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WYONG SHIRE COUNCIL
PUBLIC WORKS DEPARTMENT

TUGGERAH LAKE ENTRANCE IMPROVEMENTS ENTRANCE RESTRAINING WALL

CONCEPT DESIGN REPORT

Report No. P.W.D.88069
OCTOBER, 1988

PUBLIC WORKS

Coast and Rivers Branch

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CONCEPT DESIGN REPORT

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FOREWORD

Wyong Shire Council requested this study to devise a preliminary concept design for a sand filled geotextile wall to be built at the entrance to Tuggerah Lakes.

The wall is intended to restrain the entrance channel from migrating southwards where it becomes perched on a rock shelf causing tidal flows to weaken and thus contributing to blockage of the entrance by beach sands.

The detailed technical report contains information to assist Council to evaluate this entrance improvement option.

SUMMARY

In 1987 the Public Works Department, in response to a request from Wyong Shire Council, undertook a study into the feasibility of installing jet pumps in the entrance channel of Tuggerah Lakes as a means of maintaining a permanent entrance.

That report concluded that jet pumps may be subject to operational difficulties because of their susceptibility to blockage by kelp and other debris. The report also concluded that jet pumps would need to be located adjacent to a wall which would be necessary to prevent the channel from migrating away from the pumps.

Council considered the report and requested that the wall be proceeded with. The effects of the wall on entrance permanency would be monitored prior to a decision being taken on the need for any additional works. Construction of the wall by itself is an appropriate entrance management strategy, as it is anticipated that by restraining the channel from migrating south over the rock reef, the condition of the entrance channel will be improved.

Throughout this study the Department has worked in close consultation with Council to achieve a result that meets Council's management criteria.

These criteria are:

i) that the wall is not to be constructed of rock;

ii) a preference that the wall be constructed of geotextile tubes;

iii) that the wall be compatible with a future jet pump scheme, if required;

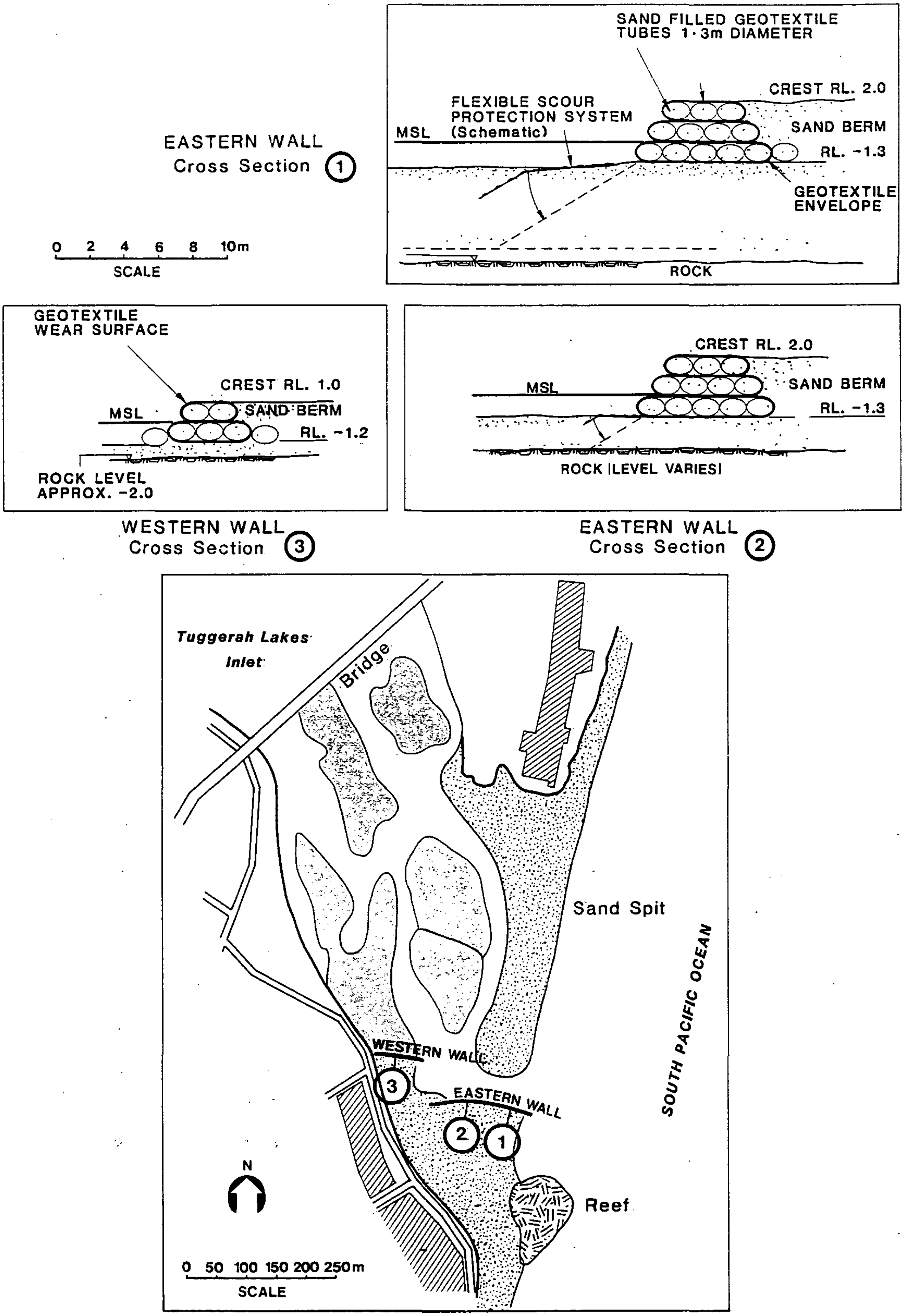
iv) that emphasis be placed on the aesthetic appearance of the structure.

Rock is the material traditionally used for this type of structure, however, Council considers it undesirable on visual and recreational grounds. It would also be troublesome if jet pumps were installed as any rocks falling into the jet pump craters would contribute to pump blockage. Construction from rock would also cause social disruption to the township of The Entrance due to the need to transport large volumes of rock to the site. Council favoured a structure composed of joined geotextile tubes as they were considered to be aesthetically pleasing, and importantly, the tubes could be readily removed if changing circumstances warranted removal of the structure. Rock was deemed to be permanent and difficult to remove. While the Department does not necessarily endorse these views they were adopted as terms of reference to the concept design.

The design that has been devised is shown on the Summary Figure hereunder. It comprises two revetments : an eastern wall and a western wall. The western wall is of smaller profile than the eastern as it is located where the rock shelf is close to the surface. The former is comprised of seven sand filled geotextile tubes each 1.3 m diameter, while the eastern wall has three layers of tubes *(12 tubes in the total cross section)*. The thick geotextile material has been used in a similar position on the Gold Coast. It would be treated to reduce the impact of vandalism.

The orientation and arrangement of the walls and their crest heights were selected to optimise the recreational amenity of the area in terms of preserving a large sandy area on the southern side which is heavily used for recreation and in minimising obstruction of water views. The area between the two walls is intended to provide a protected beach area. The location of the walls is as far south as practicable to favour natural sand flushing processes, while at the same time ensuring the depth to bedrock is at least 6 m below Australian Height Datum *(which is close to mean sea level)* in case installation of jet pumps is decided upon at some time in the future.

The life of the wall is uncertain as this is a relatively novel technology. The report estimates a service life of 10 to 15 years and an initial construction cost of $580,000. If the concept is accepted by the local community the next stage will be determination of geotechnical parameters which will allow a stability analysis to be completed and final design to proceed. The question of ensuring stability of the structure against undermining by flood scour is part of the final design procedure. Until all stability requirements are known the cost estimate is provisional, the preconstruction cost estimate may be higher than the present estimate.



RESTRAINING WALL CONCEPT
SUMMARY FIGURE

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1. INTRODUCTION

1.1 BACKGROUND

The Tuggerah Lakes system is located on the Central Coast of NSW approximately 100 km by road north of Sydney. The entrance to the lakes lies immediately north of The Entrance *(refer* **Figure 1.1***)*.

In recent years closures of the entrance channel to the Tuggerah Lakes System have become more common and persistent. This has been of considerable concern to Wyong Shire Council who have responsibility for management of the lake system. Council is anxious to create a permanent tidal entrance so as 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.

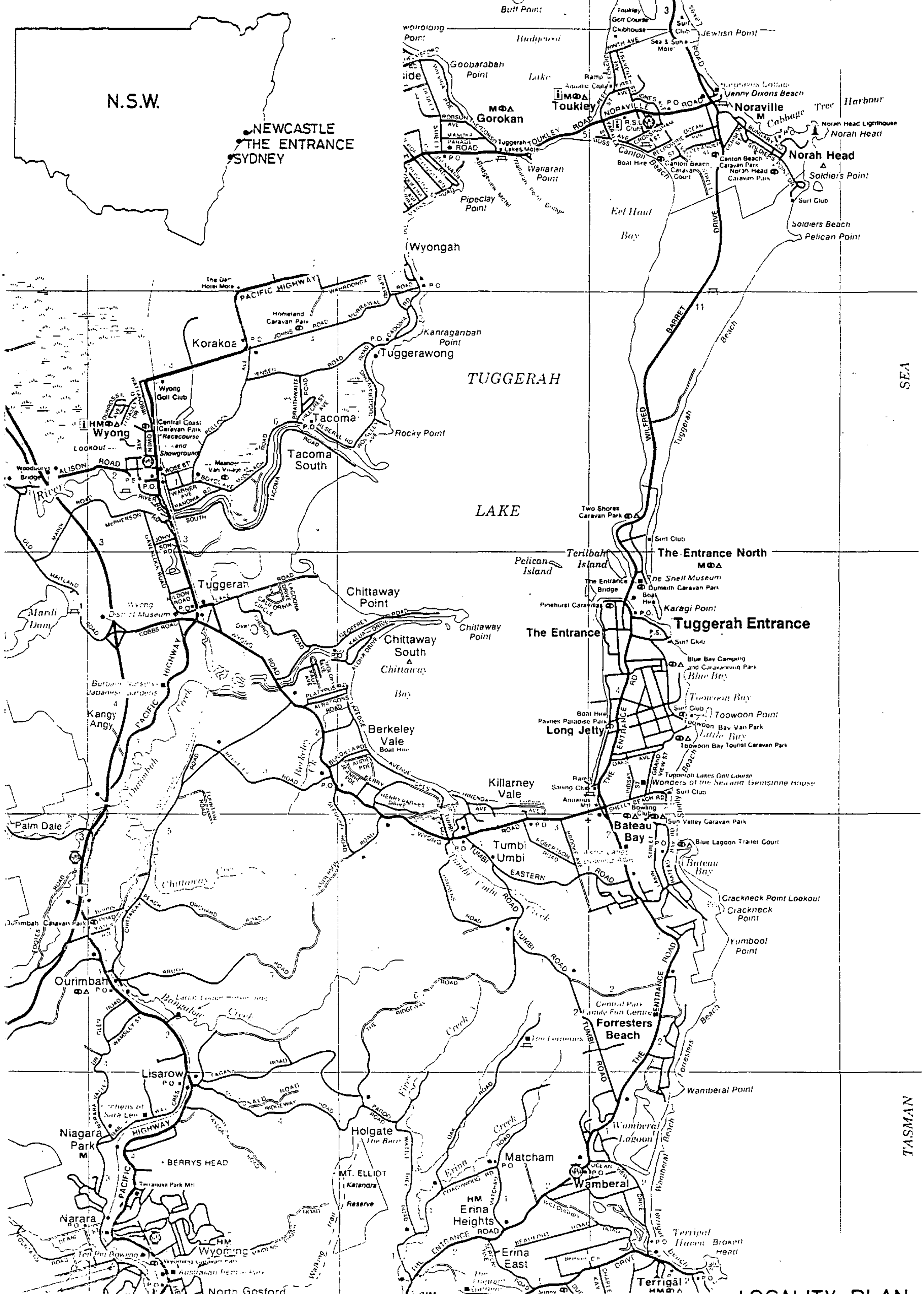
Council do not favour a trained entrance formed by two training breakwaters because of cost, aesthetic impact, reduction in sandy foreshores, and the "inflexibility" of the works should problems occur after construction such as unanticipated shoaling. Accordingly, Council approached Kirrawee Engineering Services Pty Ltd to devise a scheme whereby sand would be pumped away from the entrance channel using submerged jet pumps.

Council subsequently requested that the Public Works Department assess the technical viability of the proposed jet pump scheme. The findings of the Department's assessment were set out in the Department's Report No. PWD 87051 dated October 1987 *(***Reference 1***)*.

The Department found, inter alia, that the installation of jet pumps alone would not maintain the location of an untrained tidal inlet channel. This was due to the potential for outflanking of the jet pumps through channel migration, hence rendering the pumps ineffective. It was considered that to overcome this problem, some form of restraining wall*(s)* would be required to fix the channel location above the jet pumps. The preferred conceptual arrangement for the jet pumps and restraining wall in Report No. PWD 87051 is shown in **Figure 1.2**, and is discussed in more detail in the body of the report.

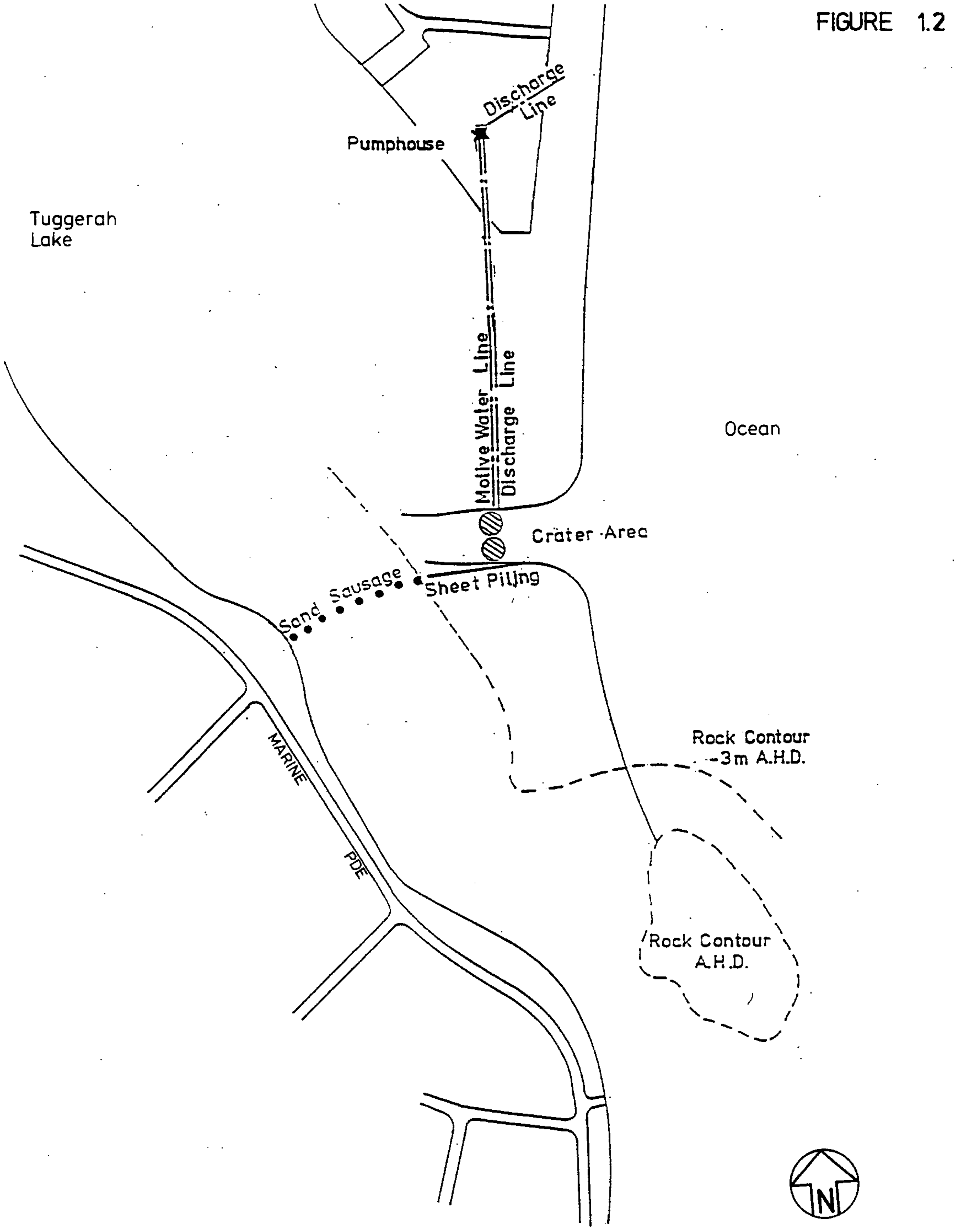
In July 1988 the Public Works Department engaged Patterson Britton & Partners, Consulting Engineers, to provide engineering advice for the design and construction of the restraining wall. The design and installation of the jet pumps was to be deferred pending an evaluation of the performance of the restraining wall alone in maintaining a permanent entrance to the Lake system.

FIGURE 1·1



LOCALITY PLAN

FIGURE 1.2



SOURCE : PUBLIC WORKS DEPARTMENT REPORT Nº PWD 87051
(REFERENCE 1)

PREFERRED CONCEPT FOR
JET PUMPS AND RESTRAINING
WALL IN REPORT Nº PWD 87051

Patterson Britton & Partners engaged Mr A Jackson, Special Projects Engineer with the Gold Coast City Council in Queensland, as a sub-consultant to assist in providing advice on the design and construction of sand filled geotextile tubes which were under consideration as one possible form of construction material.

1.2 SCOPE OF WORK

The work was to be undertaken in three separate stages as follows:

Stage I	-	investigation and concept design
Stage II	-	detailed design and tender documentation
Stage III	-	advice during construction phase.

This report sets out the findings of Stage I of the work. The scope of work for this stage, as set out in the commissioning letter, was as follows:

- review existing available information and carry out a site inspection;

- inspect similar structures recently built in Queensland by the Gold Coast City Council, in particular structures involving use of sand filled geotextile fabrics;

- establish design criteria including:

  - location of restraining wall;
  - alignment of wall;
  - length of wall;
  - design wave height*(s)*;
  - design current velocities;
  - design crest level;
  - design flood scour;

  (The Department will undertake a concurrent photogrammetric analysis to assist in establishing the optimum seaward chainage of the wall);

- specify any geotechnical investigation requirements along the proposed wall alignment *(ie for purposes of Stage II)*;

- identify the alternate design solutions *(structural systems and materials of construction)* which meet the design objectives and design criteria, and prepare preliminary designs, sketch plans and estimates of cost;

- review and discuss the alternative design solutions with officers of the Department and Council, and select a preferred option;

- prepare a budget construction cost estimate and general arrangement drawing of the preferred design solution including site plan of structure showing principal dimensions, sections and elevations and construction materials;

- document the above work in a report *(of approximately 20 pages including figures and drawings)* and submit the report to the Department and Council for formal approval of the preferred design solution;

- consider public safety including safe pedestrian access to the structure, access to wadable water, egress from the channel by the provision of suitably spaced ladders and retention of water views from sunbathing areas, which cater for child supervision for instance;

- in addition to the above, the following aspects should also be considered and addressed:
  - model testing will not be necessary for Stage I;
  - rock should not be considered when selecting the external *(visible)* surface of the restraining wall. Materials such as vegetated sand, geotextiles, concrete sheet piling or any combination of these, may be considered;
  - consideration should also be given as to the extent to which the wall needs to be overtopped by flooding. Could crest height vary along the length allowing partial overtopping? If so, could the landward end consist of stabilized sand not to be overtopped?

The work did not include an assessment of the impact of the proposed restraining wall construction on existing estuary and coastal processes.

1.3 LEVEL DATUM

All levels and channel depths in the report are relative to Australian Height Datum *(AHD)* unless noted otherwise. AHD is approximately Mean Sea Level *(MSL)*. Normal lake water level is perched about 0.2 m above MSL.

2. SITE CONDITIONS

2.1 GENERAL

The characteristics of the entrance to the Tuggerah Lakes system have been described in some detail in the Public Works Department's October 1987 report *(***Reference 1***)*, and it is not intended to restate all of this information in this report.

The following sections set out a description of selected site conditions in so far as they affect the design of the restraining wall.

2.2 GROUND LEVELS

The entrance region of the lakes system typically comprises mobile shoals, a migratory tidal channel*(s)*, beach berm areas and low dunes. The arrangement of these features is markedly altered at times of flooding when a wide channel *(up to several hundred metres in width)* is scoured to the sea.

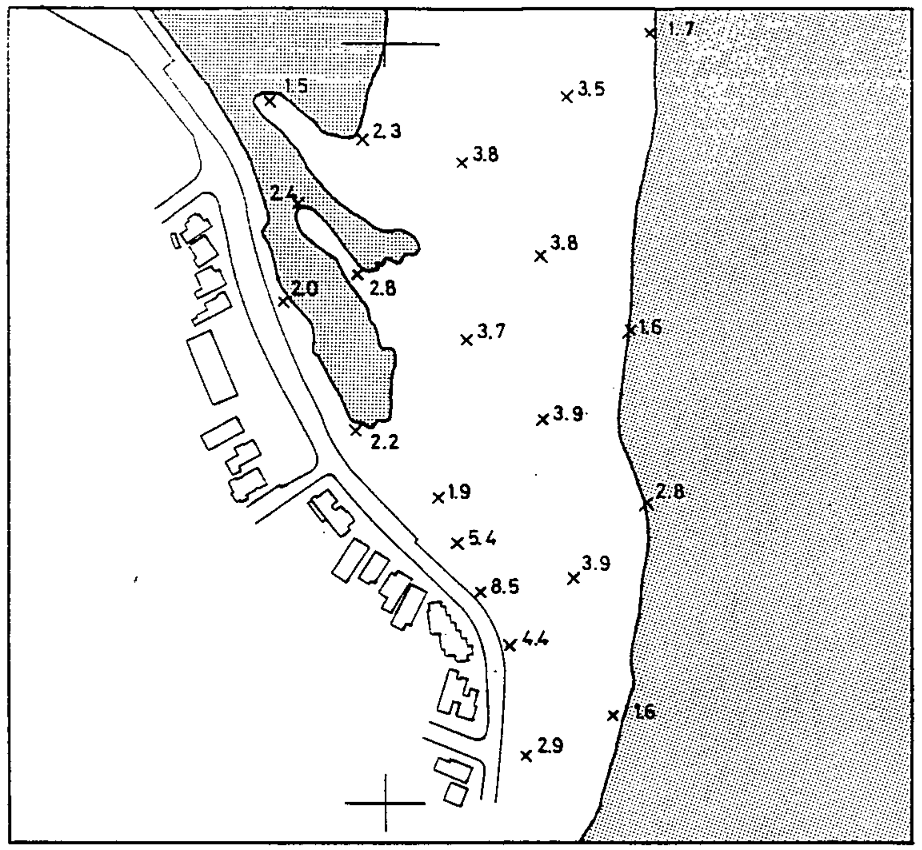
The surface level of the beach berm areas and dunes which form naturally in the entrance region, particularly in the area south of the proposed restraining wall, is of interest in view of the need to establish a design crest level for the wall and, from a water safety viewpoint, retain water views from sunbathing areas.

As part of the current work, the Public Works Department has undertaken a photogrammetric assessment of four separate years of selected vertical aerial photography of the entrance region. Details of the photography are listed in **Table 2.1**.

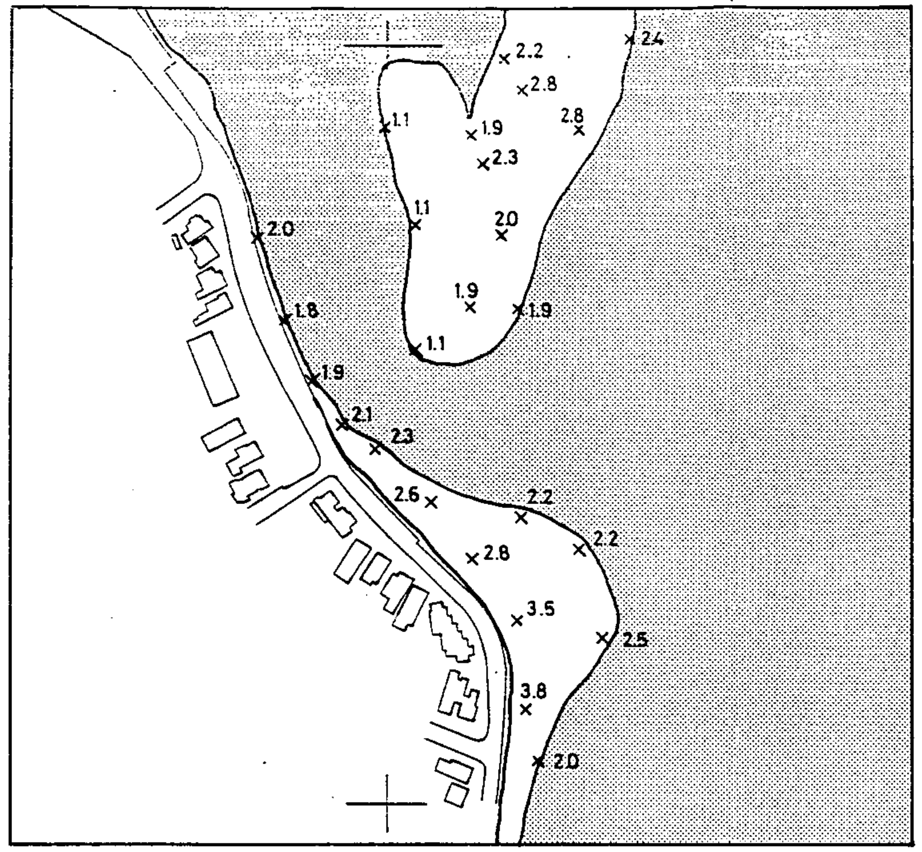
The photogrammetric analysis involved mainly plotting of natural and cultural features which were clearly visible in the photography *(channels, shoals, wave fronts, rock exposures, shorelines, roads, etc)* and observing of ground spot heights in the beach and dunal areas.

The results of the observations of spot heights in the area south of the proposed restraining wall are shown in **Figure 2.1**. These results indicate that when the entrance is fully closed for an extended period *(eg. 25/11/41 photography)* sand levels to the south of the proposed restraining wall can build up to values in excess of 3.5 m AHD. Typically, when the channel is open *(eg. 18/8/65 and 6/2/73 photography)*, sand levels can build up to about 2.2 m AHD or higher.

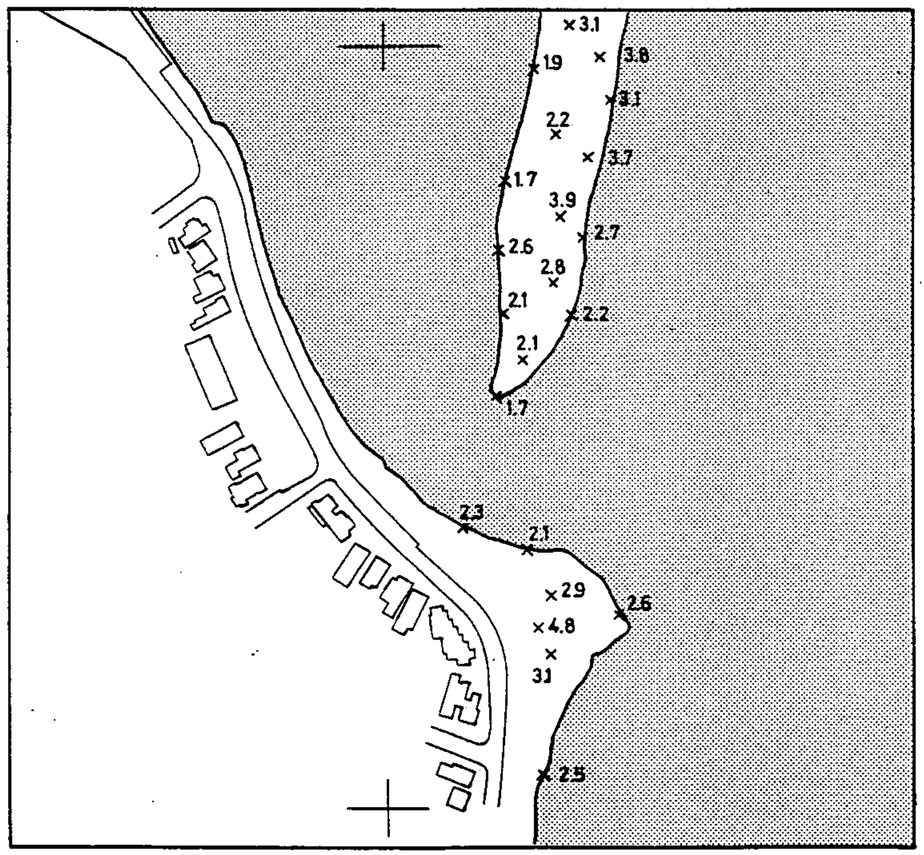
FIGURE 2.1



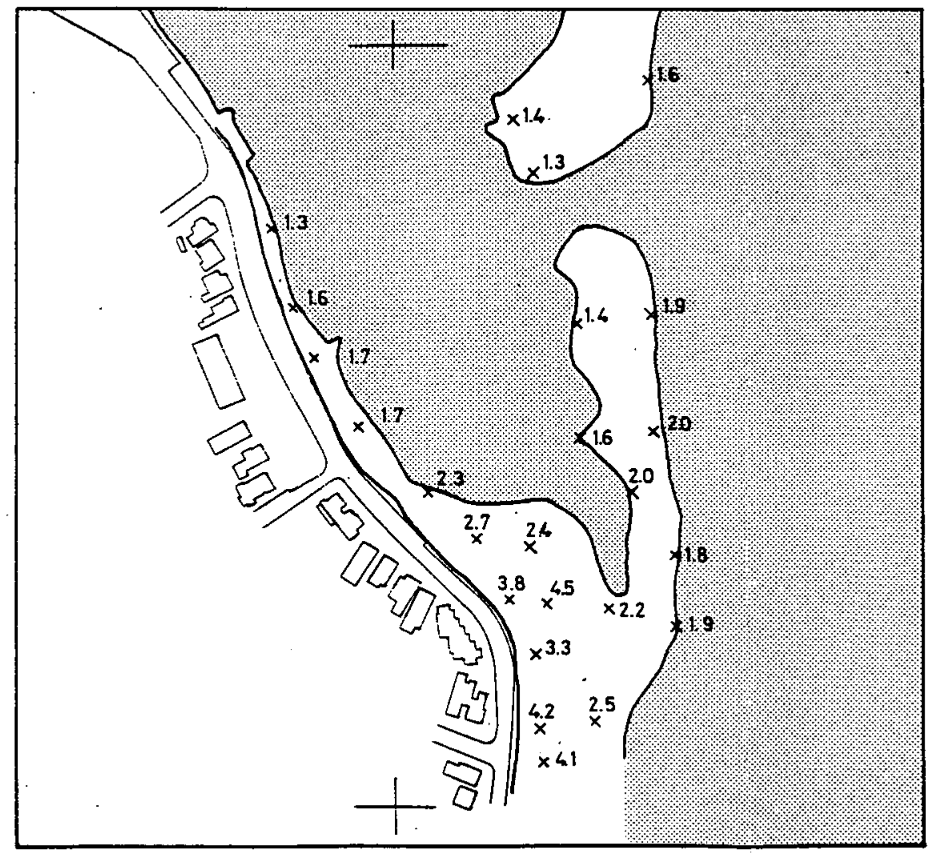
25.11.1941



18.8.1965



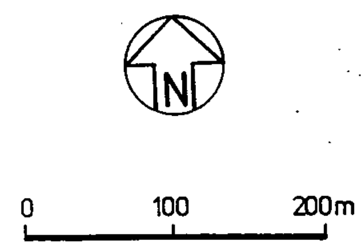
6.2.1973



19.5.1986

NOTES:

1. ALL SPOT HEIGHTS ARE IN METRES RELATIVE TO AHD.
2. THE WATER LINE SHOWN IS THAT VISIBLE AT THE TIME OF THE PARTICULAR PHOTOGRAPHY.
3. HOUSING AND ROADS ARE FROM 1986 PHOTOGRAPHY.



SPOT HEIGHTS FROM PHOTOGRAMMETRIC ANALYSIS

TABLE 2.1 PHOTOGRAPHY SELECTED FOR PHOTOGRAMMETRIC ANALYSIS

Date	Film No.	Focal Length *(mm)*	Approx. Scale	Entrance Condition
25/11/41	-	-	1:16,000	Fully closed
18/8/65	NSW 916	114.44	1:16,000	Open - against southern shore
6/2/73	NSW 2130	152.36	1:16,000	As above
19/5/86	NSW 3519	153.10	1: 8,000	Almost closed

2.3 CHANNEL DEPTHS

Two hydrographic surveys of the entrance region are available:

- a survey undertaken by the Public Works Department during the period March 1975 to September 1976 as part of an overall hydrographic survey of the Tuggerah Lakes system *(PWD Plan Catalogue No. 4401)*;

- a survey of the entrance region undertaken by the Public Works Department in August 1979 *(PWD Plan Catalogue No. 5746)*.

In both surveys the entrance channel was open and constrained against the southern shoreline *(refer* **Figure 2.2***)*. The width of the entrance channel measured between 0 m AHD in the throat area was approximately 55 m in the earlier survey and 50 m in the latter survey. These values are typical of widths in non-flood conditions measured from aerial photography over the past 30 - 40 years (**Reference 1**). Accordingly the depths in the throat area at the times of the surveys may be taken as a reasonable indication of the typical channel depths which exist under tidal conditions when the channel is constrained on its southern side *(in this case by the natural rocky southern shoreline)*. **Figure 2.2** indicates these depths have a maximum value of approximately 1.5 - 2.0 m below AHD.

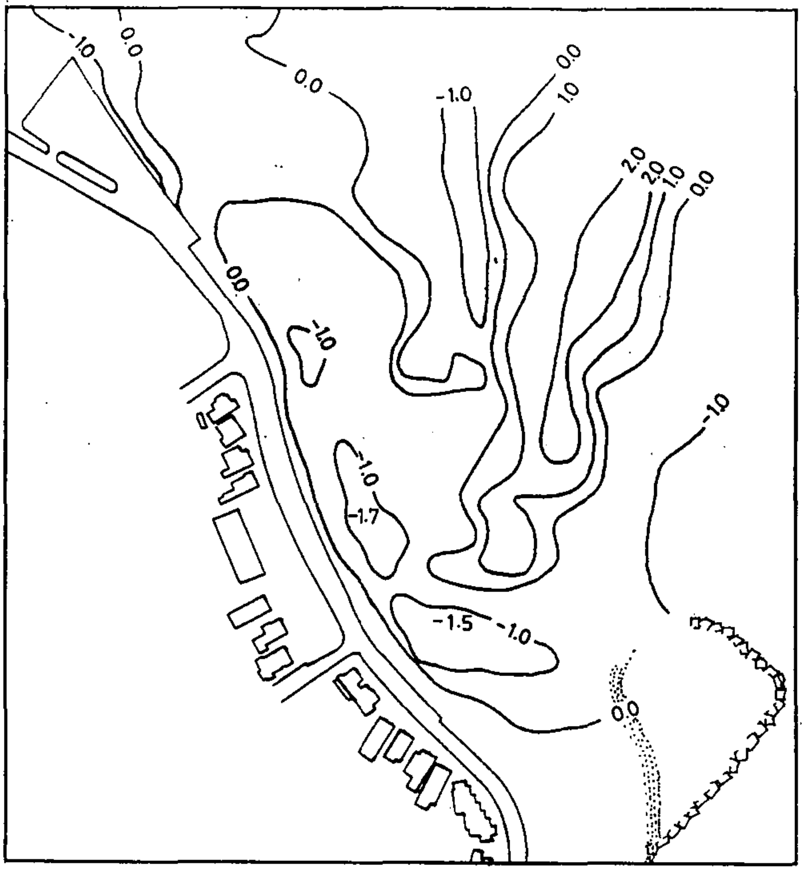
At times of flooding, increased channel depths would be expected adjacent to the southern rocky shoreline as a result of scour but no measurements are available. The scour which could occur against the proposed restraining wall is discussed in **Section 3.1.**

2.4 GEOTECHNICAL CONDITIONS

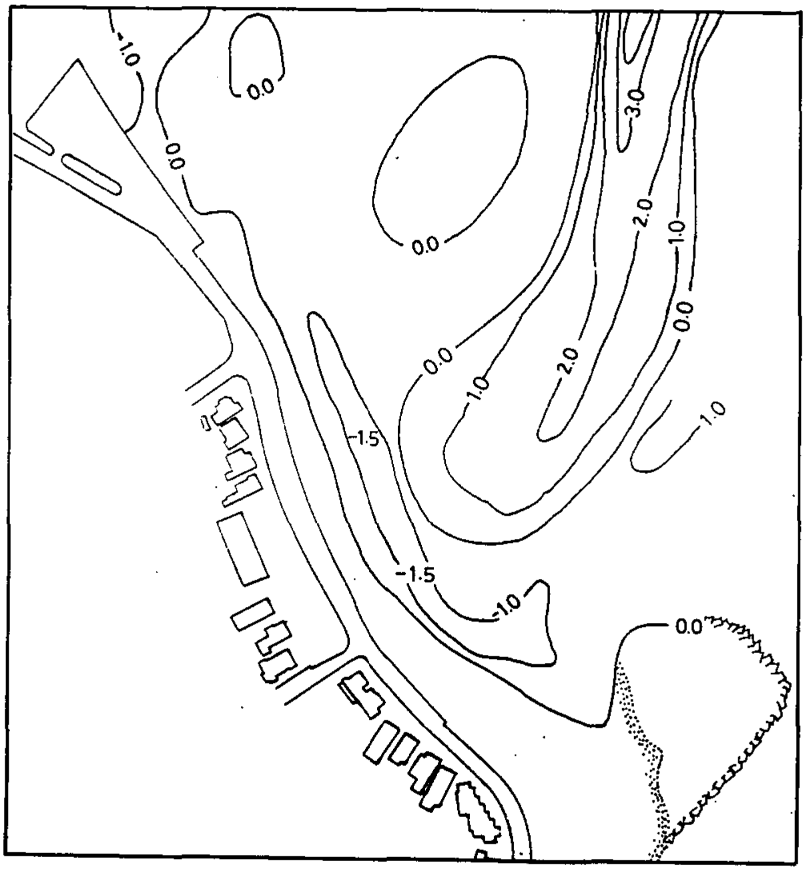
Geotechnical information in the entrance region is restricted to rock levels established by the Public Works Department as part of the Inter-Departmental Committee Tuggerah Lakes Study Report published in 1979 (**Reference 2**), supplemented by additional data gathered jointly by the Public Works Department and Wyong Shire Council in early 1988.

The rock levels established in the entrance region are shown in **Figure 2.3**. There is a large area of rock shelf above 0 m AHD at the southern limit of the entrance region which acts as a control in tidal flows as described in **Reference 1**. Further upstream of this rock shelf, the rock contours are generally parallel to the embankment along Marine Parade with the -3 m AHD contour typically located 120 - 140 m from the top of the embankment. The rock surface appears to drop in level relatively quickly in the area northward of the rock shelf and eastward of the -3 m AHD contour.

FIGURE 2.2



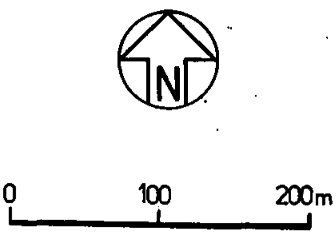
1975 / 76 SURVEY



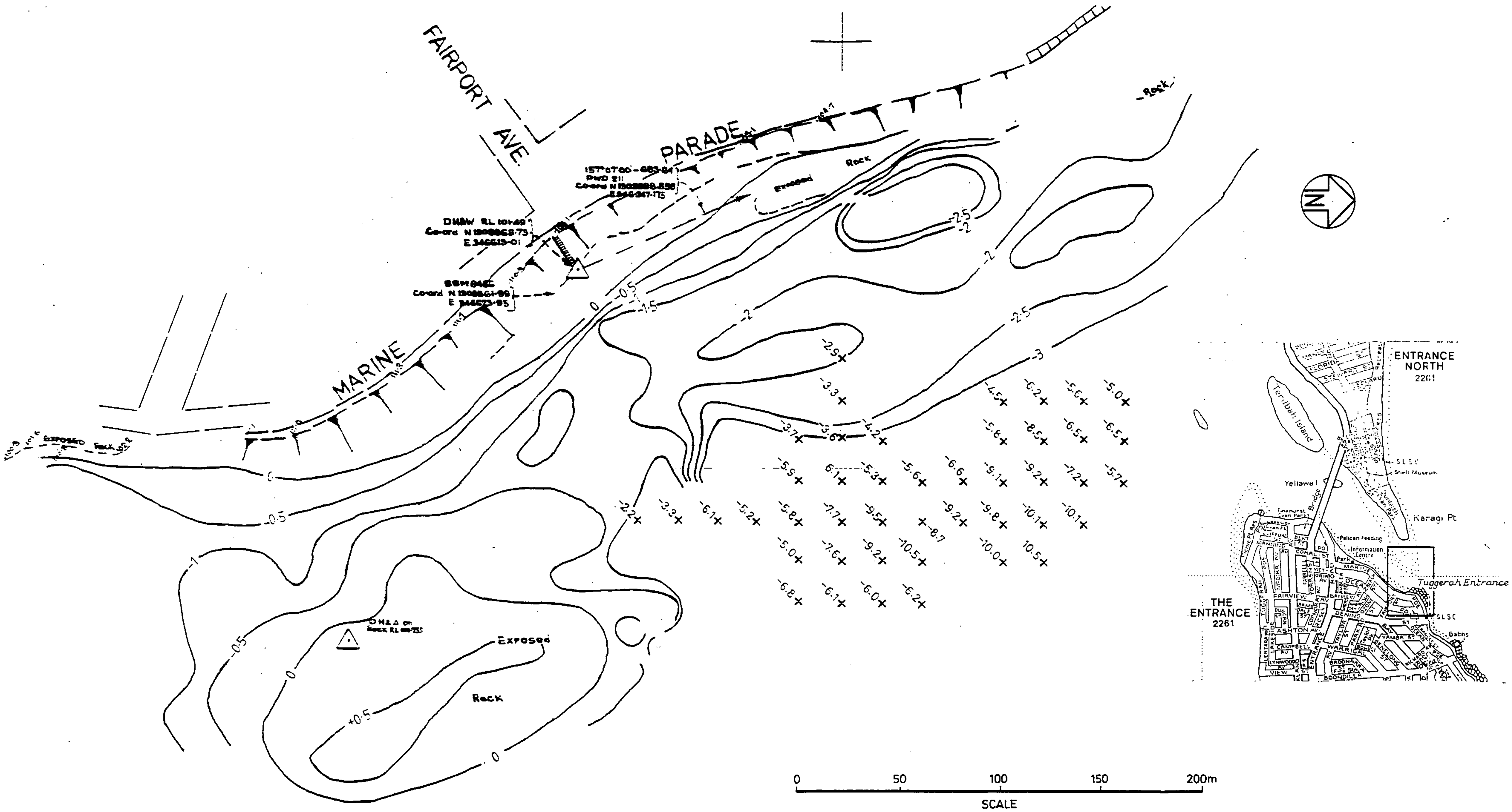
1979 SURVEY

NOTES:

1. CONTOURS AND SELECTED SPOT DEPTHS ARE IN METRES RELATIVE TO AHD.



HYDROGRAPHIC SURVEY IN ENTRANCE REGION - 1975 / 76 AND 1979



ROCK CONTOURS ARE FROM 1979 IDC REPORT (REFERENCE 2)

-5.9+ ROCK LEVEL FROM JOINT PWD / COUNCIL INVESTIGATION EARLY 1988

ROCK LEVELS IN THE ENTRANCE REGION

2.5 TIDAL ACTION

The tidal planes in the ocean at the entrance to the Tuggerah Lakes system would be virtually equivalent to those at Fort Denison in Sydney Harbour. The tidal planes at Fort Denison are summarised in **Table 2.2**. Tides are semi-diurnal with significant diurnal inequality. The tidal range between MHHW and MLLW is 1.3 m.

TABLE 2.2 TIDAL PLANES AT FORT DENISON

Tidal Plane	Level Relative to Hydrographic Datum	Level Relative to AHD
Highest Astronomical Tide *(HAT)*	2.0	1.1
Mean Higher High Water *(MHHW)*	1.6	0.7
Mean Sea Level *(MSL)*	0.9	0.0
Mean Lower Low Water *(MLLW)*	0.3	-0.6
Lowest Astronomical Tide *(LAT)*	0.0	-0.9
Hydrographic Datum is zero on the Fort Denison Tide Gauge which is approximately Indian Spring Low Water *(ISLW)*. Australian Height Datum *(AHD)* is 0.925 m above Hydrographic Datum.		

Information on tidal levels and flows in the entrance channel is available from two data collection exercises carried out by the Public Works Department Manly Hydraulics Laboratory in October 1975 and May 1976 (**References 3** *and* **4**).

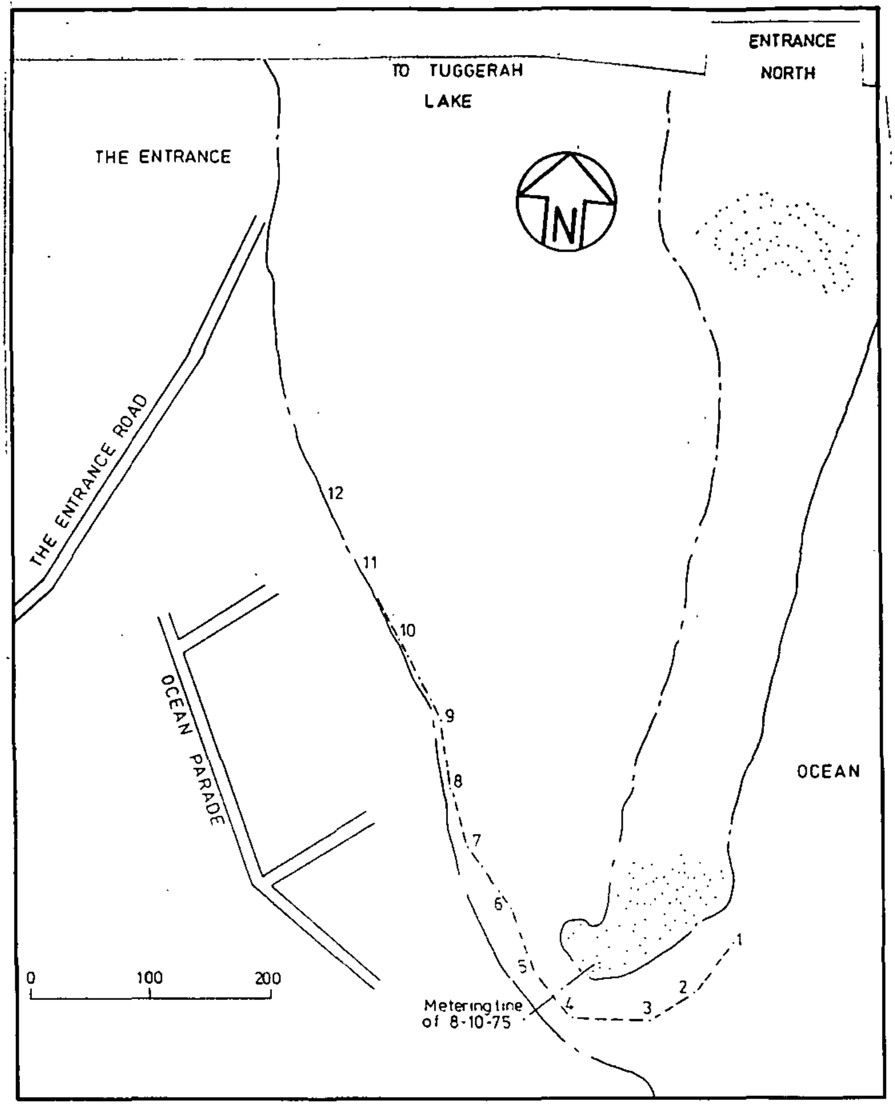
The October 1975 data collection exercise included a metering line across the throat of the tidal channel and peak tidal velocities of 2 m/s were recorded on both the ebb and flood tides for an ocean tidal range of 1.6 m.

The peak tidal gradients measured along the entrance channel during the October 1975 exercise are shown in **Figure 2.4**. This figure indicates that the majority of head losses along the channel occur within a distance of approximately 200 m from the sea.

2.6 FLOODING

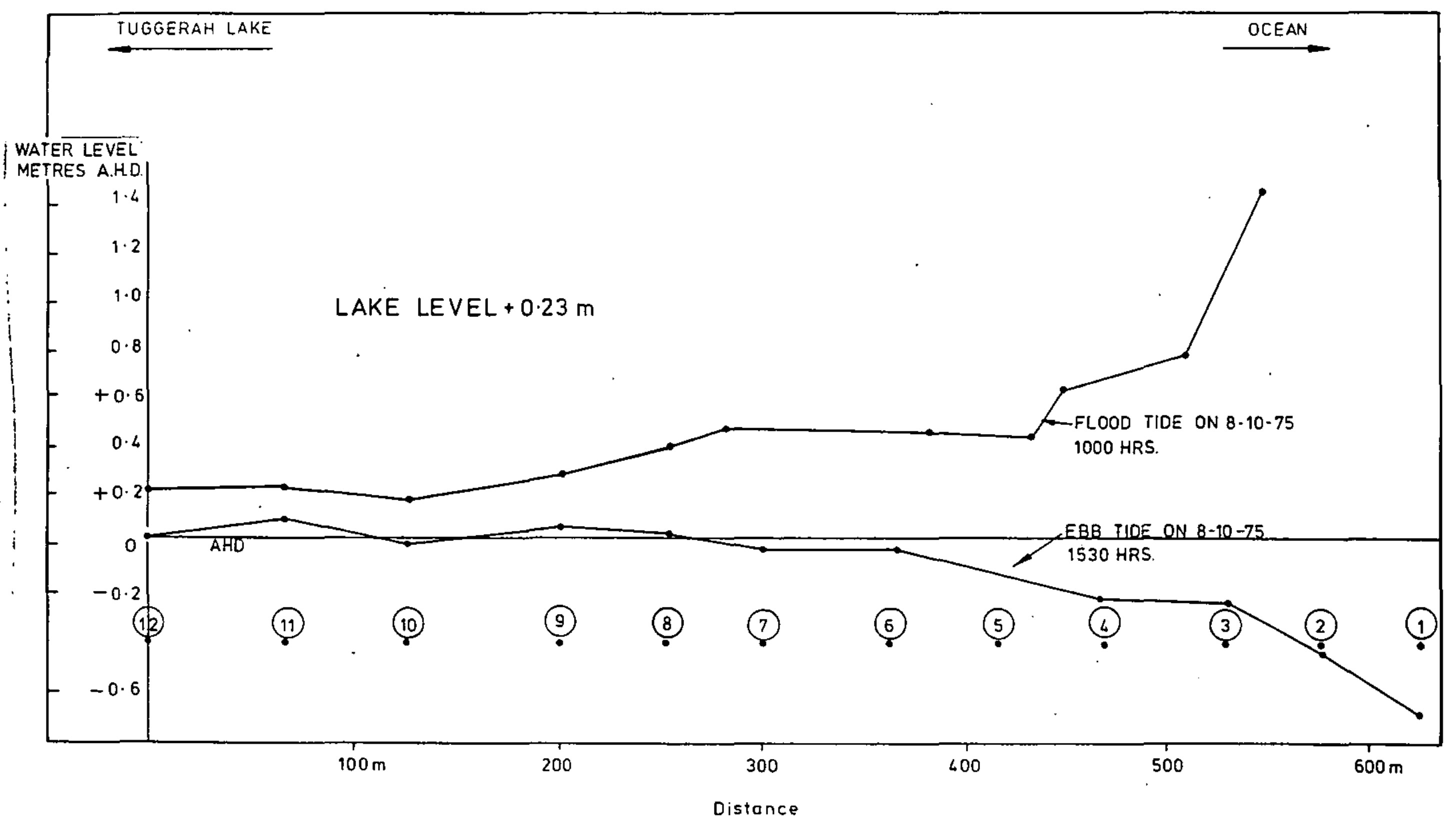
In 1971 the University of New South Wales Water Research Laboratory, on behalf of the Electricity Commission of New South Wales, published a report (**Reference 5**) which set out flood level heights in Tuggerah Lake for a range of return periods. Two methods were used to estimate the flood heights; an analytical rainfall-runoff analysis and a historical analysis of lake levels. The results of the analysis are shown in **Figure 2.5**.

FIGURE 2·4



LOCATION PLAN

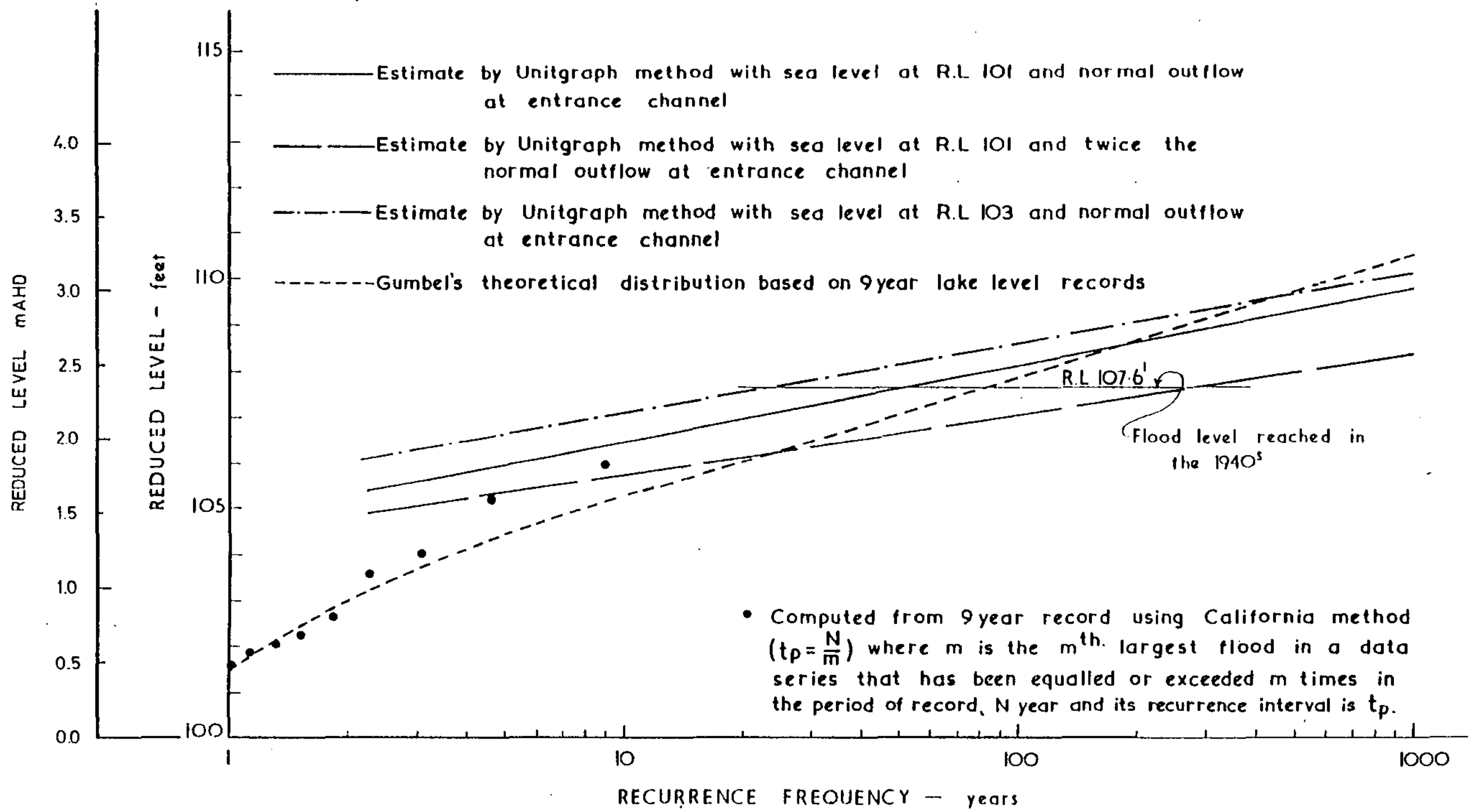
SOURCE : PWD DATA COLLECTION
8/10/75 (REFERENCE 3)



PEAK TIDAL GRADIENTS ALONG
ENTRANCE CHANNEL - 8/10/75

SOURCE : UNSW WRL REPORT Nº 123
(REFERENCE 5)

NOTE: ORIGINAL INFORMATION IS IN IMPERIAL UNITS



NOTE : ORIGINAL INFORMATION IS IN IMPERIAL UNITS

WATER LEVEL - RECURRENCE INTERVAL RELATIONSHIP FOR TUGGERAH LAKE

FIGURE 2.5

Work carried out by Sinclair Knight & Partners in 1974 *(***Reference 6***)*, extended the earlier work of the Water Research Laboratory by including six more years of data in the frequency analysis of historical records. The results of this analysis are set out below:

Recurrence Interval *(years)*	**Lake Level** *(m AHD)*
20	1.9
50	2.2
100	2.4

It is understood that the above values have been adopted by the Public Works Department and Council for control of development with lake frontage. It should be noted that the levels assume a continuation of the historical *(pre 1974)* operation and behaviour of the channel outlet. A later report prepared in 1984 by Sinclair Knight & Partners on behalf of the Public Works Department *(***Reference 7***)* pointed out that peak lake levels resulting from low to medium recurrence interval river floods are particularly sensitive to the formation or opening of bars across the entrance to the lake. It was therefore suggested that the above flood levels may prove to be overestimates should Council continue their practice adopted in recent years of mechanically opening the entrance when the lake is closed and the water level reaches a predetermined value *(1.5 m AHD, or a level of 1.2 m AHD if the water has risen 0.2 m in the last six hours)*.

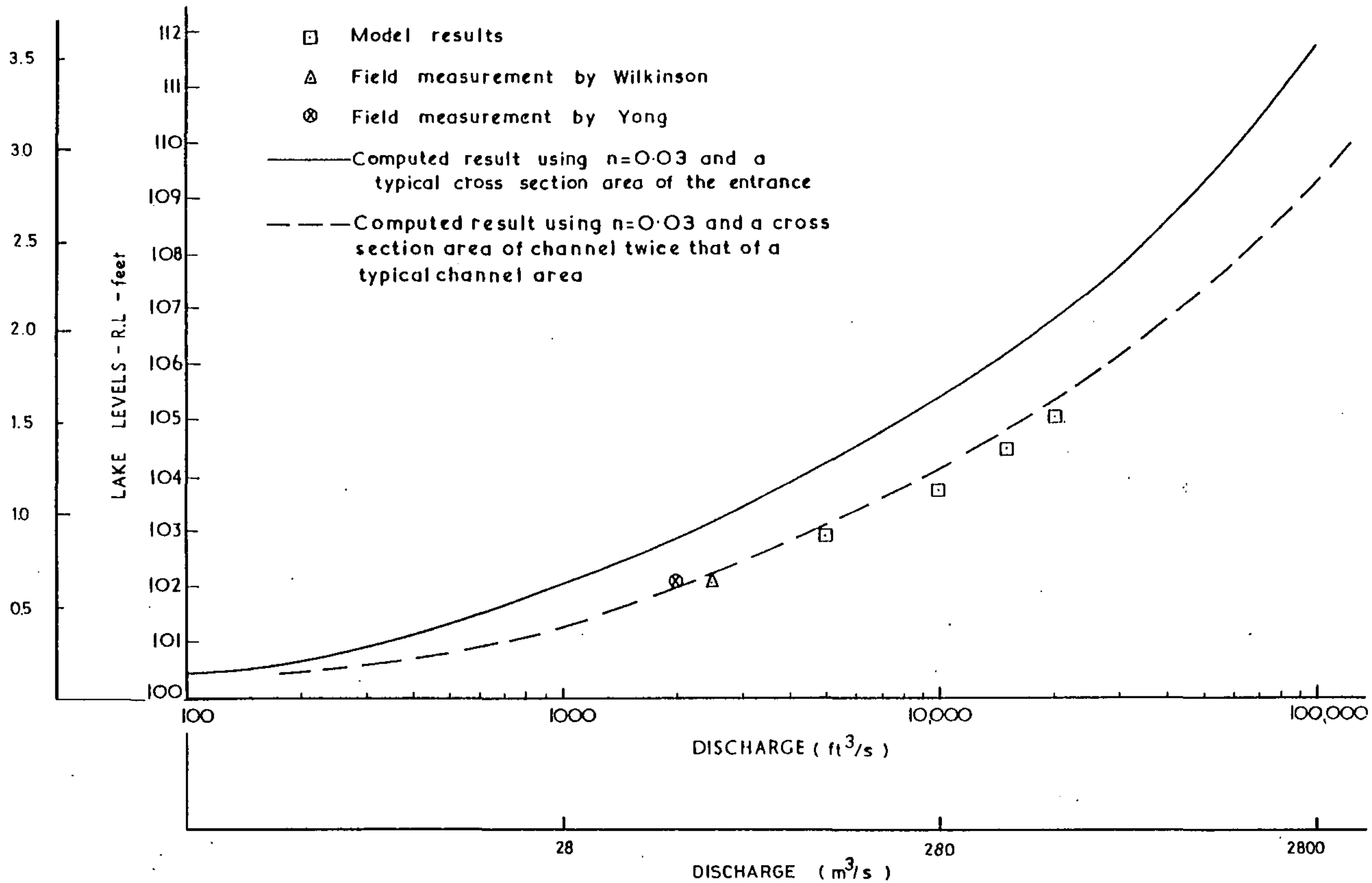
The work by the Water Research Laboratory also included an investigation of the relationship between flood discharge in the entrance channel and scour. This was achieved by use of a small scale mobile bed model.

The pre-scour condition was based on a survey carried out by Laboratory staff. At the time of the survey the entrance was open with the channel characteristically situated adjacent to the southern shoreline before flowing to sea in a north-easterly direction behind the exposed rock shelf. The width of the channel in the throat area, measured between the 0 m AHD contours, was approximately 25 m. The limits on scour due to bedrock control were established from the results of a single south-north seismic run across the entrance.

The model tests consisted of running a steady discharge through the model and measuring lake level, waterway cross-sectional area and typical waterway top width in the channel. The discharges adopted were approximately 140, 280, 420 and 560 m^3/s. In all cases the tailwater level was set constant at Mean Sea Level.

The model testing indicated that for discharges of 280 to 560 m^3/s, after initial scour, the waterway areas stabilised at just below twice the pre-scour values. Based on interpretation of the information presented in the report, it is estimated that, <u>on average</u>, the bed of the channel scoured by an amount of 1.5 to 2.0 m, from a level of about -0.5 m AHD down to a maximum level of about -2.5 m AHD.

Figure 2.6 shows the comparison in the Water Research Laboratory report between field measurements, model results and computed values. Based on this comparison, the Laboratory concluded that scour at the entrance would reduce flood levels by as much as 0.3 m below values which would have existed without any scour.



NOTE : ORIGINAL INFORMATION IS IN IMPERIAL UNITS

SOURCE : UNSW WRL REPORT Nº 123 (REFERENCE 5)

NOTE : ORIGINAL INFORMATION IS IN IMPERIAL UNITS

FLOOD DISCHARGE CHARACTERISTICS OF THE ENTRANCE CHANNEL

FIGURE 2.6

2.7 ELEVATED OCEAN WATER LEVEL

At times of storms ocean water levels are superelevated above normal astronomical tidal levels by the effects of wind and pressure surge *(referred to as storm surge)* and wave setup.

The superelevation of the water surface due to storm surge and wave setup has a dual effect. Firstly, owing to the increased water depths, it becomes possible for larger waves to attack the beach and shoreline structures *(inshore wave heights being depth limited)*. Secondly, it is possible for waves to break higher on the beach profile, thereby further increasing the erosion potential.

There have been no detailed studies of elevated ocean water levels in the immediate study area. There have, however, been a number of extreme water level studies at selected locations on the NSW open coast which may be used as a guide. **Table 2.3** sets out the findings of several of these studies. Based on the information in this table it is considered reasonable to adopt an extreme elevated ocean water level for the study area of say 2.7 m AHD.

TABLE 2.3 ELEVATED OCEAN WATER LEVELS

Study Area	Elevated Ocean Water Level *(m AHD)**	Source
Collaroy-Narrabeen Beaches	2.5 - 2.8	Reference 8
Avoca Beach	2.9 - 3.1	Reference 9
Wamberal Beach	2.8 - 3.1	Reference 9
Coffs Harbour	2.4	Reference 10

* No return periods are specified for the elevated ocean water levels for Collaroy-Narrabeen Beach, Avoca Beach and Wamberal Beach, however the values are for extreme events and would have a return period of not less than 50 years. The value given for Coffs Harbour corresponds to a return period of 50 years.

2.8 INSHORE WAVE CLIMATE

There have been no detailed studies of inshore wave climate at the entrance region to the Tuggerah Lakes system. However, the findings of coastal engineering studies for Wamberal and Avoca Beaches *(**Reference 9**)* and the Collaroy-Narrabeen embayment *(**Reference 11**)*, both of which involved "surf zone" modelling, provide a useful guide.

At Wamberal and Avoca Beaches the calculated maximum breaking waves at the back of the beach varied along the beach from values of 3.1 to 3.7 m. The wave period adopted was 12 seconds. The assumed depth of scour for purposes of estimating wave heights varied from -0.25 m AHD up to a maximum of -1.0 m AHD depending on location along the beach. These conditions would be expected to correspond to a return period of not less than about 50 years.

For the Collaroy-Narrabeen embayment, the estimated maximum inshore wave heights for a scour level of -1.0 m AHD *(typical maximum value)* varied between 2.8 and 3.3 m. These values corresponded to a design offshore significant wave height of 10 m, which, assuming a duration of four hours, would have a return period of about 50 years. There are no wave periods nominated but values would be expected to be in the order of 12 seconds.

At the entrance to the Tuggerah Lakes system, the reef areas which are known to be offshore could cause some reduction in the inshore wave climate, particularly for waves approaching from the southern sectors. However, in view of the absence of detailed site specific information no reduction of the above-mentioned design wave heights established at other beaches is considered justified.

For purposes of the concept design, it is considered reasonable to adopt an inshore wave height of, say 3.5 m, and wave period approximately 12 seconds.

3. CONCEPT DESIGN

3.1 MAIN DESIGN CRITERIA

3.1.1 Design Life

The technical brief for the concept design did not indicate a "design life" for the restraining wall, however it is clear the structure is intended to be permanent. Although the consequences of failure of the wall would not be catastrophic, it is considered prudent to design the structure to resist a design event*(s)* having a return period of not less than 30 to 50 years.

This is not to say that the structure may not need repair or even replacement during this time frame as a result of deterioration from natural wear and tear or for other reasons related to the type of construction materials.

3.1.2 Location of the Seaward End of the Restraining Wall

Location Along the Beach

In the preferred conceptual arrangement for the restraining wall and jet pumps shown in the Department's Report No. PWD 87051 *(refer* **Figure 1.2***)*, the location along the beach for the seaward end of the wall was selected on the basis of ensuring a depth to rock of nominally -10 m AHD. This depth was necessary in order that the preferred "two pump" jet pump installation could achieve the desired crater width of 40 m, assuming a crater side slope of 1:1 or slightly flatter. The "two pump" installation was preferred to a "three pump" installation since the former would create a greater width in the direction of the tidal flow, ie perpendicular to the beach line, which would be more effective in trapping suspended sand.

For a number of reasons it is considered preferable to locate the entrance restraining wall further south than that shown in **Figure 1.2**, while not ruling out the possibility of introducing jet pumps *(probably more than two)* at a later date. These reasons are as follows:

- the natural behaviour of the entrance channel during floods is to shift southwards against the exposed rock shelf - the more the restraining wall can be designed so as to be "sympathetic" with the natural flood flow behaviour, the less likely there is to be adverse hydraulic and scour impacts;

- the higher rock levels to the south provide greater insurance against potential undermining of the restraining wall should scour levels be greater than predicted;

- the entrance channel would be located closer to the existing road and carpark area, hence walking distances would be less producing some amenity benefits;

- a significant mechanism for closure of the channel following flood scour is an alongshore supply of sand from the north, hence locating the channel further south may assist in achieving a more permanent entrance;

- location of the wall further to the south would minimise the erosion of the stabilised dunes on the northern side of the entrance at times of floods.

In order not to compromise the possible later introduction of a jet pump system which could achieve economically a suitable crater width *(say 40 m)*, the wall should not be moved further south than a position corresponding to rock levels of say -6 to -7 m AHD. This would be about 100 m further south than the position of the restraining wall shown in **Figure 1.2**.

Location Relative to the Beach Line

The Department's Report No. PWD 87051 *(***Reference 1***)* sets out the criteria for establishing the position of the seaward end of the restraining wall relative to the beach line; namely, that the wall should extend out to high water mark determined as an average location under full beach accretion conditions.

The results of both the Department's photogrammetric analysis *(refer* **Section 2.2***)* and the hydrographic surveys carried out in the period March 1975 - September 1976 and in August 1979 have been used to estimate the average location of high water mark under beach full conditions. This information, together with the desire to locate the wall as far south as possible in rock levels of -6 to -7 m AHD, essentially fixes the location of the seaward end of the restraining wall, as shown in **Figure 3.1**.

3.1.3 Alignment of the Restraining Wall

The alignment of the restraining wall is influenced by a number of factors:

- at the seaward end of the wall it is desirable to align the wall directly into the oncoming waves, ie perpendicular to the direction of wave approach. This is in order to minimise the degree of oblique wave attack thus producing a more efficient and inherently stable design.

  An indication of the possible range of wave approach directions has been established from the available photography;

- the alignment should encourage streamlined flows, particularly at times of flooding, in order to minimise any potential adverse hydraulic and scour impacts;

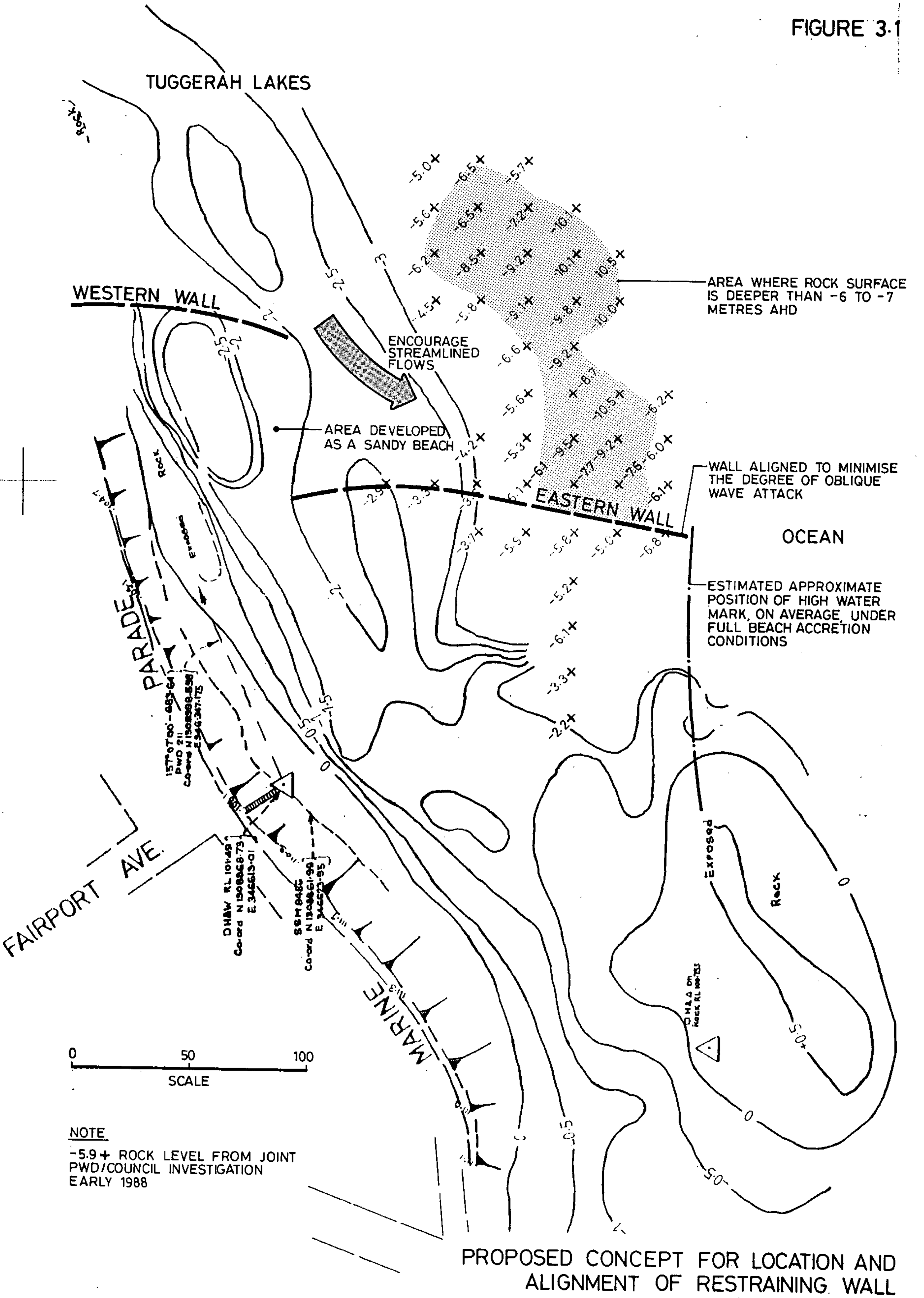
- the wall alignment should conform with, and enhance, the natural scour protection already provided by bedrock, but not constrain the channel so that flows are directed to areas where rock is higher than say -2 m AHD *(otherwise flow depths will be unnecessarily restricted leading to possible navigation and head loss problems)*;

- the alignment of the wall should be such as to encourage access to wadable water and permit retention of water views from sunbathing areas, etc as required by the brief *(refer* **Section 1.2***)*;

- consistent with achievement of the design objectives, the wall should be of minimum length *(particularly in those areas where the height of the wall and/or scour protection requirements are greatest)*, so as to minimise costs.

Taking into account the above factors, the alignment proposed for the wall is shown in **Figure 3.1**. The wall is in fact proposed to comprise two separate structures, a western wall and eastern wall. This arrangement is considered to have two major advantages:

FIGURE 3.1



PROPOSED CONCEPT FOR LOCATION AND ALIGNMENT OF RESTRAINING WALL

- the area between the two walls could be developed as a sandy beach and hence offer significant waterway access, amenity, and aesthetic benefits, as compared to a continuous wall design;
- the shorter western wall provides an opportunity to evaluate alternative construction techniques in a relatively sheltered environment before commencing construction on the more exposed eastern wall.

The overall length of the proposed "twin-wall" design is only marginally greater than the Department's original continuous wall design *(about 6% increase in overall length)*. Any potential increase in cost would be less than this relative increase in wall length since the length increase occurs in the lesser unit cost per metre run section of wall, ie the inner section where crest levels and scour allowances will be reduced. The ability to refine construction techniques during the construction of the western wall may in fact provide some saving in total cost compared to the original continuous wall design.

3.1.4 Currents

Two types of currents can be distinguished; tidal currents, and currents due to flooding.

Tidal Currents

It is considered reasonable to adopt a design value for tidal currents adjacent to the restraining wall equal to the peak ebb and flood tide currents of 2 m/s measured in the throat of the naturally constrained channel during the October 1975 data collection exercise *(refer* **Section 2.5***)*.

Currents due to Flooding

There are no measurements for the magnitude of currents in the entrance channel at the time of flooding.

The discharge values modelled by the Water Research Laboratory as part of the small scale mobile bed modelling *(range 140 to 560 m^3/s)* are below the discharges which would be expected in a 30 to 50 year return period flood event *(about 1000 to 1300 m^3/s based on the information in* **Figures 2.5** *and* **2.6***)*. The smaller discharge values resulted in estimated average current speeds of about 2.0 m/s.

Other relevant information available on likely current speeds in the entrance channel is available from studies of the Shoalhaven River entrance *(***Reference 12***)*, Dee Why Lagoon *(***Reference 13***)* and Avoca Lake *(***Reference 14***)*.

For the Shoalhaven River, it was estimated that an average velocity of about 2.5 m/s would develop at an unrestricted entrance at Shoalhaven Heads *(ie free to erode)*. This value was used to estimate the entrance flow area for a design flood having a return period of 30 years.

At Dee Why Lagoon during the so-called "weir phase" of the breakout flow, namely the period of upstream flow control via critical flow over a mobile weir feature, average velocities of between 2 and 4 m/s were estimated. The higher velocities *(peak 4.2 m/s)* occurred at a time when a consolidated clay substrate limited further bed erosion.

At Avoca Lake, a simple mathematical model was established to simulate the breakout flow and entrance channel erosion. The results of the simulations compared favourably with measured final dimensions of the breakout channel and the actual duration of the "weir phase" of the flow. Predicted maximum average velocities in the breakout flow were about 3.0 m/s.

Based on the above, it is considered reasonable to adopt a design value of 3.0 m/s for the average current speed during a flood event having a return period of 30 to 50 years.

3.1.5 Wave Height and Period

From the discussion in **Section 2.8**, it is considered reasonable for concept design purposes to adopt an inshore wave height of 3.5 m and wave period of approximately 12 seconds.

3.1.6 Crest Level

The crest levels to be adopted for the restraining wall are influenced by a number of factors, including:

- flood levels
- elevated ocean water levels
- tidal levels
- wave action
- retention of views from sunbathing areas *(for safety reasons)*
- visual impact generally
- cost.

The significance of some of the above factors differs with position along the wall, and it is convenient to discuss the western and eastern walls separately.

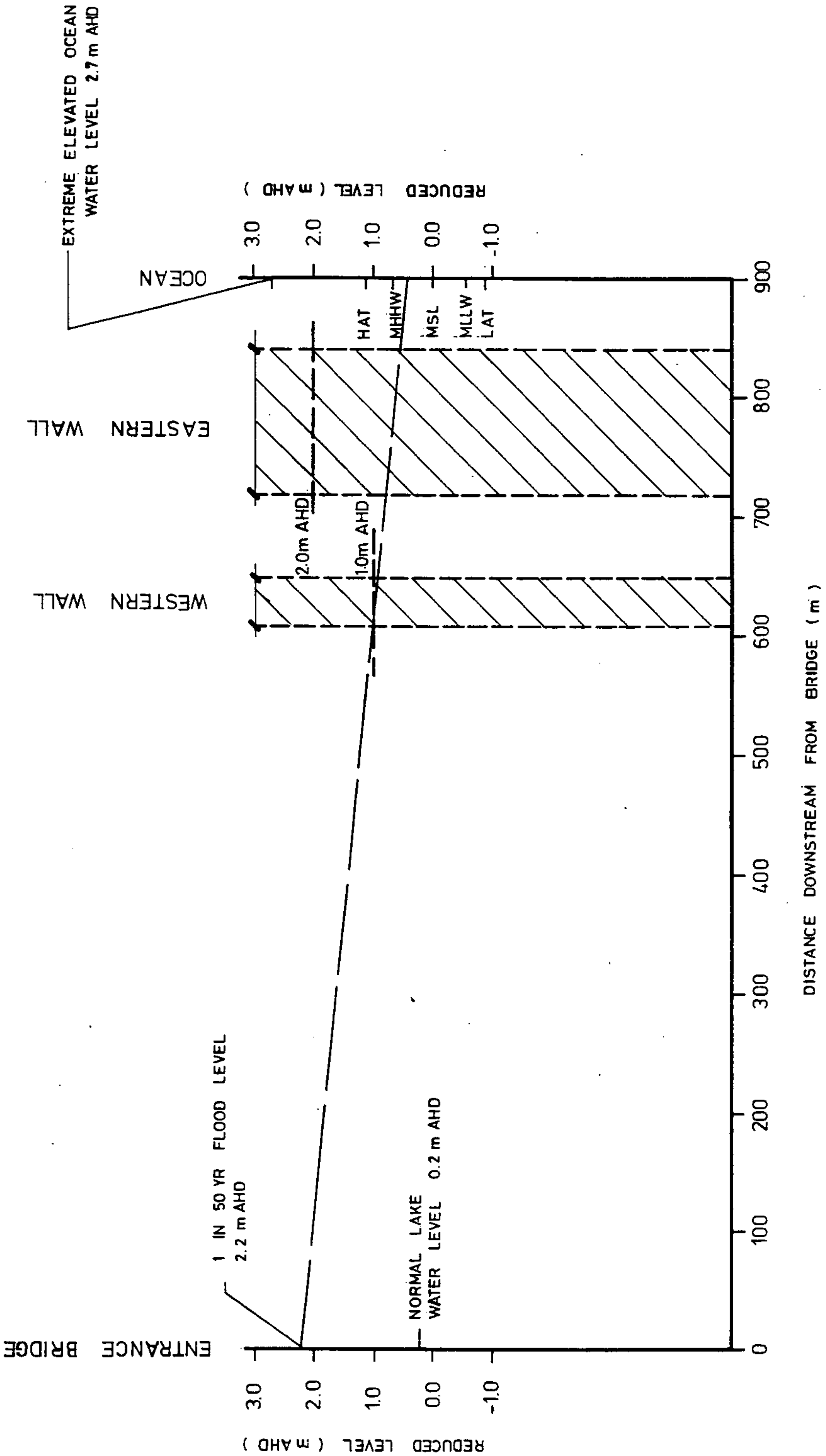
Western Wall

The western wall is well upstream from the beach and the effects of wave action are very much diminished compared to the seaward end of the eastern wall.

It is essential that the wall fully constrains tidal flows and therefore the crest of the wall must be higher than the water surface elevations due to tides. At this point in the entrance channel, maximum tidal levels would be expected to be about 0.6 m AHD or less *(corresponding approximately to positions 5 - 6 in* **Figure 2.4***)*. Allowing for some freeboard and possible settlement of the wall *(if a flexible structure)* it is considered that to achieve effective training of tidal flows the crest level should be not less than 1.0 m AHD.

In terms of flooding, a crest level of 1.0 m AHD would just contain flood flows corresponding to a 50 year return period lake level *(2.2 m AHD, refer* **Section 2.6***)* and a tailwater condition at approximately 0.5 m AHD *(a little below Mean Higher High Water)*, assuming uniform head loss between the lake and the sea *(refer* **Figure 3.2**). For higher tailwater conditions submergence of the western wall will occur. This is not considered to be a problem except where any associated scour on the downstream side of the wall could possibly lead to a situation where the tidal flows, once re-established, might "back-cut" the eastern wall and flow to sea to the south of the intended channel. For this reason it would be desirable to stabilise the area to the south of the two walls by means such as construction of a vegetated berm and dunal system.

FIGURE 3.2



RELATIVE LAKE
AND OCEAN LEVELS

A crest level of 1.0 m AHD is well below the level at which the adjacent beach and dunal area tends to form naturally *(refer* **Figure 2.1***)*. This will assist in retaining views from sunbathing areas and minimising visual impact. The outer section of the wall would be expected to be continually visible.

Eastern Wall

For the eastern wall, particularly near the seaward end, the design crest level will need to be raised in order to constrain tidal flows *(tidal elevations will approach open coast values at the seaward end of the wall - refer* **Table 2.2** *and* **Figure 2.4***)*, and to accommodate storm wave attack.

Weighing up each of the factors which influence the crest level, as listed at the start of this section, and based on experience, it is considered that the crest level for the seaward end of the eastern wall should not be less than 2.0 m AHD. Even so, during extreme storm events the wall would be submerged, thus design of the head of the structure will require special consideration *(extreme elevated ocean water level adopted as 2.7 m AHD - refer* **Section 2.7***)*.

A crest level of 2.0 m AHD for the eastern wall, while necessarily higher than for the western wall, is still at or below the level at which the adjacent beach and dunal area tends to form naturally *(refer* **Figure 2.1***)*. The channel side of the eastern wall, and the seaward section *(particularly after storms)*, would be expected to be continually visible.

3.1.7 Scour

Scour will occur adjacent to the structure due to a number of processes:

- tidal action
- flooding
- storm erosion.

Tidal Action

Depths of scour adjacent to the structure due to tidal action would not be expected to exceed the depths of 1.5 - 2.0 m below AHD measured in previous hydrographic surveys when the channel has been constrained against the southern rocky shoreline *(refer* **Section 2.3***)*. As will be outlined below, these depths of scour are less than those predicted due to flooding, hence tidal scour does not control the restraining wall design.

Flooding

The scour depths which may occur adjacent to the northern side of the restraining wall due to flooding have been estimated based on the work by Inglis carried out at the Central Waterpower Irrigation & Navigation Research Station in Poona, India *(***Reference 15***)*. Inglis recommended the following scour factors for estimating maximum local scour below water level:

. bridge piers	2 d
. large radius bank revetments	2.75 d
. spurs along river banks	1.7 d to 3.8 d.

where 'd' is the flow depth in metres given by the Lacey regime formula, namely:

$$d = 1.34\, q^{2/3}/F_1^{1/3}$$

and q = discharge per unit surface flow width *(m^3/s per m)*

F_1 = sediment factor = 1.0 for sand.

The situation in the case of the restraining wall does not conform strictly with any of the specific structures referred to above, although the western wall can be likened to a spur and the eastern wall to a large radius bank revetment *(but less severe)*.

The western wall is fully located in an area where bedrock is at a level of about -2 m AHD *(refer* **Figure 2.3***)*, hence potential scour to deeper levels as a result of flooding cannot eventuate.

The eastern wall is, however, critical. While open to subjective assessment, it is considered that a reasonable scour factor to apply is 2d which, for a design discharge of approximately 1300 m^3/s *(refer* **Section 3.1.3***)* and width of 250 - 300 m, would give predicted scour levels in the order of -5 to -7 m AHD.

The bedrock levels along the outer 90 m length of the proposed wall are comparable to the predicted scour levels and therefore provide insurance against any potential underestimation of scour levels. Further landward, the higher rock levels will control the level of scour.

Storm Erosion

The commonly adopted scour level for open coast shoreline structures subject to storm erosion is -2 m AHD *(***References 8** *and* **9***)*. This value is considered appropriate for the seaward end of the eastern wall. Scour protection to this level should be provided along the southern side of the wall for a distance landward from the head of the structure corresponding to the storm erosion demand *(flood scour will control the requirements for scour protection along the northern side of the wall)*. In the absence of any detailed studies of storm erosion demand a distance of 80 m for provision of the scour protection is considered reasonable.

3.2 ALTERNATIVE STRUCTURAL SYSTEMS AND MATERIALS

3.2.1 Introduction

The study brief is quite specific in terms of the type of materials which can be used for construction of the wall; namely that rock is not to be considered when selecting the external *(visible)* surface of the wall, whilst materials such as vegetated sand, geotextiles, and concrete sheet piling *(or any combination)* are to be considered *(refer* **Section 1.2***)*. In addition, the Department's earlier Report No. PWD 87051 advised that no quarry run or small rock armour should be used in the restraining wall construction owing to the problems that would occur if wave action deposited such material into the craters formed by the jet pumps *(the possibility that jet pumps could be introduced at a later date must be considered in the design of the wall as noted previously)*.

In view of the above, alternative construction materials have been limited to the following:

- vegetated sand;
- sand filled geotextile fabrics;
- concrete sheet piling.

Sheet piling was indicated for the outer section of the restraining wall in the conceptual design presented in Report No. PWD 87051 *(refer* **Figure 1.2***)*. Use of sheet piling is, however, not favoured for a number of reasons.

To be laterally stable, a sheet pile wall would require either a tie-back system along its southern side or need to be constructed with a toe and rely, inter alia, on the weight of sand on the toe to resist sliding and overturning. Both of these means for providing lateral stability would be at definite risk at times of storm erosion and/or flooding when scour can be expected along the southern side of the wall. As the wall must be designed to be inherently laterally stable even at times of storms and flooding, sheet piling would need to be arranged in the form of a caisson structure and incorporate scour protection.

Caissons would be a much more expensive form of construction than a sand filled geotextile wall, and therefore attention has been directed to a sand filled geotextile structure *(in combination with vegetated sand)* as the means of construction of the restraining wall.

3.2.2 Sand Filled Geotextile Structures

Sand filled geotextiles have been used for a number of seawall, revetment and groyne structures overseas, and for three groynes constructed in recent years in south-eastern Queensland by the Gold Coast City Council. These structures are located within the southport Broadwater *(two structures)* and on the open beach at North Kirra.

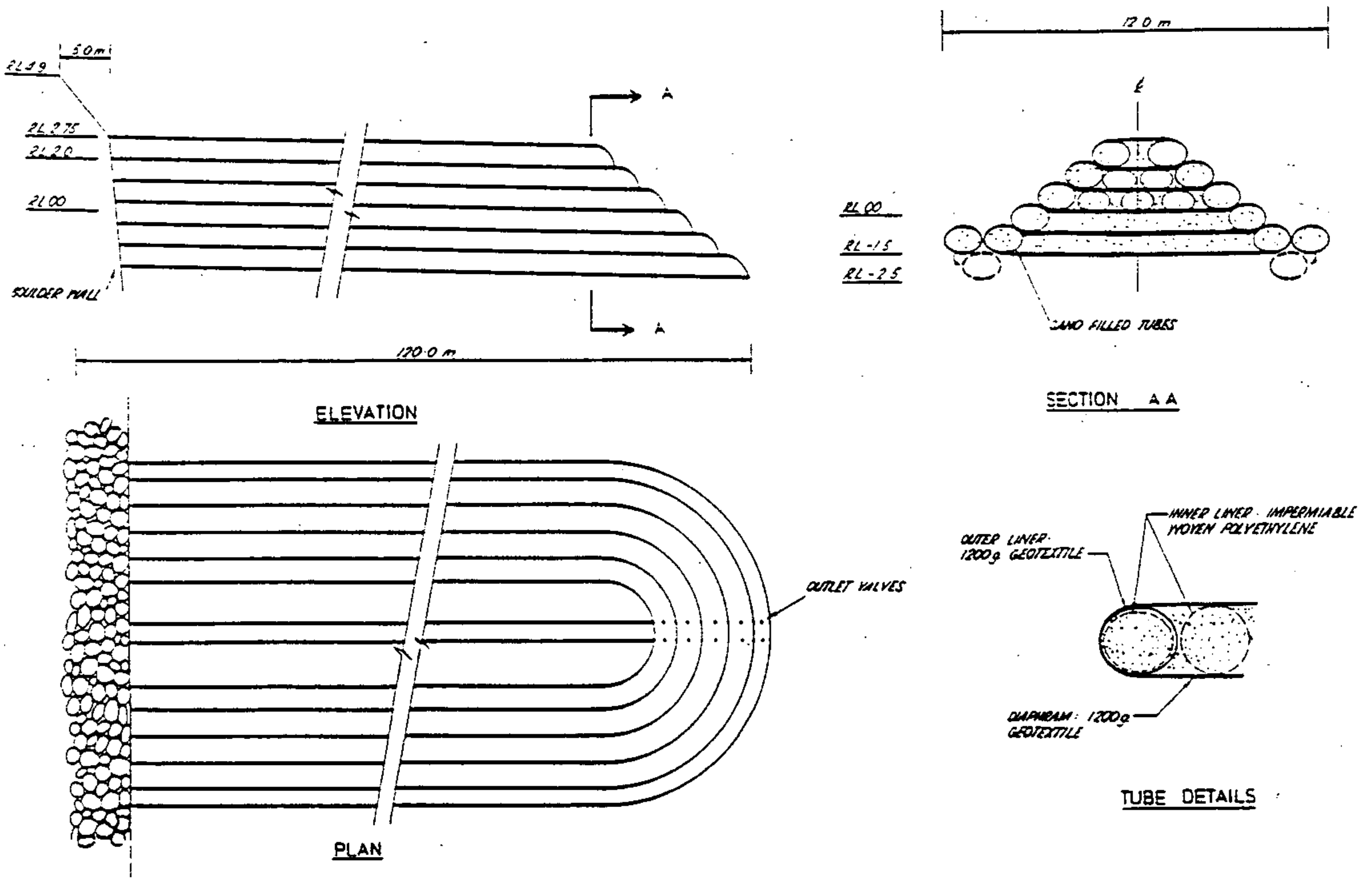
The two groynes located within the Broadwater are relatively small structures and exposed to low velocities and waves *(maximum tidal velocity less than 0.5 m/s and maximum significant wave height less than 0.5 m)*.

The groyne at north Kirra is, however, a more substantial structure having the following design criteria:

length	110m
height	5 m *(-2.5 m AHD to +2.5 m AHD)*
wave period	5 - 13 sec
maximum wave height	5 m *(breaking)*

The groyne was constructed utilising a composite structure of two high tensile strength materials - an impermeable inner liner of woven polyethylene coated polypropylene and a durable outer sheath of needle punched polyester geotextile *(mass 1200 gm/m^2)*. The general arrangement of the groyne is shown in **Figure 3.3**. The multiple tubes forming the structure had a diameter of 1.2 m, which represented the maximum diameter achievable for the 1200 gm/m^2 material based on the allowable radial stress developed during the hydraulic filling process.

FIGURE 3.3



NORTH KIRRA (QLD.) GROYNE
GENERAL ARRANGEMENT

Prior to the construction of the groyne, small scale physical model testing was carried out by the University of New South Wales Water Research Laboratory to determine qualitatively the possible modes of failure. The testing revealed that failure of the model occurred by dislodgement of the top layers at high water levels submerging the crest and for equivalent prototype waves of 4.5 m *(22.5^{o} wave attack angle)* and 5.0 m *(zero wave attack angle)*. Migration of the internal sand fill and localised settlement was also noted. However, the usual large scour at the head of groyne structures did not occur.

A structural analysis of the proposed material was also carried out prior to construction, by Lawson Consulting *(Queensland)*. The analysis indicated that failure could occur if:

- the outlet ports blocked during filling
- the internal sand fill liquifies
- the groyne is undermined by more than 3.5 m free span
- damage occurs due to vandalism.

An evaluation of the performance of the sand filled geotextile groynes has been subsequently carried out by the Special Projects Engineer of Gold Coast City Council *(**Reference 16**)*. The following main points were noted:

- vandalism was evident within the first month of construction of the groyne, taking the form of knife cuts. These cuts were easily repaired by Council's day labour staff using geotextile patches with contact adhesive;

- after storms *(2.5 to 3.0 m waves)* some slumping of the end of the structure occurred. An inspection underwater revealed loss of sand through a large tear caused by the excavator bucket teeth during construction. As the groyne was compartmentised, the damage remained localised;

- the hydraulic smoothness of the groyne proved popular with adjacent surfing beach users and was used as a water slide by children;

- little scour occurred around the head of groyne and it was evident from the adjacent beach shape that a wave splitting action rather than the normal wave reflection action of rock groynes takes place;

- in January 1986 a large knife cut, some two to three metres long, was observed on the second bottom tube near the seaward end of the groyne. Due to constant heavy swells over the period from January to June no repair work could be carried out on this damage and progressive slumping occurred. The compartmentisation effectively localised the problem until the damaged tube collapsed sufficiently for the tube on top to be displaced sideways tearing the envelope and causing a major collapse of the end 20 metres;

- whilst the problems with vandalism and the inadequate nose details have led to the almost total collapse of the seaward 30 m, the groyne has since been subjected to 5 m waves on several occasions and is still performing satisfactorily.

It is apparent from the North Kirra project that the concept of sand filled geotextile groynes is viable but that special attention must be given to vandal protection and detailing of the seaward end of the structure. It is also apparent that, providing the top layers are securely connected, a lower crest level could be adopted *(ie lower than 2.5 m AHD)*.

3.3 DESCRIPTION OF PROPOSED CONCEPT DESIGN

3.3.1 General

A concept design has been prepared for the restraining wall utilising sand filled geotextiles and vegetated sand, and is shown in **Figure 3.4.** The proposed arrangement of the wall superimposed on a March 1987 aerial photograph is shown in **Plate 3.1.**

The design incorporates longitudinal tubes connected to form a pyramidal structure. Use of tubes laid transversely was also considered but this system did not provide any cost benefit and was less suited to installation of flood scour protection.

The area to the south of the restraining wall is proposed to be stabilised, where practicable, by vegetation to achieve a minimum level of 2.5 m AHD and so assist in directing flood flows along the proposed entrance channel. An appropriate landscape plan should be prepared for this area.

The western wall and eastern wall have a number of common design features:

- based on work recently carried out and inspected in Germany, it is considered that significant extra strength and durability could be achieved using a single permeable layer of 2000 gm/m^2 geotextile, rather than the composite impermeable liner and 1200 gm/m^2 geotextile as used previously by Gold Coast City Council.

  By using a higher strength geotextile, the diameter of each tube can be increased. The maximum diameter with one stitched seam is approximately 1.3 m resulting in an effective height of 1.1 m. The permeable tubes would need to be filled with at least a 150 mm dredge;

- the geotextile would need to have maximum UV resistance and be needle punched *(such as polyester)* for greatest durability and strength, and abrasion resistance;

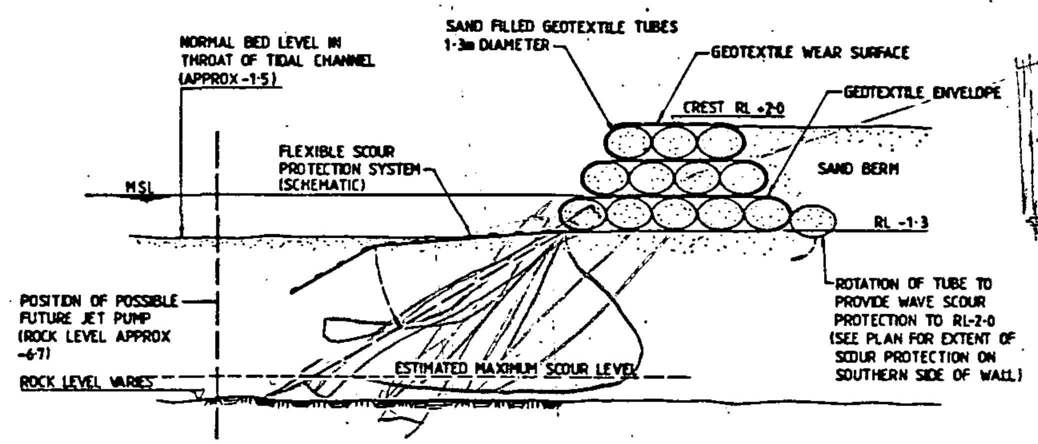
- unless individual tubes could be adequately interconnected, each layer of tubes would be enveloped with 2000 gm/m^2 geotextile and the layer then connected with the layer above and below, as appropriate;

- all joints and connections in the geotextile would be designed to ensure the full strength of the geotextile was developed. This might be achieved by either stitching or glueing;

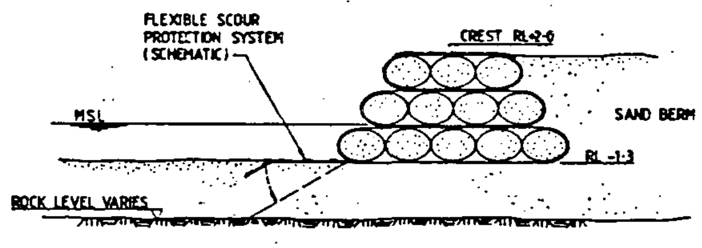
- vandal resistance would be provided by epoxy sand coating. A first coating would be applied to exposed surfaces in the shop prior to installation, and a second coating applied after installation *(filling of the tubes stretches the fabric leaving cracks in the initial coating)*. While this protection should be adequate, further protection could be provided if required by, say, a glued-on epoxy coated geotextile wear surface *(1200 gm/m^2)* or stainless steel mesh;

- even though waterway access is substantially enhanced by the "twin-wall" design which incorporates a beach area, from a public safety viewpoint it is considered necessary to provide for regular egress from the channel up the side of the wall. It is likely that potential slime growth on the geotextile will make egress difficult unless some form of ladder is incorporated in the design. It is suggested that climbing handles be constructed so as not to impose undue stress on the tubes, eg by anchoring the handles between each layer of tubes.

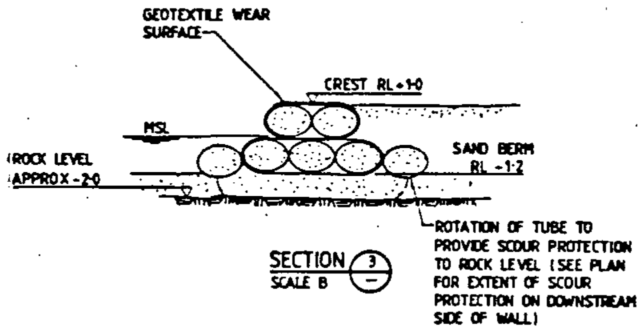
FIGURE 3.4



SECTION 1
SCALE B



SECTION 2
SCALE B



SECTION 3
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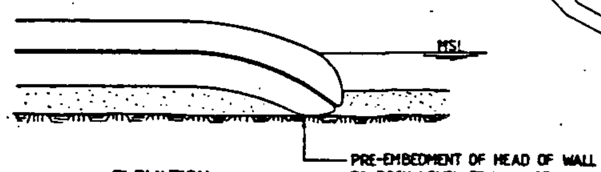
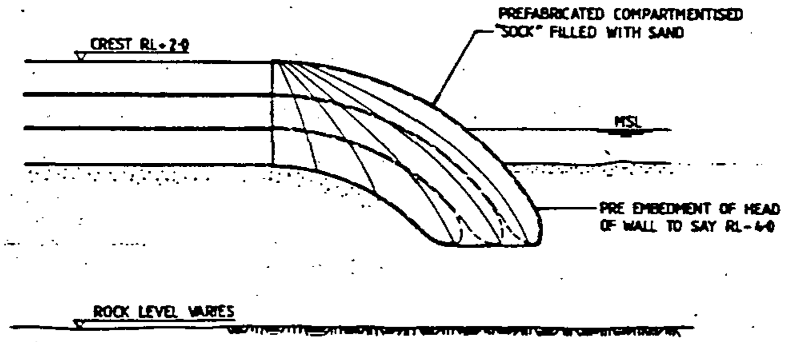
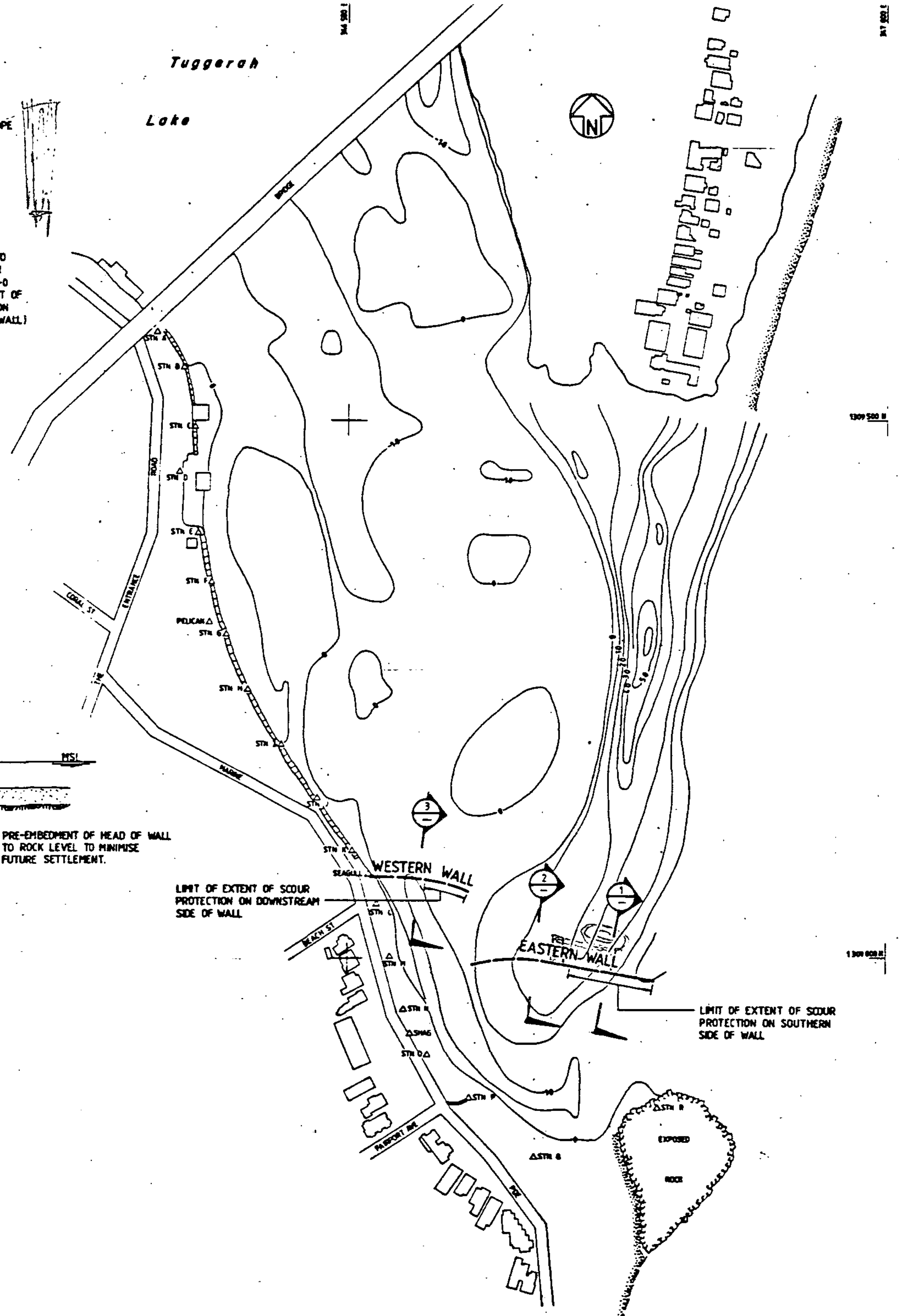


ELEVATION
END OF WESTERN WALL
SCALE B



ELEVATION
END OF EASTERN WALL
SCALE B



PLAN LAYOUT
SCALE A

NOTES

1. GEOTEXTILE TO HAVE MAXIMUM UV RESISTANCE AND BE NEEDLE PUNCHED.
2. MASS OF ALL GEOTEXTILE TO BE 2000 gm/m^2 EXCEPT FOR WEAR SURFACE WHICH CAN BE 1200 gm/m^2.
3. FLEXIBLE SCOUR PROTECTION SYSTEM ON CHANNEL SIDE OF EASTERN WALL TO BE PHYSICALLY CONNECTED TO ROCK AT UPSTREAM END OF WALL.
4. HYDROGRAPHIC SURVEY INFORMATION IS BASED ON 1979 PWD HYDROSURVEY (PWD PLAN CATALOGUE No 5746).
5. ALL LEVELS AND CONTOURS ARE IN METRES RELATIVE TO AUSTRALIAN HEIGHT DATUM (AHD).

0 50 100 150 200m
SCALE A

0 2 4 6 8 10m
SCALE B

PROPOSED RESTRAINING
WALL CONCEPT DESIGN

- sand for filling of the tubes would be obtained from a nearby borrow area *(to be nominated by the Public Works Department and Council)* by means of a dredge having a minimum 150 mm diameter discharge line.

Further particular details of the western and eastern wall designs are set out in the following sections.

3.3.2 Western Wall

The western wall is approximately 90 m long with a base at -1.2 m AHD and crest level at 1.0 m AHD. The height of the wall would be made up of two layers of 1.3 m diameter tubes. The crest of the wall has a width of two tubes for stability. A single 1.3 m diameter tube would be placed along the upstream side of the wall and along a proportion of the downstream side, and be designed to rotate downwards to rock level *(approximately -2 m AHD)* to provide scour protection.

At the head of the structure each of the individual tubes would be stitched together in the form of a "chisel" profile. It is intended that the head of the structure would also be given some pre-embedment down to rock level *(by means of jetting)*. This would minimise future settlement due to scour *(the head of the structure will be most susceptible to scour)* and also assist in presenting a streamlined profile to wave action which may penetrate up the channel.

3.3.3 Eastern Wall

The eastern wall is approximately 180 m long with a base at -1.3 m AHD and crest level at 2.0 m AHD. The length of the wall is made up of three layers of 1.3 m diameter tubes. The crest of the wall has a width of three tubes for stability.

The scour protection along the southern side of the wall comprises a single 1.3 m diameter tube which is designed to rotate downwards to at least -2.0 m AHD to provide scour protection against wave action. This tube would be provided landward from the head of the structure for a distance of approximately 80 m.

The scour protection along the northern *(channel)* side of the wall, to be provided down to a level of say -6 m AHD, could be either a continuation of the sand filled tubes or a weighted geotextile fabric. In either case, some pre-embedment of the scour protection *(by means of jetting)* would be desirable to control the initial lowering and rotation of the scour blanket. The leading *(upstream)* end of the scour blanket would be securely fastened to the natural rock surface to minimise the possibility of flood flows penetrating under the blanket and causing the scour blanket to "sail".

The arrangement of the head of the structure is critical and must be carefully detailed to avoid separation of the tubes and fatigue failure of the seams. The following arrangement is proposed:

- each individual tube in a layer to be interconnected, each layer to be enveloped with 2000 gm/m^2 geotextile, and each layer to be connected to the layer above and below as appropriate *(as for the remainder of the structure)*;

- a prefabricated hollow compartmentised "sock" to be placed over the end of the wall and secured to the outer layer of geotextile;

- the end of wall to be lowered by controlled jetting to a level of say -3 to -4 m AHD, if practicable, to minimise future settlement due to scour;

- the prefabricated sock to be filled with sand by hydraulic placement from crest level.

3.4 NOTES ON CONSTRUCTION METHOD

The western and eastern walls would be constructed sequentially with the western wall constructed first to provide an opportunity to refine construction techniques in this more sheltered environment.

It is envisaged the construction method would be similar to that adopted for the North Kirra groyne; namely construction "in the dry" by excavation to the base level of the walls, formation of bunds, and dewatering. From a construction viewpoint the work would be best carried out when the entrance was heavily shoaled or naturally closed *(obviously the wall is also not required until this time)*. Temporary artificial closure of the channel should be contemplated if the channel is heavily shoaled but not fully closed.

Generally speaking, the wall is then constructed from the bottom to the top through systematically rolling out the pre-coated geotextile tubes and enveloping material, and hydraulically placing the sand fill. After filling is complete, a further epoxy sand coating would be placed on exposed areas, as noted previously.

It is understood it is intended to call tenders for supply of materials and construction of the wall *(and probably for the detailed design of some selected elements)*. It is of interest to note that this was also the approach adopted by Gold Coast City Council for the North Kirra groyne project but that no tenders for the construction were received owing to the relative complexity of the project and the risks involved.

While it is probably desirable to call tenders, it is considered that, due to the nature of the works, construction should ideally be carried out by local Council's day labour force, provided specialised supervision is available. This is in view of the difficulty of specifying the actual time of commencement for the work *(as this will be subject to the natural fluctuation in entrance conditions)*, the contractor's likely assessment of the risk associated with the specialised nature of the work and occurrence of flooding and storms, the ability of Council to divert resources onto other projects during weather downtime, and the availability of appropriate Council plant *(earthmoving equipment and dredge)*.

3.5 ESTIMATED COST

A preliminary estimate of the construction cost for the restraining wall has been prepared and is set out in **Table 3.1.** This estimate is based on the experience of Gold Coast City Council using day labour resources. Note that the estimate does not allow for:

- escalation after September 1988
- survey, geotechnical and design costs
- future maintenance and repair costs or replacement costs.

Future maintenance and repair costs are difficult to estimate but, assuming that good vandal and abrasion resistance is provided *(eg two coatings of sand epoxy and a top wear surface)*, costs are unlikely to exceed $5,000 per annum based on the experience of Gold Coast City Council.

Similarly, it is difficult to estimate accurately the 'life' of the sand filled geotextile structure. Aside from vandal attack and abrasion, the main potential problems are degradation due to UV exposure, and fatigue of seams and other connections. Based on the experience of Gold Coast City Council and discussions with geotextile manufacturers, the estimated life of the structure is likely to be between 10 to 15 years. The present value replacement cost, for a discount rate of 8% p.a., would be $270,000 and $185,000 for a 10 and 15 year structure life respectively.

TABLE 3.1 PRELIMINARY ESTIMATE OF CONSTRUCTION COST FOR RESTRAINING WALL

	Item	Amount $ 1988
1.	Supply of geotextile *(see Note)*:	
	. western wall *(90 m length)*	$ 75,000
	. eastern wall *(180 m length)*	$260,000
2.	Construction	
	. establishment/disestablishment	$ 5,000
	. closure of channel *(Provisional Item)*	$ 5,000
	. excavation and bunding	$ 15,000
	. filling of geotextiles tubes: 150 hrs @ $60/hr	$ 9,000
	. labour costs: 2 weeks @ $10,000/wk	$ 20,000
	. vandal and wear protection:	
	- two coats of epoxy sand, 2500 m^2 @ $20/m^2	$ 50,000
	- glued-on epoxy coated wear surface along top	$ 25,000
	. supervision, say	$ 20,000
		$484,000
	Contingencies, say 20%	$ 95,000
		$579,000
	Say	**$580,000**

Note:

. Supply cost of 2000 gm/m^2 geotextile formed to a 1.3 m diameter tube - $50/m.
. Supply cost of 2000 gm/m^2 geotextile - $10/m^2.

3.6 GEOTECHNICAL INVESTIGATION

A geotechnical investigation will be required prior to commencement of the detailed design and documentation stage for the project.

This work should provide the following information:

- the surface level of rock along the proposed alignment of the western and eastern walls at, say, 10 m centres;
- insitu strength and classification of any firm strata *(other than rock)* encountered within the above holes which may prove resistant to scour;
- the grainsize distribution and composition of sand from the proposed borrow areas for filling of the geotextile tubes *(these areas to be nominated by the Public Works Department and Council)*.

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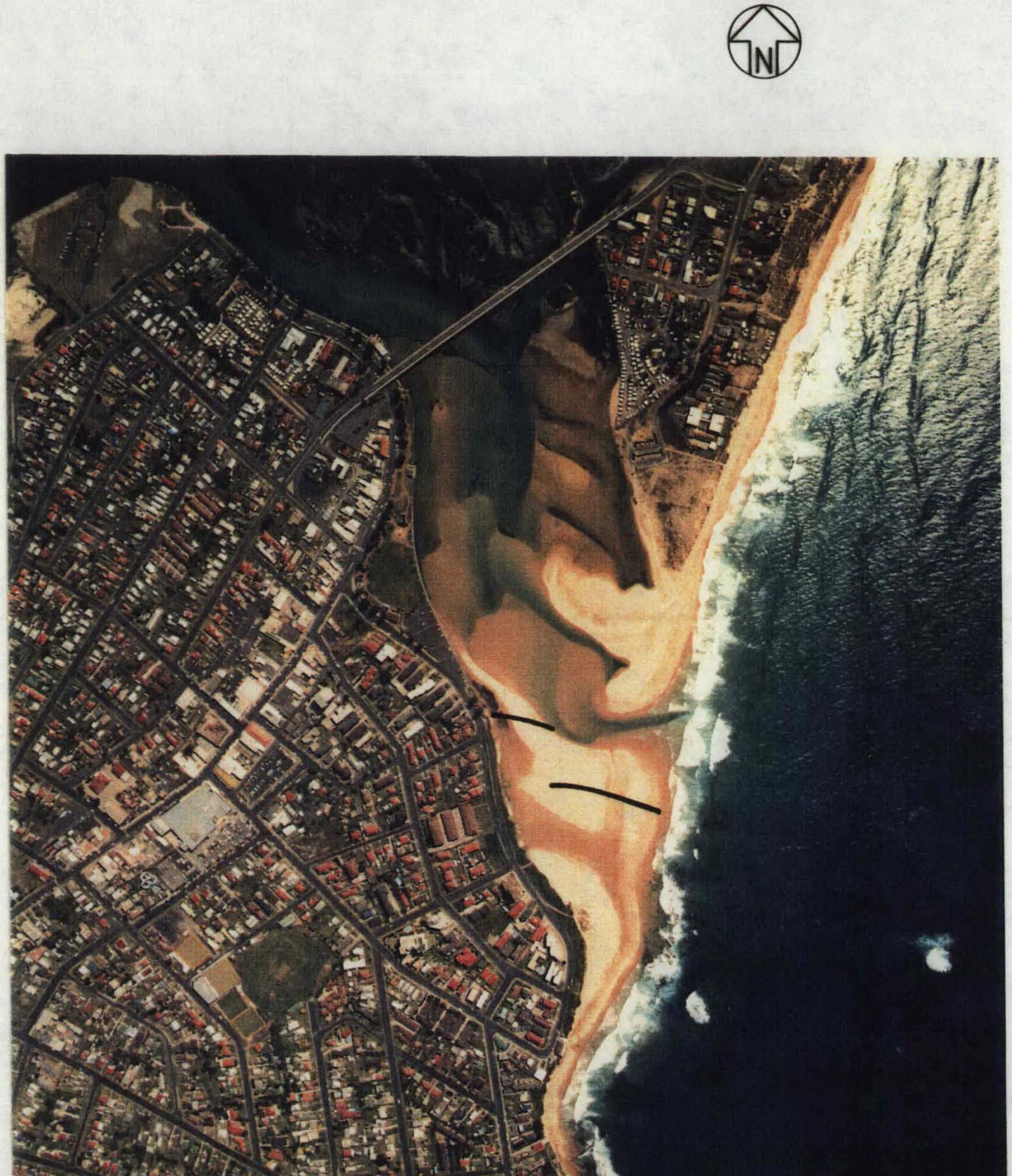

PROPOSED ARRANGEMENT OF
RESTRAINING WALL SUPERIMPOSED ON
MARCH 1987 AERIAL PHOTOGRAPH