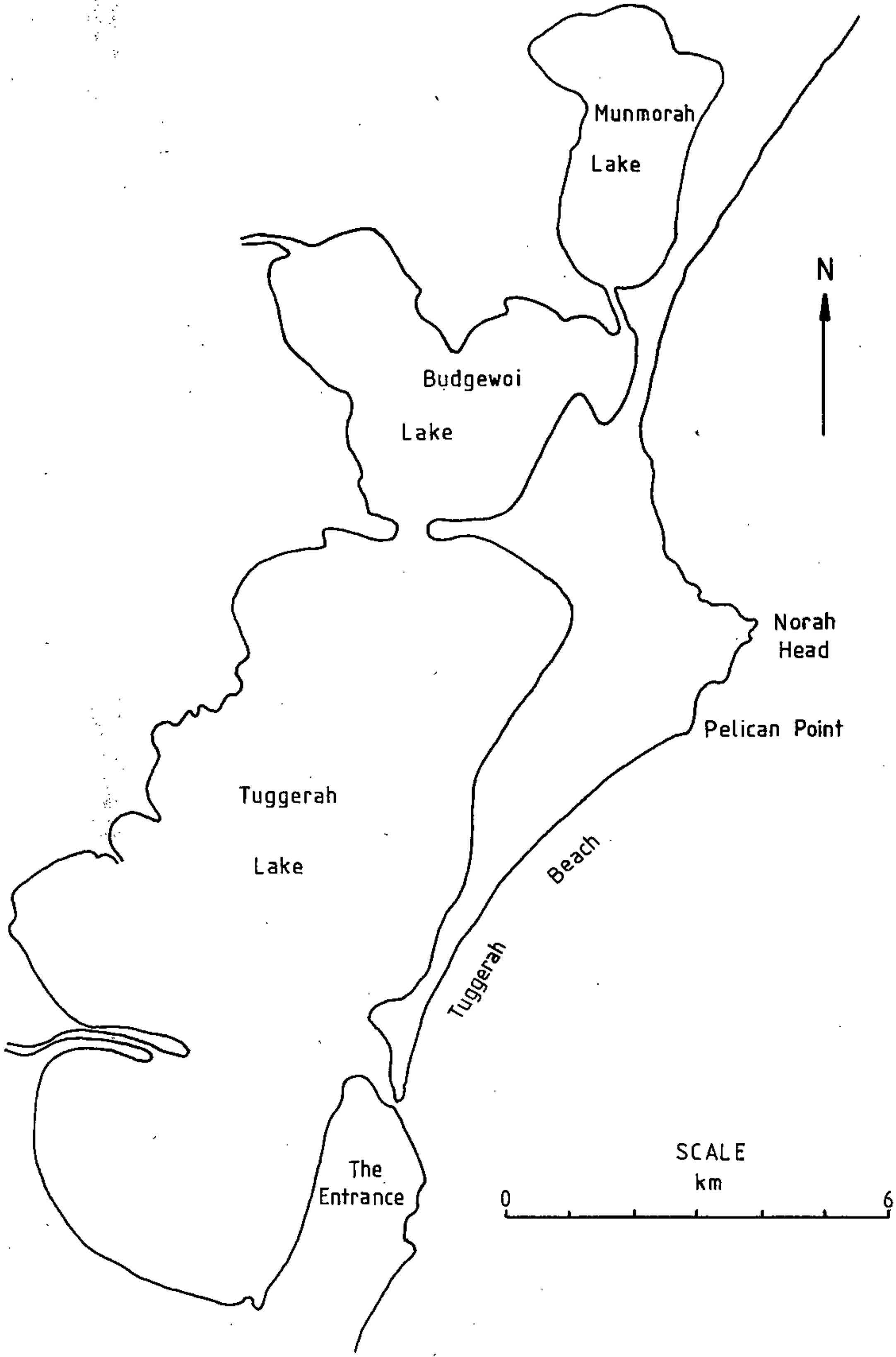
ENTRANCE CHANNELS TIDAL CHARACTERISTICS

FIGURE 4

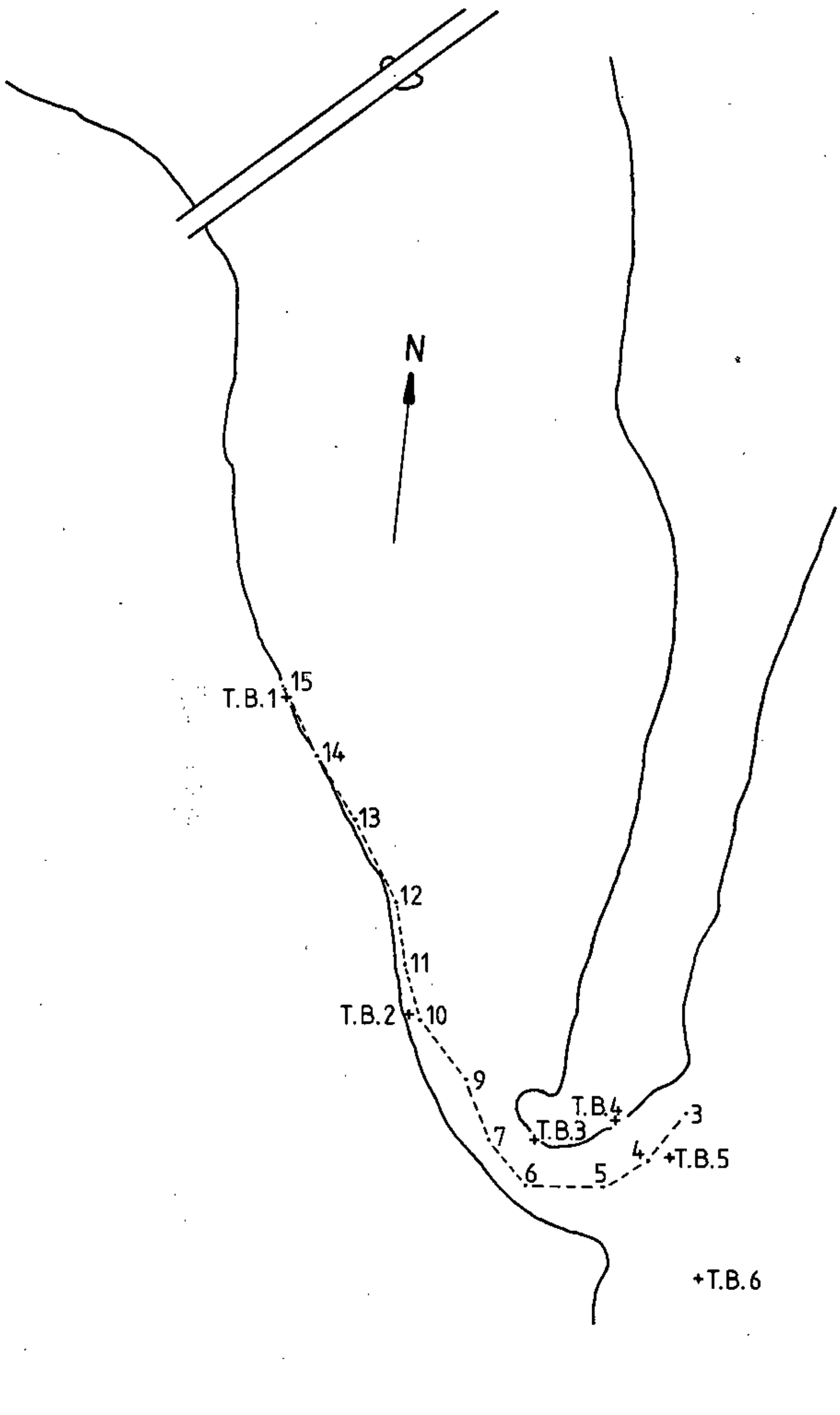


ESCOFFIER STABILITY DIAGRAM

FIGURE 5

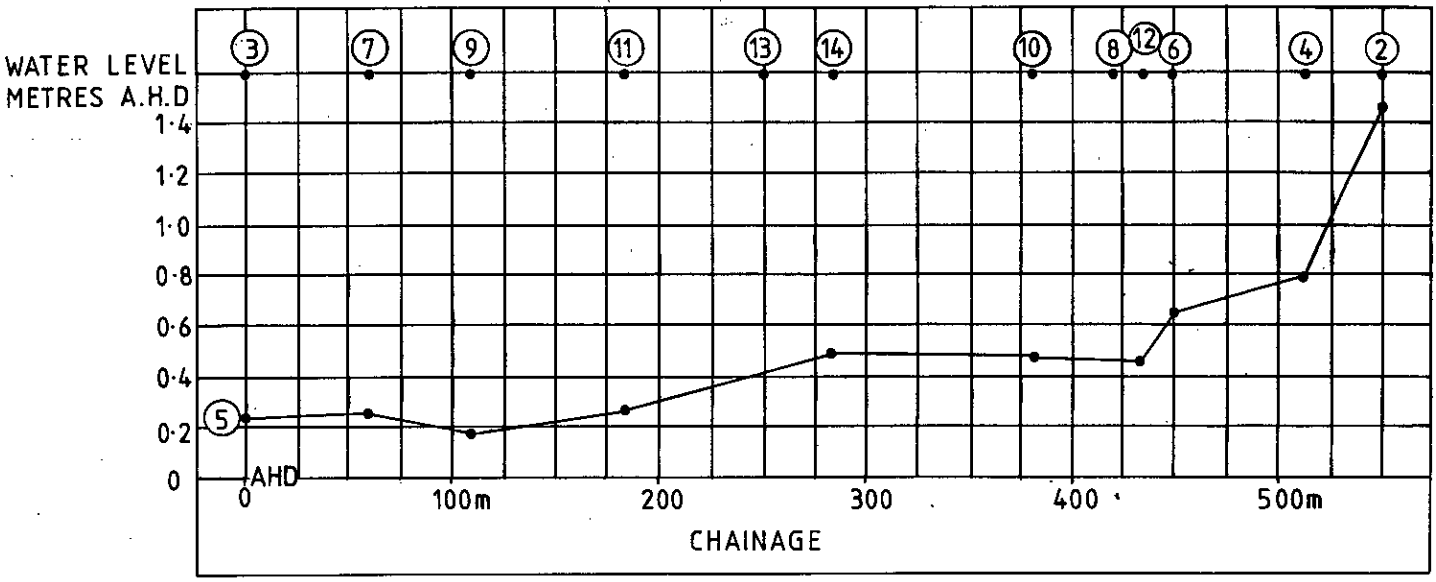


TUGGERAH LAKES SYSTEM SCALE 1: 100,000 FIGURE 6

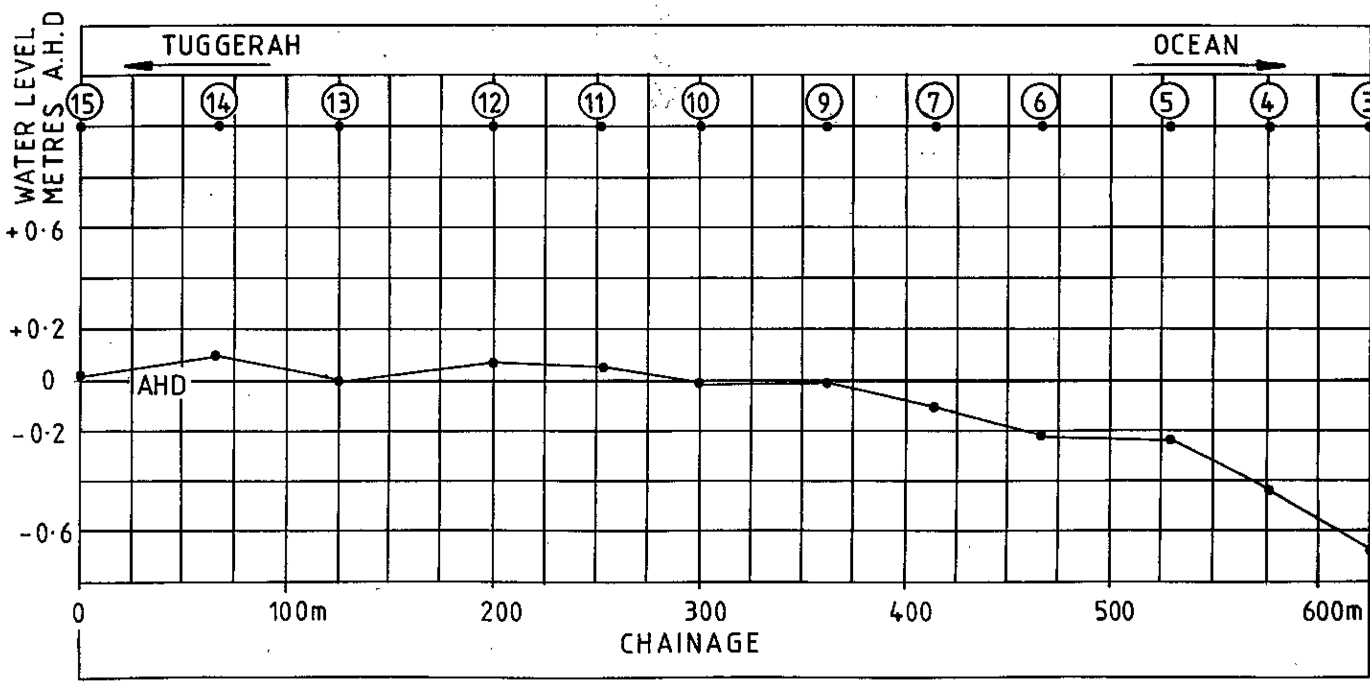


LOCATION OF WATER MEASURING POINTS
8/10/75

FIGURE 7



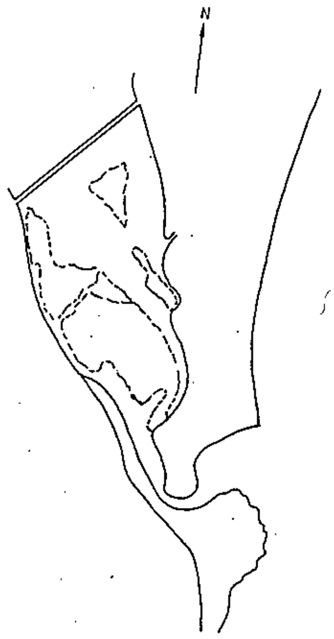
FLOOD TIDE - 10·00 to 10·30



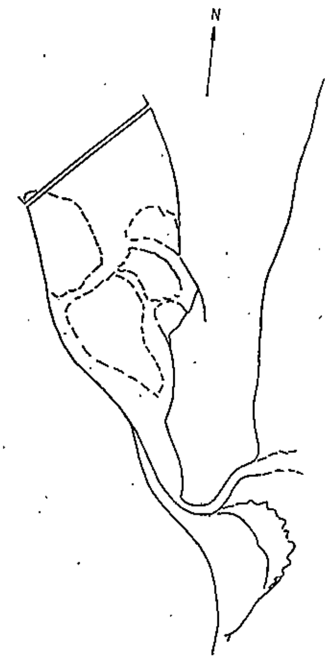
EBB TIDE - 1530 to 1555

NOTE: FOR LOCATION OF MEASURING POINTS SEE FIGURE 7

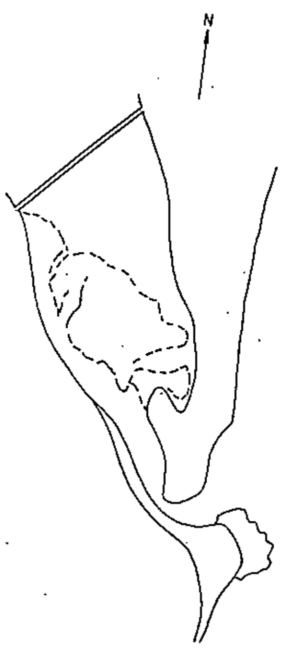
PEAK TIDAL GRADIENTS 8/10/75 FIGURE 8



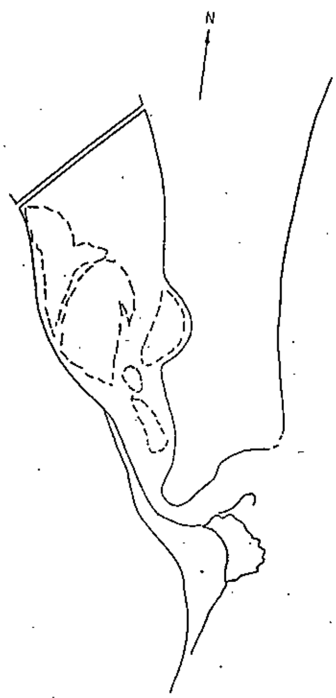
HISTORY OF ENTRANCE MOVEMENT
DATE: MARCH 1954 SCALE 1:16 000 FIGURE 9A



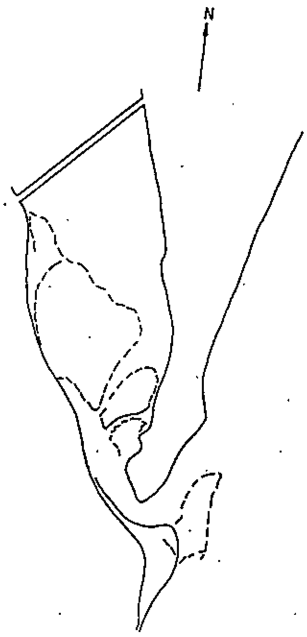
HISTORY OF ENTRANCE MOVEMENT
DATE 23/9/61 SCALE 1:16 000 FIGURE 9B



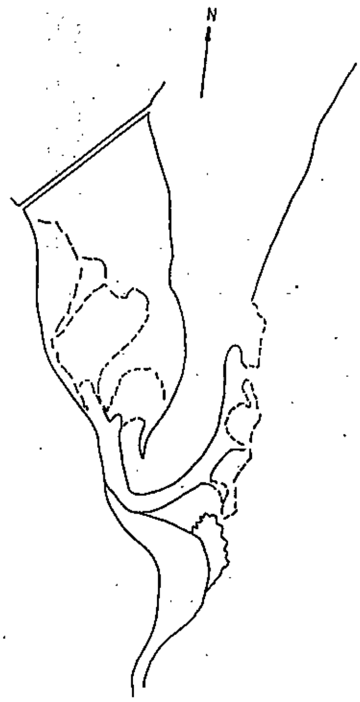
HISTORY OF ENTRANCE MOVEMENT
DATE 18/8/65 SCALE 1:16 000 FIGURE 9C



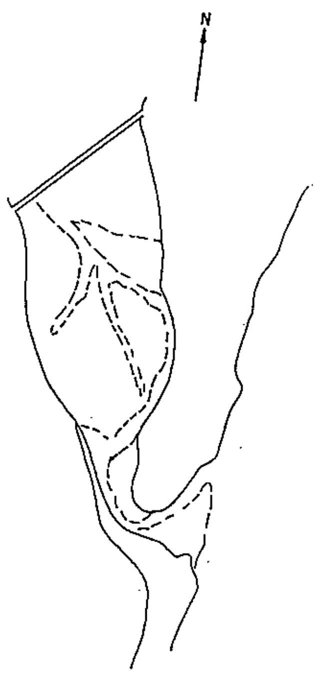
HISTORY OF ENTRANCE MOVEMENT
DATE 30/7/67 SCALE 1:16 000 FIGURE 9D



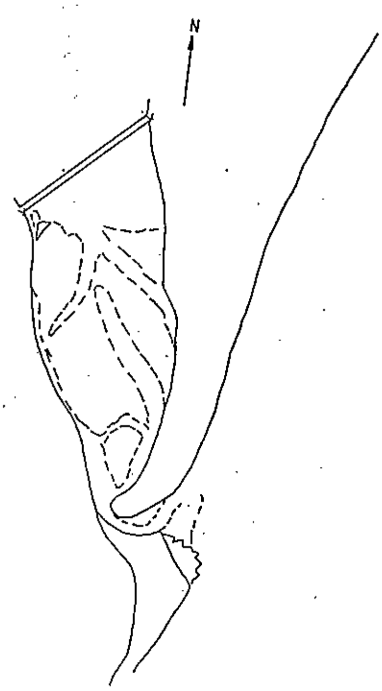
HISTORY OF ENTRANCE MOVEMENT
DATE 6/7/69 SCALE 1:16 000 FIGURE 9E



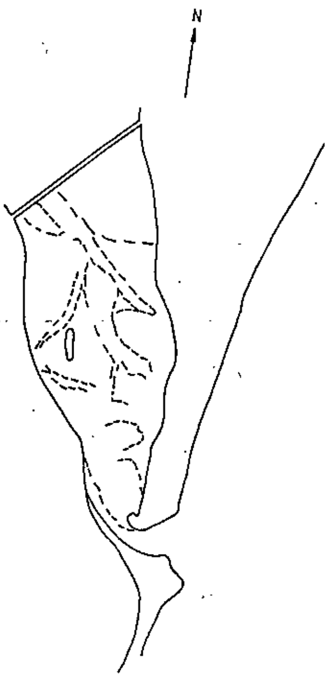
HISTORY OF ENTRANCE MOVEMENT
DATE 16/5/70 SCALE 1:16 000 FIGURE 9F



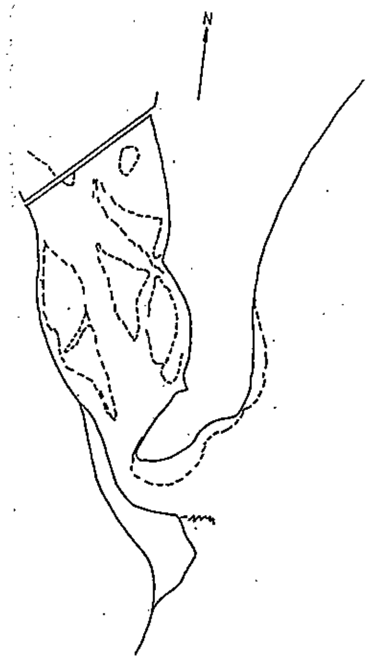
HISTORY OF ENTRANCE MOVEMENT
DATE 22/9/71 SCALE 1:16000 FIGURE 9G



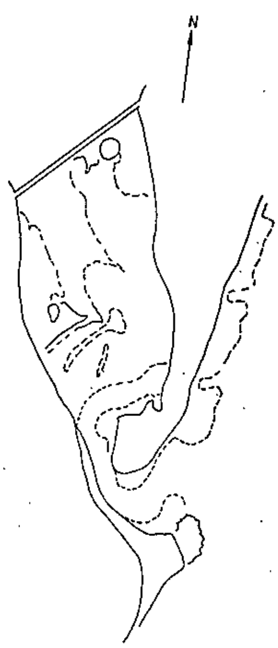
HISTORY OF ENTRANCE MOVEMENT
DATE 1/7/72 SCALE 1:16000 FIGURE 9H



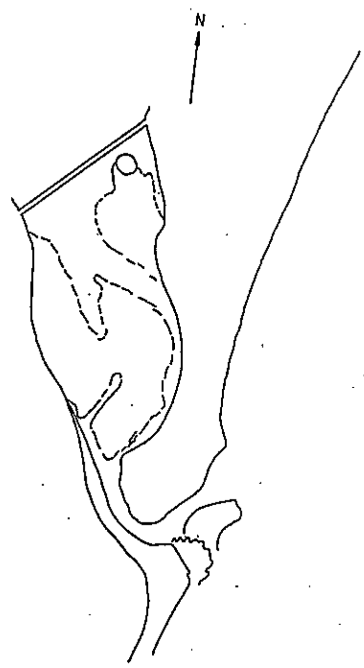
HISTORY OF ENTRANCE MOVEMENT
DATE 6/2/73 SCALE 1:16 000 FIGURE 9I



HISTORY OF ENTRANCE MOVEMENT
DATE 19/6/74 SCALE 1:16 000 FIGURE 9J



HISTORY OF ENTRANCE MOVEMENT
DATE 17/11/76 SCALE 1:16 000 FIGURE 9K



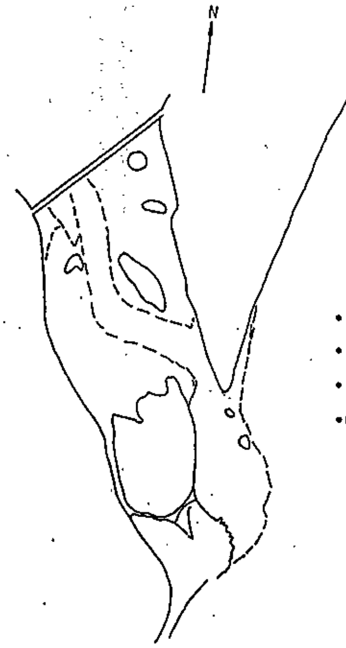
HISTORY OF ENTRANCE MOVEMENT
DATE 7/7/79 SCALE 1:16 000 FIGURE 9L

- ENTRANCE CLOSED
- BULLDOZER CUT (SHOWN CROSS HATCHED)
- LAKE LEVEL 0·93m AHD

NOTE: FULL OUTLINES DENOTE WATER LEVEL
BROKEN LINES DENOTE SHOALS & CHANNELS BELOW WATER LEVEL

ENTRANCE MOVEMENT AUG 86 TO APR 87

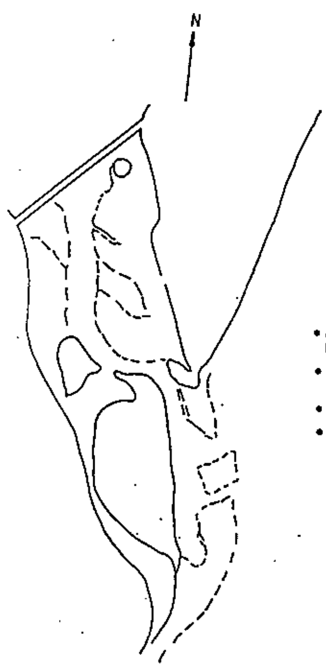
DATE 11/8/86 SCALE 1:16 000 FIGURE 10A



- WELL DEVELOPED CHANNEL UPSTREAM OF BEACH BERM
- 300 METRE WIDE CHANNEL AT BEACH OUTLET
- EXTENSIVE SUBMERGED SHOALS IN THE NEAR OFFSHORE
- LAKE LEVEL 0·4m AHD

ENTRANCE MOVEMENT AUG 86 TO APR 87

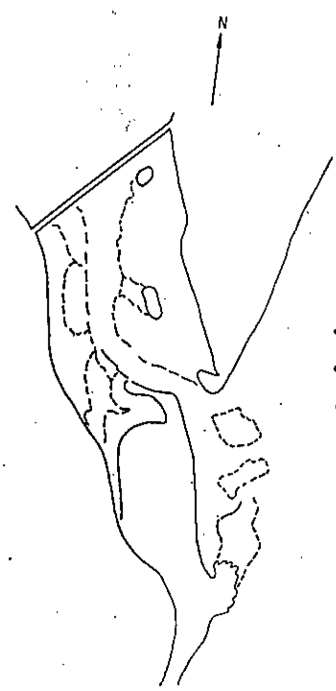
DATE 18/8/86 SCALE 1:16 000 FIGURE 10B



- MAIN CHANNEL SHOWING SLIGHT MEANDERING
- MULTIPLE BEACH OUTLET CHANNELS DEVELOPING
- NORTHERN SPIT DEVELOPING
- LAKE LEVEL 0·14m AHD

ENTRANCE MOVEMENT AUG 86 TO APR 87

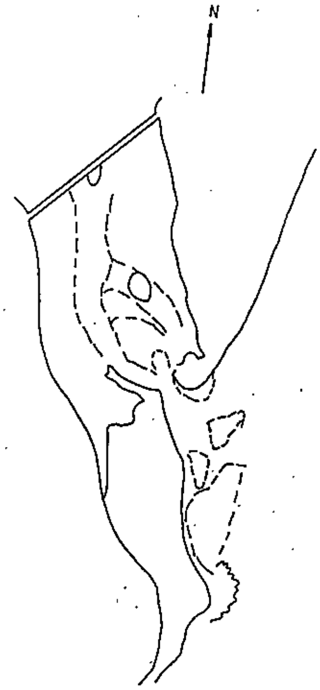
DATE 4/9/86 SCALE 1: 16 000 FIGURE 10C



- THREE WELL DEFINED CHANNELS TO OFFSHORE
- SLIGHT CHANGE IN POSITION OF MAIN CHANNEL
- LAKE LEVEL 0·16m AHD

ENTRANCE MOVEMENT AUG 86 TO APR 87

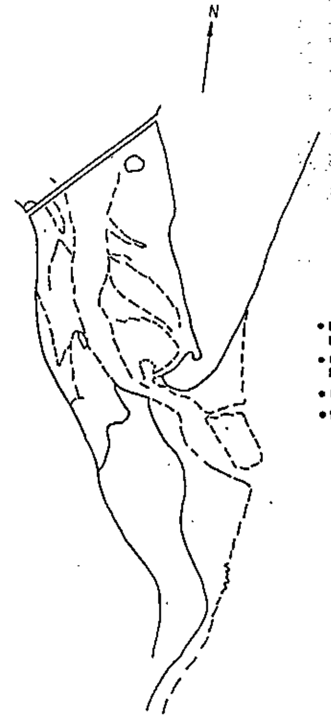
DATE 11/9/86 SCALE 1:16 000 FIGURE 10D



- NORTHERN SPIT DEVELOPING FURTHER
- OFFSHORE SHOALS BEGINNING TO REATTACH
- TREND TOWARDS ONE CHANNEL
- LAKE LEVEL 0·18m AHD

ENTRANCE MOVEMENT AUG 86 TO APR 87

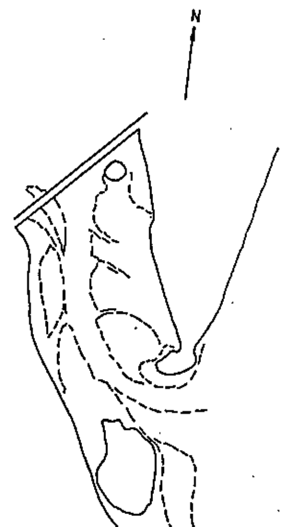
DATE 24/9/86 SCALE 1:16 000 FIGURE 10E



- FURTHER DEVELOPMENT OF NORTHERN SPIT
- ONE MAIN CHANNEL TRENDING S.E IN THE NEAR OFFSHORE
- OFFSHORE SHOALS REATTACHED
- LAKE LEVEL 0·17m AHD

ENTRANCE MOVEMENT AUG 86 TO APR 87

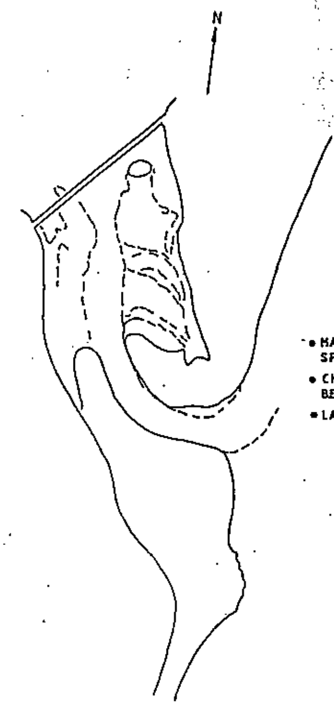
DATE 14/10/86 SCALE 1:16 000 FIGURE 10F



- WELL DEVELOPED SINGLE CHANNEL
- SHOREWARD MOVEMENT OF OFFSHORE SHOALS
- LAKE LEVEL 0·22m AHD

ENTRANCE MOVEMENT AUG 86 TO APR 87

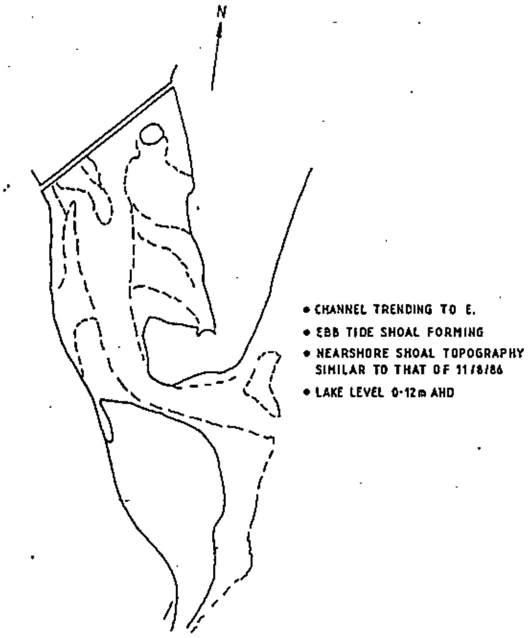
DATE 3/11/86 SCALE 1:16.000 FIGURE 10G



- MAJOR DEVELOPMENT OF NORTHERN SPIT
- CHANNEL SHOWING N.E TREND AT BEACH LINE
- LAKE LEVEL 0·22m AHD

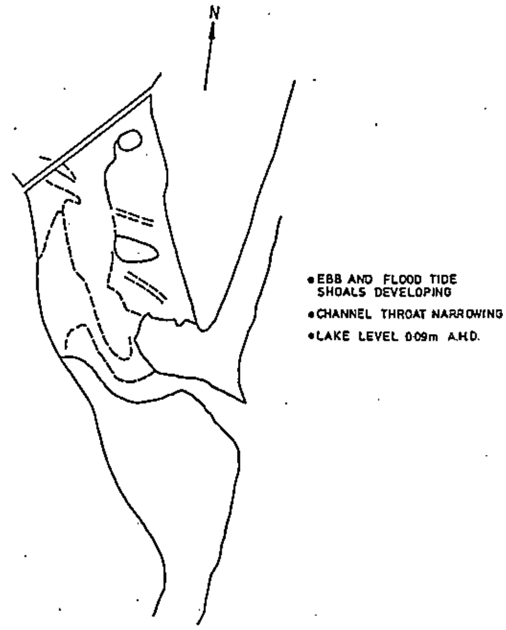
ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 26/11/86 SCALE 1:16 000 FIGURE 10H



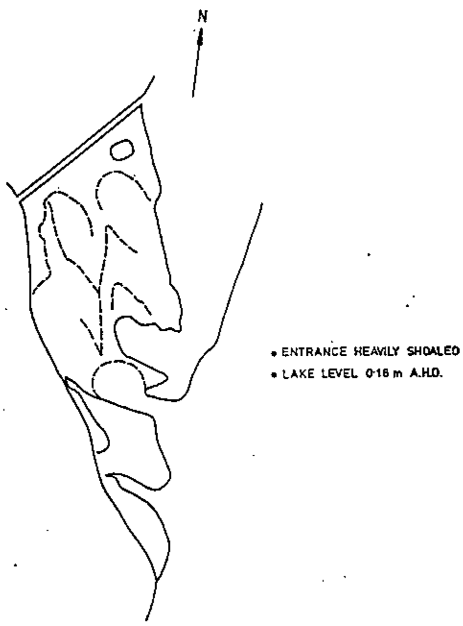
ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 12/12/86 SCALE 1:16 000 FIGURE 10I



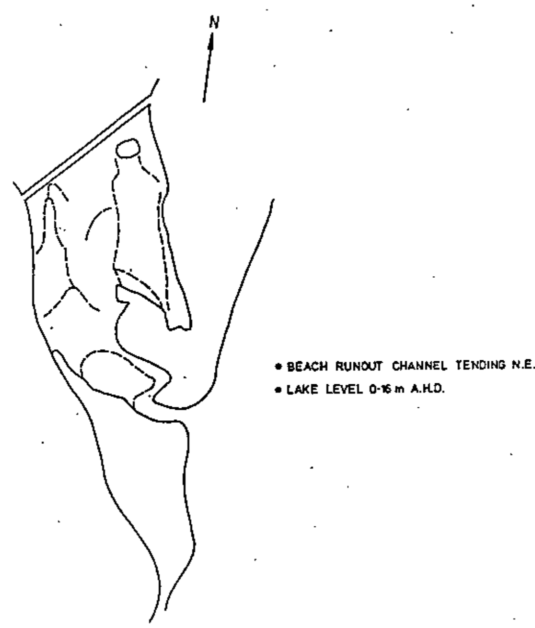
ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 14/1/87 SCALE 1:16 000 FIGURE 10J



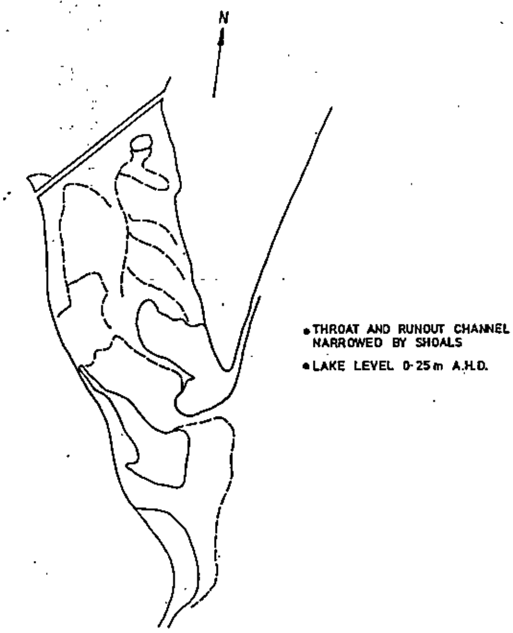
ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 30/1/87 SCALE 1:16 000 FIGURE 10K



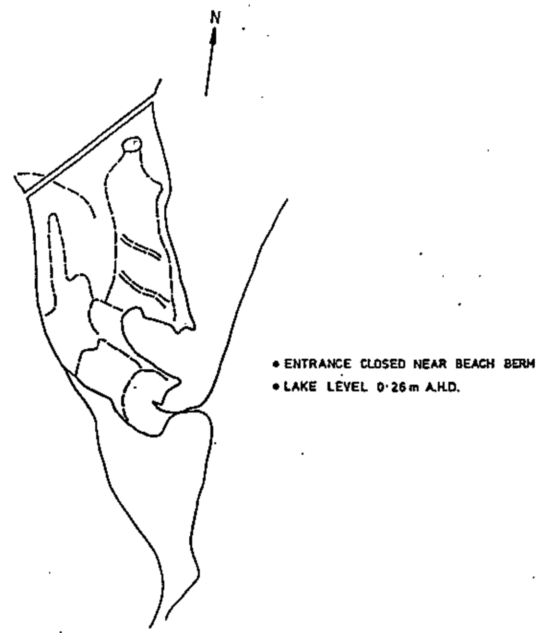
ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 12/2/87 SCALE 1:16 000 FIGURE 10L



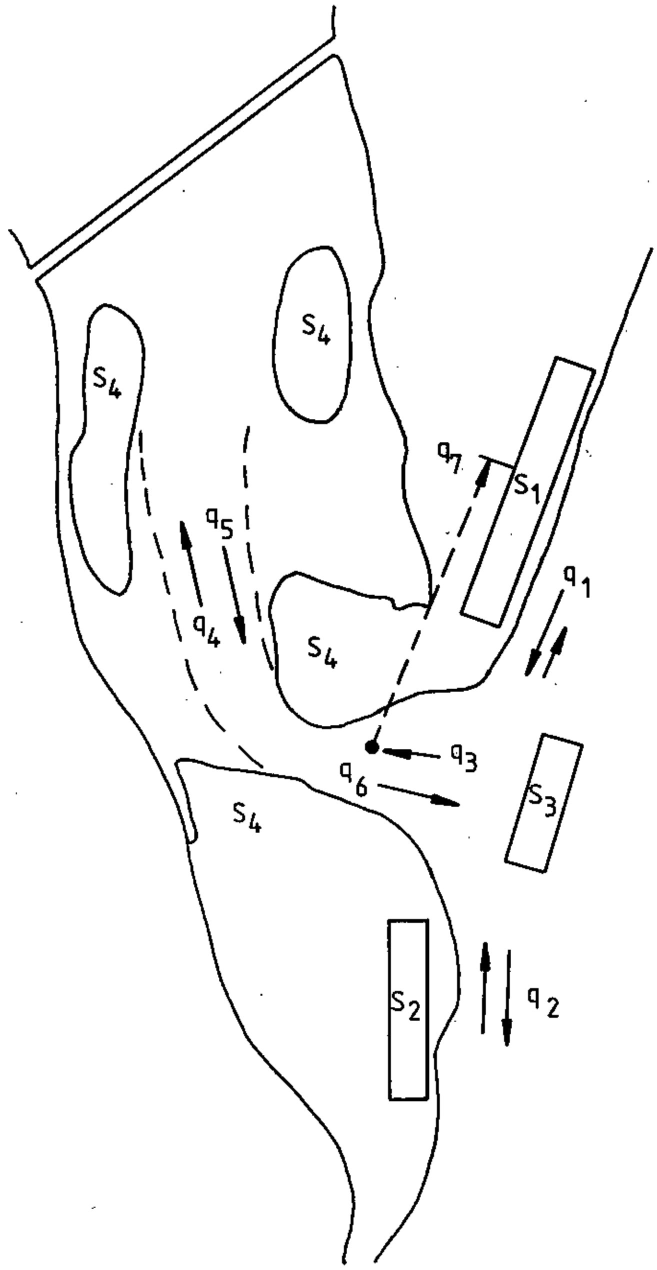
ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 18/3/87 SCALE 1:16 000 FIGURE 10 M



ENTRANCE MOVEMENT AUG 86 TO APR 87

DATE 6/4/87 SCALE 1:16 000 FIGURE 10 N



S_1 - Sand in store on beach to the North

S_2 - Sand in store on beach to the South

S_3 - Sand in store in offshore bars

S_4 - Sand in store in entrance channel shoals

q_1, q_2 - Longshore sediment flux (littoral drift)

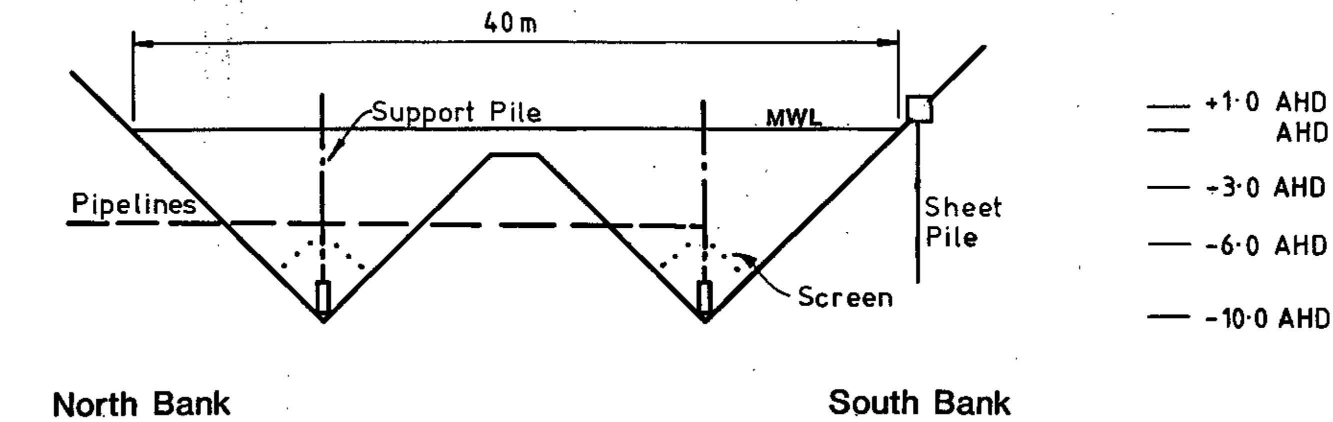
q_3, q_4 - Flood tide sediment flux

q_5, q_6 - Ebb tide sediment flux

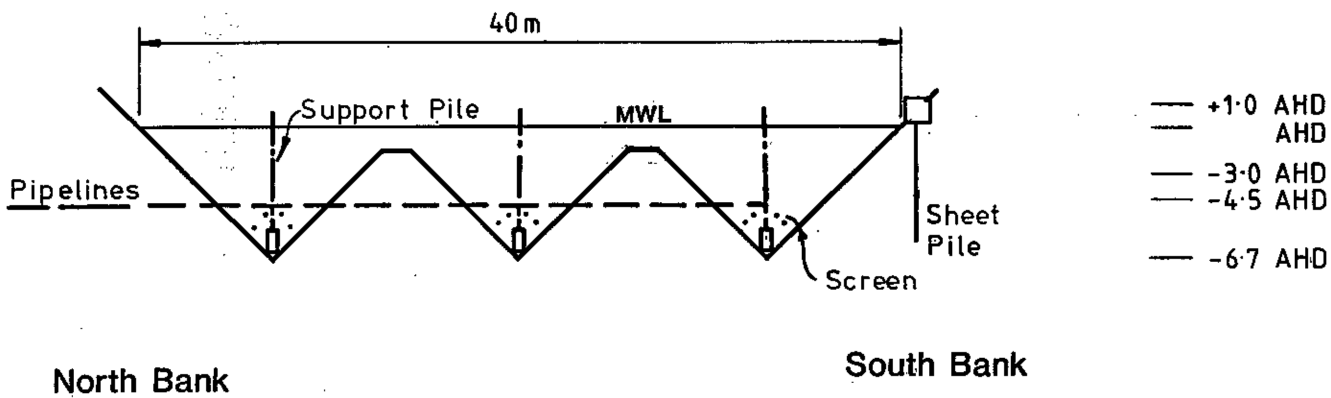
q_7 - Sediment flux through jet pump

SEDIMENT BUDGET DEFINITION PLAN

FIGURE 11



TWO PUMP ARRANGEMENT

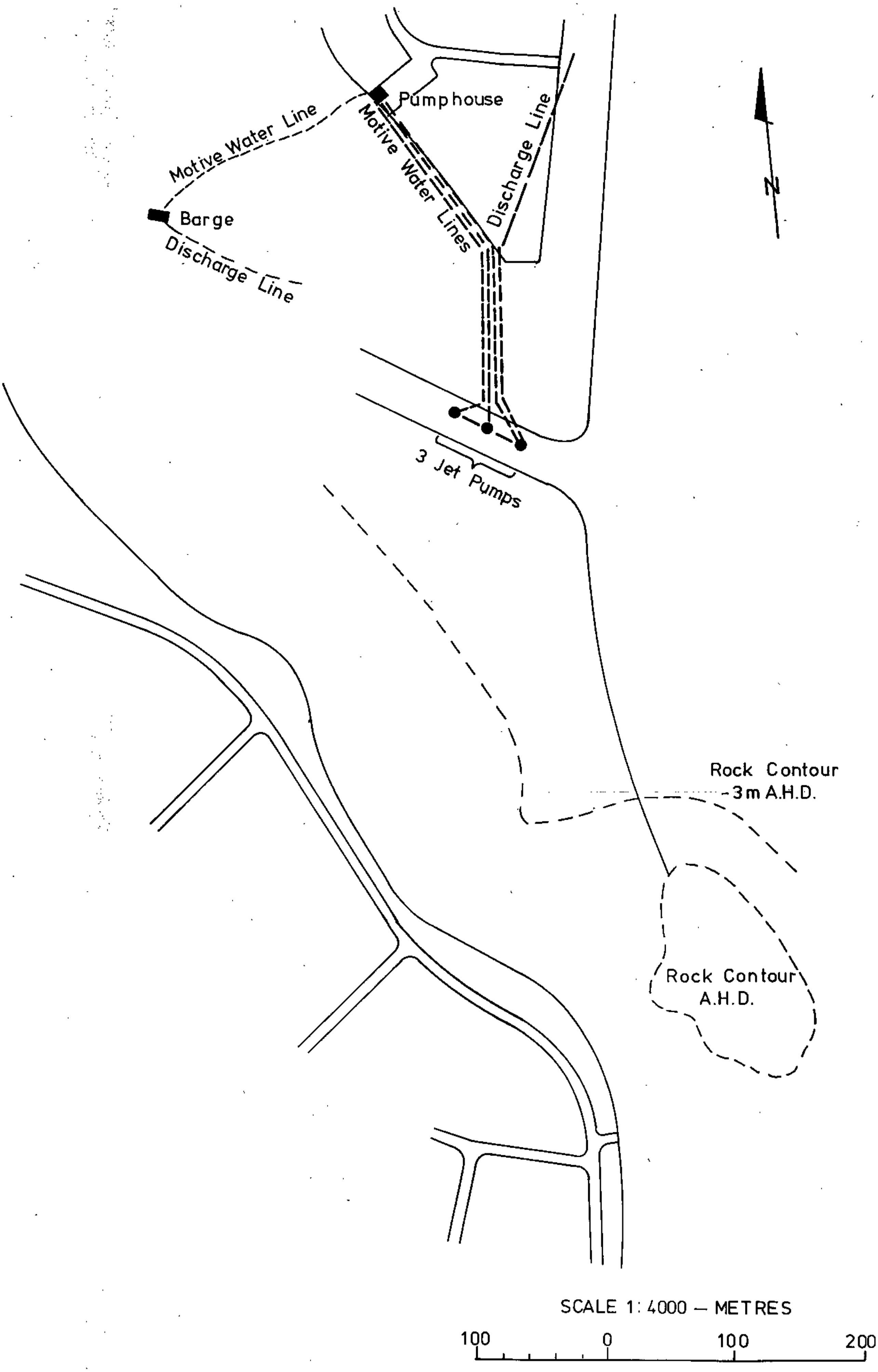


THREE PUMP ARRANGEMENT

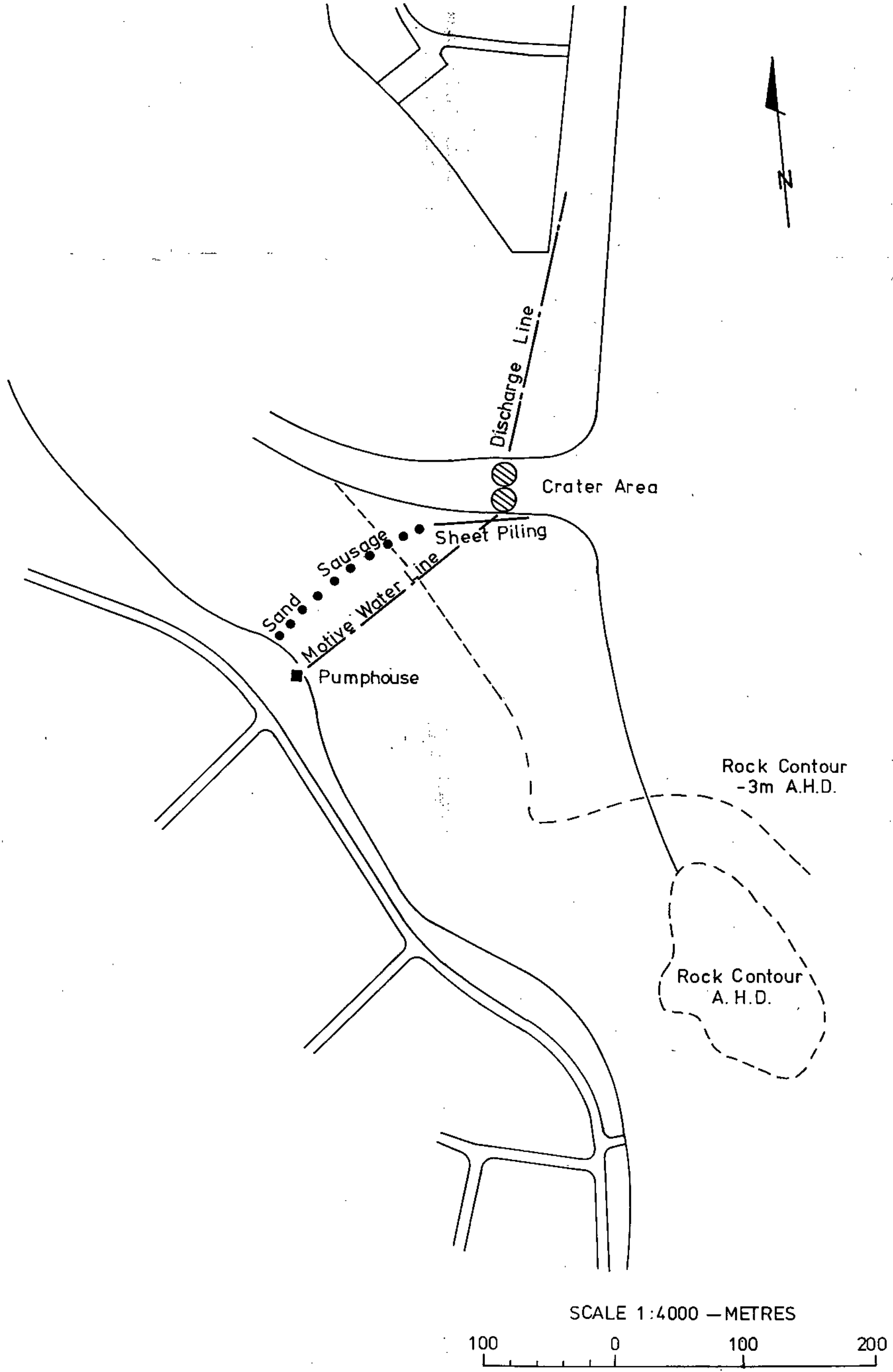
CROSS SECTION AT CRATER AREA

SCALE 1:400

FIGURE 12

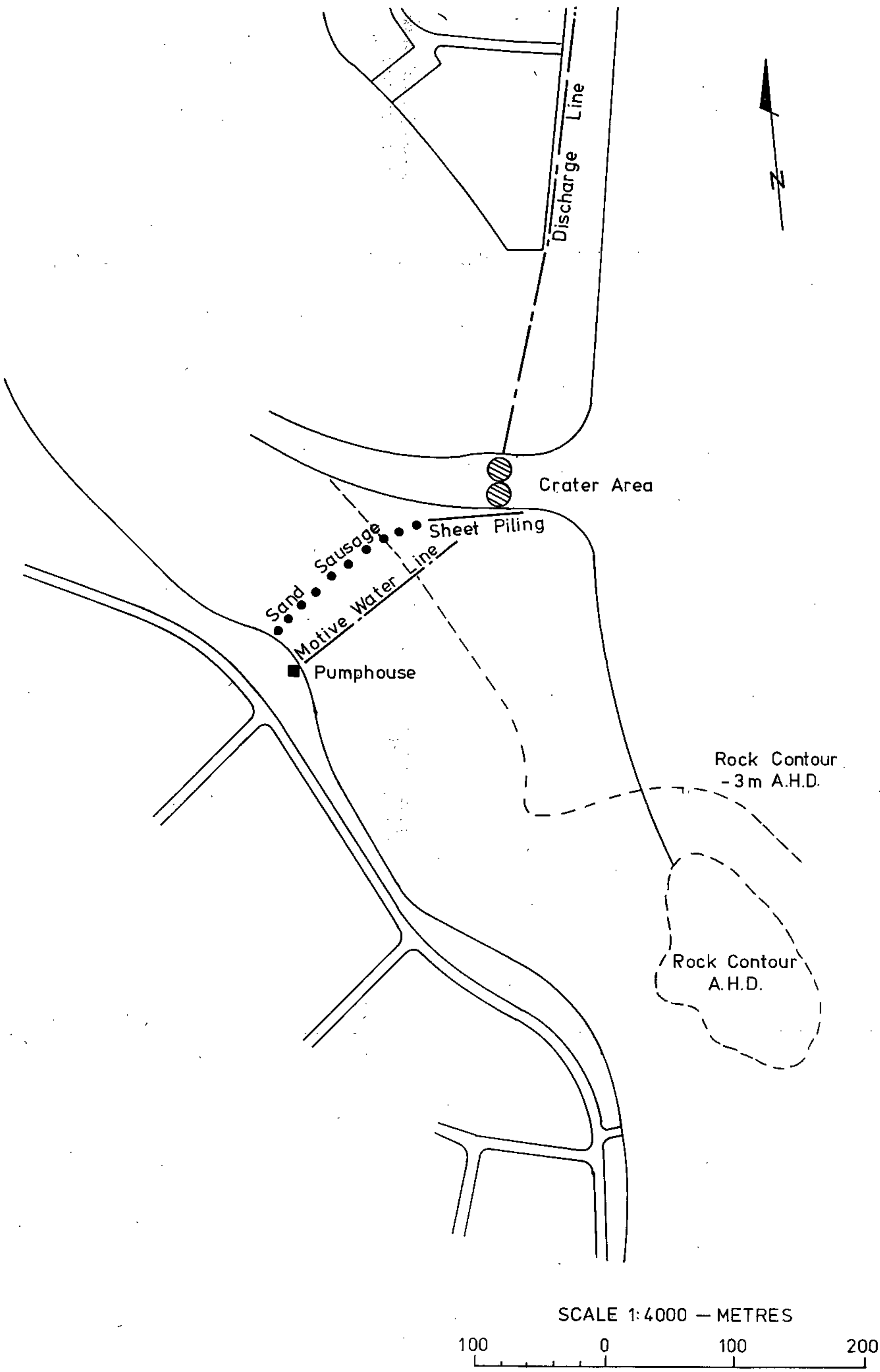


KIRRAWEE ENGINEERING SERVICES PROPOSAL FIGURE 13



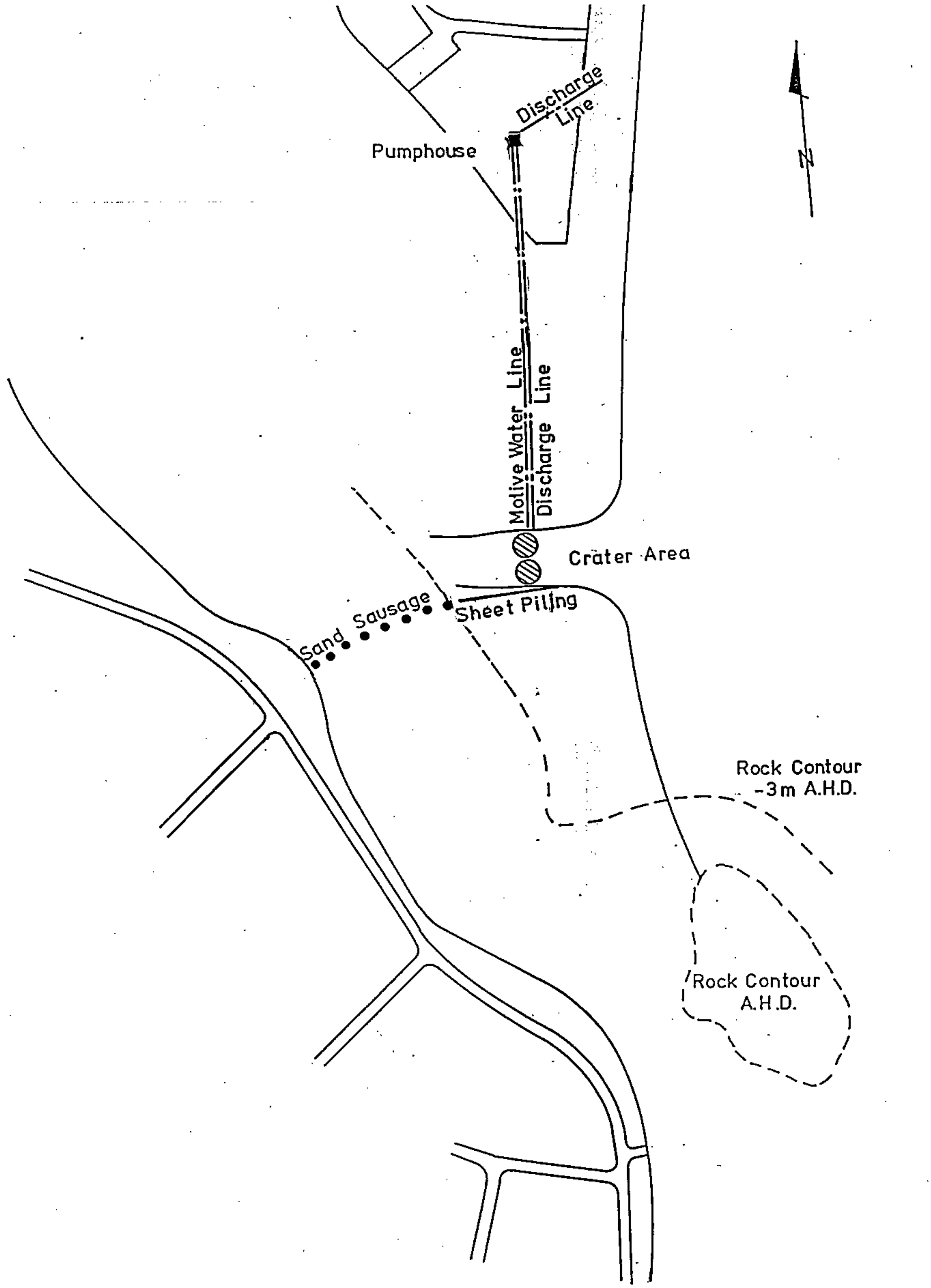
OPTION 1

FIGURE 14



OPTION 2

FIGURE 15

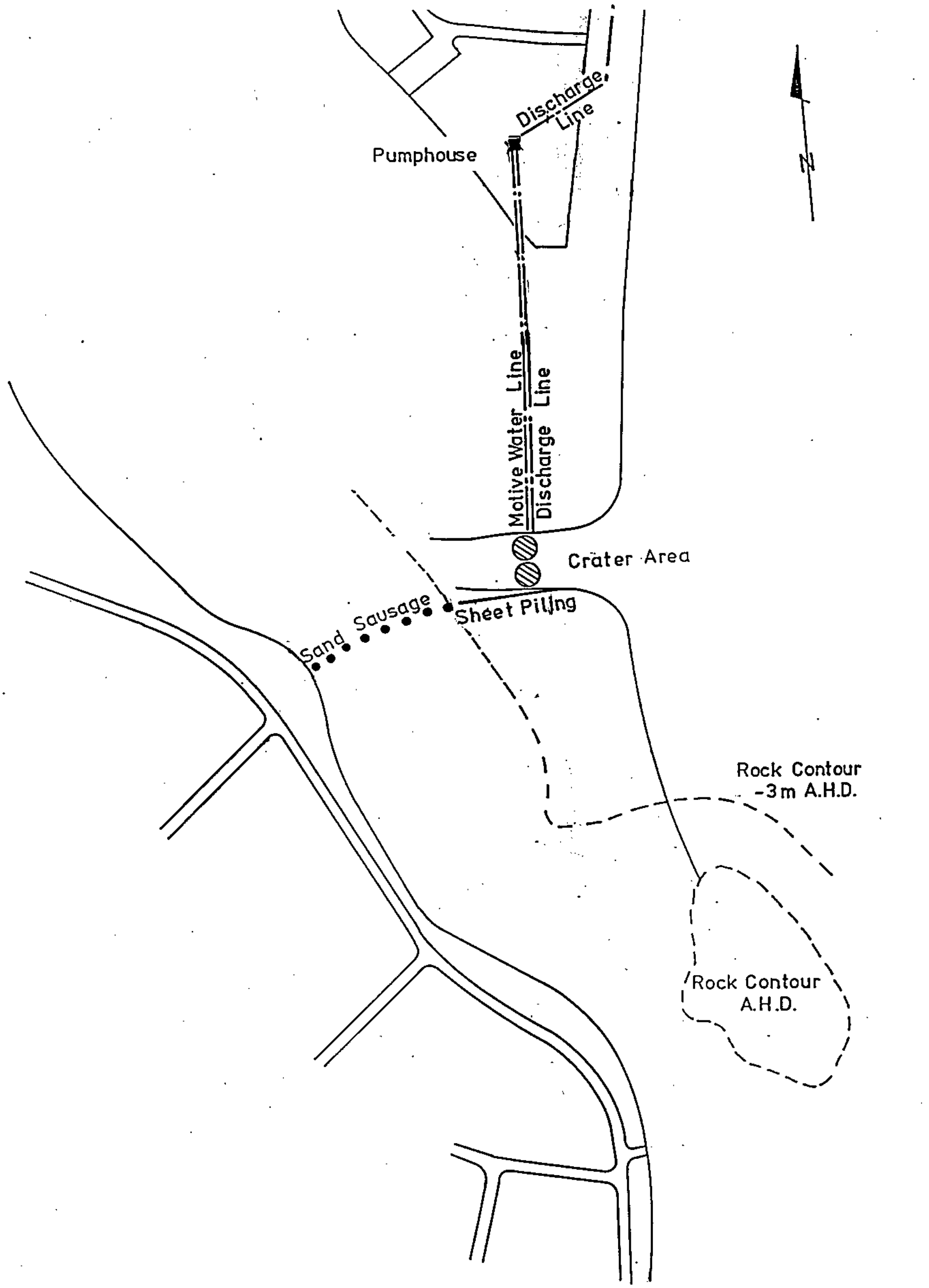


SCALE 1: 4000 – METRES

100 0 100 200

OPTION 3

FIGURE 16



SCALE 1: 4000 – METRES

100 0 100 200

OPTION 4

FIGURE 17

APPENDIX A : LETTER FROM AMERICA

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P.O. BOX 631
VICKSBURG, MISSISSIPPI 39180-0631

REPLY TO
ATTENTION OF

April 24, 1987

Engineering Development
Division

Mr. Bruce Druery
Supervising Engineer, Estuary Management
Public Works Department
140 Phillip Street
Sydney 2000
AUSTRALIA

Dear Mr. Druery:

Enclosed are a number of references on sand bypassing and, in particular, jet pump bypassing systems, as requested in your letter and in conversations I have had with Mr. Chris Miller. Enclosure 1 is a list of non-Corps of Engineers contacts in the United States for locations where fixed bypassing plants either have been built or are contemplated. I am sure all of the persons listed would be pleased to share their experiences with you. Enclosure 2 is a keyworded bibliography on sand bypassing covering a variety of systems and projects. Enclosure 3 is a paper describing the conceptual design of a large experimental jet pump bypassing system currently under construction at Oceanside Harbor, California. Enclosure 4 describes the Supervisory Control and Data Acquisition (SCADA) system that will be used to run the Oceanside system and report on its performance.

Enclosure 5 is included in response to discussions with Mr. Miller about nozzle erosion problems encountered in the Nerang River jet pump bypassing system, and the performance characteristics of jet pump designs other than the one used on that project. Enclosure 6 is a set of specifications for a trailer-mounted jet pump system used quite successfully by the Canadian Department of Public Works to maintain the entrances to a number of small fishing harbors on Prince Edward Island in the Maritime Provinces. Their design was an improvement and refinement of a prototype system we designed and constructed approximately 10 years ago for the Detroit District of the Corps of Engineers. If you would like further information on the Canadian system, the appropriate contact is Mr. Charles Panciuk (pronounced Pan-Chuck), Marine Directorate, Public Works, Sir Charles Tupper Building, Ottawa, Canada, K1A OM2. His telephone is (613) 998-8171. Mr. Panciuk has been extremely cooperative in sharing his operating experiences.

Enclosure 7 describes a system being marketed in the United States for sand bypassing that uses the crater principle. We have very limited experience in its application and operational characteristics, but the concept is interesting. The mechanical transport mechanism may be useful in overcoming one of the most nagging operational problems with any type of crater system; i.e., how to handle the incredible variety of non-sand material that seems to find its way into your crater. Enclosure 8 is a transcription of a tape recorded report one of our engineers made after observing a prototype of this system in operation at an inlet in Florida.

Enclosure 9 is a feasibility study performed for sand bypassing and beach restoration in Florida. You will notice that the study recommends a combination of a jet pump system and structural measure (weir jetty) for sand bypassing. In general, our experience has been that most types of bypassing systems are more efficient when used in combination with structures that channelize sand flow or that cause sand to deposit in a specified region. Even a hydraulic dredge used for bypassing seems to work more efficiently when the area it has to span for sand recovery is limited in this fashion.

Enclosures 10 and 11 describe the concept of using buried fluidizers to maintain flow through small tidal inlets susceptible to closure. The concept has been demonstrated successfully in a very limited fashion at several locations in the United States, but not on an operational basis. Fluidizers offer a logical way to overcome one of the inherent limitations of a jet pump crater system; i.e., the steep side slopes of the crater limit its "capture diameter" rather severely. As you can see from reading the Oceanside references, fluidizers in combination with jet pumps will be part of that experiment.

I hope the enclosed material will be of some assistance to you. I apologize for my delay in answering your initial letter, but the subsequent discussions with Mr. Miller served to transmit some of our experience and also to clarify our understanding of your needs. After you have reviewed these references, please do not hesitate to write or call for clarification or further information. I will keep you abreast of developments in the Oceanside experiment or any other applicable bypassing projects, and would be interested in following your experiences in sand bypassing. I believe that the proper design and operation of a bypassing plant is a subtle art; most of us that work in bypassing have learned what little we know through shared experiences. Good luck with your efforts.

Thomas W. Richardson
Chief,
Engineering Development Division
Coastal Engineering Research Center

Enclosures

APPENDIX B: COST ESTIMATES (SEPTEMBER 1987)

TABLE B1: CAPITAL COSTS

The following costs are common to each of the four jet pump optional arrangements:

	$
* 2 Genflo Jet Pumps	24,000
* Mobile Dredge-Mounted Jet Pump	61,000
* Excavation of Jet Pump Craters (8000 m^3)	25,000
* Pipe and Jet Pump Supports - piles (4), bracing and headstocks - pipe carrier, pump carriages and guides - pump screens, pipe fittings - installation of all mechanical equipment	50,000
* Restraining Wall - comprising 75 m of sheet pile (strengthened at the seaward end to resist wave attack) and 140 m of flexible geo-textile formed sand revetment	120,000
* Investigation and Design - comprehensive drilling to establish rock levels - investigation of restraining wall, pump supports, etc. - detailed system design - system commissioning, evaluation and optimization	100,000
Total for Common Items	**$380,000**

TABLE B1: CAPITAL COSTS (contd)

OPTION 1

			$
* Common Items			380,000
* Pipelines:	Excavation & Backfill	$40,400	
	Pipes	$11,200	
	Lay and Test	$ 2,000	51,600
* Pumphouse			16,000
* Motive-water Pumpset			40,000
* Switchgear and Power Connection			28,000
* Contingency#			279,400 54%
			$795,000

OPTION 2

* Common Items			380,000
* Pipelines:	Excavation and backfill	$48,400	
	Pipes	$14,600	
	Lay and Test	$ 4,000	67,000
* Pumphouse			16,000
* Motive-water Pumpset			55,000
* Switchgear and Power Connection			28,000
* Contingency#			287,000 52.5%
			$833,000

TABLE B1: CAPITAL COSTS (contd)

OPTION 3

			$
* Common Items			380,000
* Pipelines:	Excavation and backfill	$34,200	
	Pipes	$18,200	
	Lay and test	$ 3,250	55,650
* Pumphouse			25,000
* Motive-water Pumpset			48,000
* Switchgear and Power Connection			28,000
* Contingency#			284,350
			$821,000

OPTION 4

* Common Items			380,000
* Pipelines:	Excavation and Backfill	$54,200	
	Pipes	$66,900	
	Lay and Test	$ 8,000	129,100
* Pumphouse and Slurry Tank			40,000
* Motive-water Pumpset and Slurry Pumpset			98,000
* Switchgear and Power Connection			38,000
* Contingency#			320,900
			$1,006,000

\# Contingency calculated as 25% of the item costs for each option, plus $150,000 should rock excavation be required to obtain the depth of 10 m below mean sea level for the jet pumps. A significant contingency allowance is required because the location of rock has not been investigated in detail and no single scheme has been subject to detailed design.

TABLE B2: ANNUAL OPERATING COSTS (FIXED PLANT)**

		$
*	Power - 1000 hours @ $12/hour	12,000
*	Operator - 1 day/week @ $120/day	6,000
*	Sand Spreading - say 20000 m^3 @ $0.25/$m^3$	5,000
*	Pump Clearance - 5 No @ $3,000	15,000
	TOTAL FOR OPTIONS 1,2 and 3	**$ 38,000**
	Additional for Option 4	
*	Slurry Pump Power -1000 hours @ $12/hour	12,000
	TOTAL FOR OPTION 4	**$ 50,000**
	Contingency for additional pump operation:	
	Options 1, 2 and 3; 500 hrs at $12/hr	**$ 6,000**
	Option 4, as above and slurry pump	**$ 12,000**

TABLE B3: ANNUAL MAINTENANCE COSTS (FIXED PLANT)**

		$
*	Remove and Replace Jet Pumps	5,000
*	Overhaul Jet Pumps	2,000
*	Overhaul Motive Water Pump	3,000
	TOTAL FOR OPTIONS 1,2 and 3	**$ 10,000**
	Additional for Option 4	
*	Overhaul Slurry Pump	5,000
	TOTAL FOR OPTION 4	**$ 15,000**

** No provision has been made for operating the mobile barge since the extent of use of this plant would be largely at Council's discretion.