Brunswick River Floodplain Management Committee

BRUNSWICK RIVER

FLOODPLAIN MANAGEMENT INVESTIGATION

NOVEMBER, 1989

WEBB, McKEOWN & ASSOCIATES PTY. LTD.

CONSULTING ENGINEERS

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1. SUMMARY

A number of studies have been carried out in the Brunswick Valley to identify and resolve flooding problems. These culminated in a Floodplain Management Study which examined options for protecting existing development, and looked at the impacts on flood behaviour of a number of options for further development. More recently residents in the Marshalls Creek catchment have requested action by Council to investigate and solve the flooding problems in that area. At the same time Council has had applications to re-zone and develop flood liable land in the Marshalls Creek floodplain. In particular, the Ocean Shores Development Corporation Pty Ltd (OSDC) has submitted a Proposed Development Concept Plan for the northern portion of the area known as North Ocean Shores.

In response to the above requests Byron Shire Council formed a technical sub-committee to the Brunswick River Floodplain Management Committee, consisting of representatives of the main developer (OSDC), Council and the Public Works Department (PWD). They have subsequently supervised the carrying out of a Floodplain Management Study by a consultant, the subject of this report.

Marshalls Creek forms the northern tributary of the Brunswick River which enters the Pacific Ocean at Brunswick Heads. The Creek has experienced a number of floods within recent times including the May 1987 flood, which approached a 1% event in the upper reaches. This flood inundated the townships of Billinudgel, South Golden Beach and New Brighton and flooded at least 60 houses. Four houses were also inundated within South Ocean Shores Estate.

The study was commenced in late 1987 using as a basis a Development Plan already submitted to Council by OSDC for consideration at that time. An exhaustive analysis of the impacts of the development was then carried out. A wide range of flood mitigation measures were considered, some of which would protect existing development and others that would offset any possible impacts of future development. The objective was to accomodate an increase in development of the area while at the same time reducing the flood damages for existing development. Comments from the public were invited at a public

meeting held in May 1988 when preliminary results were available.

Using early results from the study, and taking into account environmental and social concerns, the proposed OSDC development was amended in December 1988 to the Development Concept Plan whose floodplain management impacts have been considered herein.

The major components of the Proposed OSDC Development Concept Plan are:

- a residential subdivision surrounding a 16ha lake which would link to and complete the present canal system within North Ocean Shores,
- partial development of the flat grassland area behind the beach dune system towards the township of Wooyung for a tourist development,
- a low scale tourist development on approximately 13ha of land between New Brighton and South Golden Beach termed the Holiday Village Site,
- residential development within South Ocean Shores Golf Course,
- development of the land adjacent to Shara Boulevarde encompassing a retail centre, motel, educational and residential facilities,
- rural residential and rural lands within the Yelgun Creek catchment.

The Committee also considered as part of this study, future development of two parcels of land adjacent to Balemo Drive owned by Mr J Mangleson and two parcels of Crown Land within the area encompassed by the Development Concept Plan. Possible development of the Crown Land was considered at this time, as it is possible that it could be sold for development at some future date.

The Committee requested that the following proposals for benefiting existing development be considered at the same time to ensure a balanced approach:

- levee around South Golden Beach,
- levee or other means of flood protection for New Brighton,
- levee around Billinudgel,
- lowering of the railway line at Mullumbimby,
- widening of the road and rail bridges at Billinudgel.

It was required that the study should broadly consider any environmental issues which arose as a consequence of any proposed flood mitigation option. The possible impact of the Greenhouse Effect was also to be considered.

It was quickly established in the early stages of the study, that lowering of the railway line at Mullumbimby and/or widening of the bridges at Billinudgel, had no significant impact on flood levels in the Marshalls Creek floodplain. These options were therefore not considered any further in this study. If they are considered at a future date (Mullumbimby is presently being studied) they can proceed independently of any other proposed options for the Marshalls Creek floodplain.

Investigation of the land owned by Mr Mangleson showed that development of one parcel of land (Site B) may produce significant hydraulic impacts which could not be readily mitigated. Access during a flood could be hazardous, and it appears unlikely that a detailed assessment of the hydraulic impacts of developing this land, would show that these problems could be overcome. Development of the remaining parcel of land (Site A), adjoining Balemo Drive, would appear to have relatively minor hydraulic impacts provided that a reasonable setback from the creek was specified. Environmental effects of clearing and filling this area would have to be addressed.

The Committee resolved that the hydraulic impacts of the Development Concept Plan (including the Crown Lands), development of Site A of the Mangleson land and a levee around South Golden Beach, should be investigated as a single development and flood protection package. Levees around Billinudgel and New Brighton were not explicitly included in the package as the model showed that the additional hydraulic impacts of these options were negligible.

A comprehensive analysis of the effects of the above package was undertaken. There are no precise details available on the likely extent of fill. Reasonable assumptions were therefore made on the extent of development within the various sites using the best available information. Overall it was assumed that the development would achieve the maximum likely extent as designated on the Development Concept Plan. However, in practice it is expected that the actual development will be somewhat less extensive and thus the final design flood levels will be less than determined herein.

The hydraulic model showed that the proposed future development and flood protection levees would increase flood levels by up to 264mm if no compensatory works were included. Modelling of alternative compensatory works was then undertaken to ensure that flood levels along Marshalls Creek would not be increased. The aim was in fact to significantly reduce existing flood levels and provide identifiable benefits.

The following possible flood mitigation measures were evaluated:

- flood outlets at three locations,
- dredging of Marshalls Creek,
- widening of Orana Bridge,
- floodway across South Ocean Shores Golf Course,
- separation of the Yelgun Creek and Marshalls Creek systems including a flood-gated system,
- widening of the link between Yelgun Creek and Marshalls Creek,
- floodway immediately downstream of the Pacific Highway.

The brief required that environmental factors be considered in proposing any flood mitigation works. Of the above works, dredging and the flood outlets could cause environmental concerns, so in considering combinations of flood mitigation works, alternatives which do not require these measures have also been considered.

Preliminary assessment of the environmental impacts of dredging Marshalls Creek indicated that the proposal should be acceptable. A possible outcome from more detailed consideration of the environmental impacts of dredging, could be that dredging to a lesser depth may be acceptable. To investigate this further, computer simulations were carried out for a range of dredging depths.

The flood outlet at North Ocean Shores has been the subject of a separate study which is described in Appendix E. The study showed that it is technically feasible, but doubts remain over the likely maintenance costs. There do not appear to be any significant adverse environmental effects.

A comprehensive series of computer simulations was carried out for a range of design floods. The above measures were initially considered individually and then in combination. Three combinations were considered - the first included both dredging and the NOS flood outlet; the second involved no dredging; and the third involved no flood outlet.

By combining the following flood mitigation measures with the proposed development, the adverse effects of the development and flood protection package were eliminated and at key locations flood levels were significantly lowered below their existing levels:

- dredging of Marshalls Creek to a maximum depth of -2.5m AHD,
- widening the Yelgun Creek bridge and waterway to 20m,
- construction of an outlet (in the form of a localised lowering of the dune) at North Ocean Shores. This was assumed to be a 200m wide weir with a hard crest at 1.9m. When a flood event was expected, the dune sand covering the weir would be reduced to RL 2.3m and would then scour when overtopped,
- provision of a floodway (500m long) on the northern bank of Marshalls Creek, immediately downstream of the Pacific Highway.

The second combination involved the same measures, but instead of dredging, the waterway area of Orana bridge was improved. This still produced an acceptable outcome, but the flood level reductions were less than for the first combination and flood levels were increased downstream of Orana Bridge in a 1987-type flood. Lesser depths of dredging were also modelled and these produced results between those for the first two combinations. It is clear therefore that even if the scale of the dredging has to be reduced or eliminated altogether, by widening Orana Bridge in combination with the other measures set out above, the proposed development and flood protection package is not precluded because of adverse effects on flood behaviour.

The third combination involved the same set of measures outlined above for Combination 1, but with the NOS flood outlet eliminated and the Orana Bridge waterway area improved. Again the results were acceptable, but the reductions in flood levels are not as useful as in the first combination.

It is clear, that either the dredging or the NOS flood outlet, has to be part of any acceptable combination of flood mitigation works in order to adequately compensate for the proposed developments. The optimal outcome from a hydraulic viewpoint is when both measures are included.

The timing of both the development and flood mitigation elements of the package will depend upon the rate at which development proceeds and the funds available to Council to construct flood mitigation works. Dredging of Marshalls Creek could be undertaken in the absence of other works and would produce a significant reduction in flood levels. It could be undertaken by Council for little or no cost assuming that the material could readily be sold. However further environmental investigations will be required and a EIS prepared. Disposal of the dredged material would logically be to the development sites, this being the only apparent way to make use of the material in a short time frame. If development was not proceeding at the same time a slower rate of dredging would probably result.

Leveeing of South Golden Beach could also be undertaken in the absence of other works, except that it would increase flood levels at New Brighton by up to 12mm in a 1% flood, and an additional levee would be required on the northern boundary if it proceeded independently of the development package.

The Billinudgel levee could also proceed in the absence of the development and flood protection measures. Construction of the levee has only a nominal hydraulic impact downstream.

A levee around parts of New Brighton could be constructed in the absence of other works. However, it is likely to have an adverse impact on upstream flood levels which have not been quantified as part of this study. There are a number of difficulties in constructing such a levee which have not yet been resolved. It would not be cost effective to protect all of New Brighton using levees. Other measures, such as flood proofing, would have to be considered.

Limited development of Site A of the Mangleson land could proceed independently of the remainder of the development package, but it may require part of the floodway located opposite to be implemented to ensure no adverse impacts at Billinudgel.

The remaining flood mitigation options may also be implemented in the absence of the development package, but they would be costly and would provide little advantage to the present community.

2. INTRODUCTION

2.1 General

The township of Ocean Shores is located approximately 3 kilometres north of Brunswick Heads on the far North Coast of NSW (Figure 1). It is being developed as a major new development area and the Southern Division of the project, known as South Ocean Shores, has already been completed over the past 15 years. The latter comprises a major residential subdivision surrounding the golf course adjoining Marshalls Creek (Figure 2).

Ocean Shores Development Corporation (OSDC) are intending to complete the development of the Northern Division known as North Ocean Shores and two sites in South Ocean Shores. The Northern Division has already been partially developed and currently comprises a small residential subdivision adjoining a canal network (Capricornia Canal). The canal provides a hydraulic connection between Yelgun and Billinudgel Creeks to the North and West, and drains to Marshalls Creek to the south. The canal divides the township of South Golden Beach.

In addition to the OSDC proposal, the study has also examined two parcels of land owned by Mr J Mangleson (referred to in this report as Site A and Site B), possible future development of two parcels of Crown Land and flood protection works proposed by Council (Figure 2).

Since a large part of the proposed development is on flood liable land the Brunswick River Floodplain Management Committee commissioned Webb, McKeown & Associates Pty Ltd to undertake a Floodplain Management Study of the area.

The study was administered by a Steering Committee consisting of:

- G Alderson - Shire Engineer, Byron Shire Council,
- L Pilkington - Representing OSDC
- J Bodycott - Programme Manager, Public Works Department.

2.2 Aims of the Study

The Floodplain Management Study constitutes the first stage of a two stage study. The second stage, which is nearing completion, examines in a preliminary manner the water quality implications of floodplain management proposals proposed in the first stage, particularly the proposed dredging.

The aims of the Stage 1 study are to:

1. Determine if there are any adverse impacts from the proposed developments:

   - within North Ocean Shores,
   - at Holiday Village (including the Crown Land),
   - at South Ocean Shores Golf Course,
   - along Marshalls Creek (including Sites A & B and the Crown Land).

2. Evaluate mitigation works to reduce any adverse impacts and thus ensure that existing properties are not adversely affected, and,

3. Investigate the impacts of other possible future flood mitigation works to ensure that these are not precluded by the developments investigated under 1 above.

The study was also to keep in mind the possible environmental effects of the mitigation options to ensure that no option was put forward which would "fallover" on environmental grounds.

2.3 Previous Studies

Several hydrologic/hydraulic investigations have previously been carried out for the North Ocean Shores site. However, these studies all relate to previous development concepts which have now been considerably modified.

Flood behaviour along Marshalls Creek and the Brunswick River has previously been investigated for Council. A Hydrology Report (Reference 1) was completed in July 1984 which established the rainfall parameters of the area. The Marshalls Creek Flood Study (Reference 2) used data from the Hydrology Report to determine the design flood levels. A Flood Study and a Floodplain Management Study for the Brunswick River were also undertaken (References 3 and 4).

The hydrologic model adopted is known as the Watershed Bounded Network Model (WBNM) (Reference 5). The hydraulic model (Reference 6) adopted is termed the "Cell Model" as it simulates the floodplain as a series of interconnected cells. Details of both models are given in References 2 and 3 and the Cell Model Manual (Reference 7). A short description of the models is given in Section 3.

2.4 Approach to the Study

The previously calibrated and verified hydrologic and hydraulic models established in the Marshalls Creek Flood Study were expanded to encompass the entire North Ocean Shores catchment.

The models were also refined to ensure that they could accurately reflect any changes that may be caused by the proposed development. The models were then tested on the May 1987 flood which occurred following completion of the earlier Flood Study. Along Marshalls Creek, where the May 1987 flood was the largest flood on record, the hydraulic model was fine tuned to match the recorded flood data. At Billinudgel, the May 1987 flood approximated a 1% event.

The 1%, 2% and 5% design floods as well as the May 1987 flood were run through the models for existing conditions. The hydraulic model was then modified to simulate the proposed total development. Various flood mitigation alternatives were then examined to reduce flood levels and protect existing development.

2.5 Survey Data

The majority of the survey data were obtained from the sources given in the Flood Study (Reference 2). All levels are on Australian Height Datum (AHD).

To supplement this information, further survey was undertaken specifically for this study. The majority of this additional survey was obtained from Sharp & Skehan Pty Ltd (Licensed Surveyors), Gold Coast, Queensland. Council also provided additional survey data at Billinudgel.

3. HYDROLOGIC AND HYDRAULIC ANALYSES

3.1 General

Flood behaviour is modelled using two separate computer models. The first is a hydrologic rainfall-runoff model. This converts the rainfall over the catchment into runoff or flow. In this study the Watershed Bounded Network Model (WBNM) was used (Reference 5). This model was developed at the University of NSW and is recommended as a suitable model for use in flood studies in Australian Rainfall and Runoff (Reference 8).

The second model is a hydraulic model which converts the river flows obtained from the hydrologic model into river heights throughout the floodplain. Apart from flood heights the model also determines velocities, flow distributions, depths and duration of flooding. The model used in this study is the Cell Model (Reference 6). This model is a sophisticated mathematical model which is able to take account of changing flood flows and varying ocean levels with time.

The Cell Model represents the river as a series of cells connected by cross-sections and weirs. It is able to simulate the effects of a proposed development through adjustment of the cross-sections, weirs or storage areas of the cells.

3.2 Calibration

Prior to use of the models for design purposes both models must be calibrated to actual data. This is an extremely important part of any investigation and ensures that the results obtained from the models are credible.

The first step of the calibration process is to run rainfall data through the hydrologic model to produce flows. The data consist of rainfall depths and temporal patterns. These data are obtained from the Bureau of Meteorology or other Government Authorities and from local residents who record their own rainfall. The hydrologic model is calibrated to flow data obtained from a gauging station. In the Brunswick River Valley the only gauging station for which estimates of flow are available is at Durrumbul (Figure 1) located upstream of Mullumbimby.

The hydraulic model is calibrated to recorded flood height data. For this study these generally consist of flood marks noted by residents and later tied into a datum by survey. Peak flood levels are also recorded on special maximum height recorders installed by the Public Works Department and Council. The variation of flood height with time can be obtained from stream gauges, such as the one located at Billinudgel, or by residents recording flood heights at intervals during the course of a flood.

The models are calibrated by adjusting various parameters so that the model outputs match the observed data. The calibration of the model is then verified where possible using a different flood.

The above calibration and testing procedure was undertaken in the previous studies using the best available data. Since the May 1987 flood occurred after the publication of these studies, the models were re-examined using this new data.

3.3 Hydrologic Analysis

The WBNM model of the catchment was expanded to provide better definition within the catchments of Yelgun and Billinudgel Creeks (Figure 4). The calibration parameters which were previously adopted in the Flood Study, and were again used in this study, are given below:

"C"	2.2
Initial Loss (mm)	0
Continuing Loss (mm/h)	4.0

3.3.1 May 1987 Flood

Daily rainfall data and pluviograph data from the official rain gauges were obtained from the Bureau of Meteorology and the Public Works Department (PWD). In addition, a considerable amount of rainfall information was obtained from local residents who record rainfall for their own use. This information was collected by an extensive telephone survey of schools, stores and other premises. These data were particularly valuable over the Marshalls Creek catchment which has no official rain gauges.

The rainfall data were fed into the WBNM model and run using the adopted parameters. A good fit was achieved at the gauging station at Durrumbul (Figure 5). The model peak was within 7% and the model volume within 8% of the actual values.

3.4 Hydraulic Analysis

The Cell Model of the floodplain established for the Marshalls Creek Flood Study (Reference 2) was expanded to provide further cell subdivision along Marshalls Creek and to refine the layout within the Yelgun and Billinudgel Creek floodplains. The model layout for Marshalls, Yelgun and Billinudgel Creeks is shown on Figure 6 and the remainder of the model on Figure 7.

3.4.1 May 1987 Flood

A considerable amount of flood data were available for lower Marshalls Creek and Yelgun Creek. This included surveyed flood marks and data from maximum height recorders and automatic river recorders. A time history of river heights (stage hydrograph) was available from the automatic gauge at Billinudgel and from manual readings at the New Brighton Post Office. All the available data are listed in Appendix A and the peak flood levels are shown on Figure 8.

The tide gauge at the entrance to the Brunswick River failed to operate during the flood and therefore tidal data were synthesised by the PWD using data from the Tweed River recorder.

The flow hydrographs from the WBNM model were input to the Cell Model and the model used to simulate the flood. Preliminary results from the Cell Model showed that it was impossible to match the observed flood gradient near the mouth of the Brunswick River. As the mouth is restricted by the breakwaters to approximately 70m width, there are high velocities during a flood. It was decided therefore to investigate the possibility that the mouth would have scoured during the flood and subsequently filled in. The results from sand jetting undertaken by the PWD showed that up to 2m of sand could be removed from the bed in the course of a major flood. The cross-sections in the model were adjusted to account for this scouring and a satisfactory match to the observed data was obtained. The scoured sections were subsequently used for all design floods.

Minor recalibration of the Cell Model was undertaken along Marshalls Creek due to the further subdivision of the model. A satisfactory fit to the observed data was obtained and a longitudinal profile along Marshalls Creek is shown as Figure 9. Longitudinal profiles along the Brunswick River and Capricornia Canal are shown on Figures 10 and 11. A comparison of the model and recorded stage hydrographs at Billinudgel and New Brighton is given on Figure 12.

The May 1987 flood was the largest recorded flood at Billinudgel, reaching approximately the 1% level. However as the flood coincided with a relatively low ocean level, compared with the previously adopted design floods, the peak flood levels rapidly decreased downstream to less than a 2% event at New Brighton. Downstream of Orana bridge near the mouth of Marshalls Creek, the flood was less than a 5% event.

The flood inundated the township of Billinudgel and water flowed over the railway to the south of the town. Ten houses within the town had water through them. Within South Ocean Shores four houses were inundated along Narooma Drive by up to 100mm. Along Balemo Drive the flood did not reach floor level but several houses were close to being flooded and were affected by local runoff.

Within South Golden Beach 11 houses were flooded. At New Brighton approximately 40 houses were flooded and at least 3 houses have subsequently been raised. A further 30 two-storey dwellings in these two townships had water through their non-habitable ground floor areas and may have suffered minor damages.

No attempt has been made to quantify the flood damages from the 1987 flood.

3.5 Design Floods

The 1% and 5% design flows were established in the Hydrology Report (Reference 1). The same procedure was adopted to produce the 2% design flows.

Design rainfall data are given in Australian Rainfall Runoff (AR&R). At the time of the publication of the Hydrology Report the current AR&R was the 1977 version (Reference 9). The Hydrology Report analysed various methods for obtaining design rainfalls which were provided in Reference 9. It was concluded that the generalised charts given in the 1977 AR&R produced design rainfalls which were too low. A procedure utilizing data from the local stations was therefore adopted. Temporal patterns were obtained from analysis of several local storms.

Subsequent to the publication of the Hydrology Report a new version of AR&R (Reference 8) was produced. A comparison was made between the previously adopted values and values derived using the design information in the new AR&R. The new design information produced peak flows which are 28% lower mainly due to the substantial changes in the temporal patterns. Using the 1987 AR&R approach the design 1% flood levels would change by the following amounts:

Billinudgel	-255mm
New Brighton	+54mm

The increase at New Brighton is due to a change in the timing of the flow hydrograph relative to the ocean hydrograph and the double peaked shape of the flow hydrograph.

It was decided to adopt the old procedure as given in the Hydrology Report for the following reasons:

- the design rainfalls given in the new AR&R would appear to be too low, compared to data from the local rainfall stations,
- the new AR&R procedure would increase the return period of the May 1987 flood beyond a 1% event. This is not borne out by the limited amount of flood data available,
- the temporal pattern recommended for use over the Brunswick River is not considered representative of the actual storms which have occurred over the catchment.

In addition to the normal approach of using 1%, 2% and 5% "design" floods to investigate the development options, the recently experienced May 1987 flood was also used. The 1987 flood was used for the following reasons:

- most recently remembered major flood,
- largest flood at Billinudgel for over 80 years, approximating a 1% event,
- the theoretical design floods have high ocean levels combined with high river flows whilst the May 1987 flood had high flows and comparatively low ocean levels. Velocities are therefore higher and the various development/mitigation options could have different impacts with such a flood.

Flood levels near the mouth of the river are dominated by ocean levels. This issue was discussed in Reference 10 which suggests that the one meteorological event in this region is likely to produce rainfalls and ocean levels of approximately the same probability of occurrence. Thus, for example, 5% catchment flows and 5% ocean levels would occur in the same event to produce the 5% flood. The 5%, 2% and 1% design events were modelled in accordance with this assumption.

Reference 10 also suggests that for a catchment like the Brunswick River, maximum rainfall intensities would occur when a cyclone crosses the coast and peak surge tides would occur at the same time. The design flows and tides were therefore input to the Cell Model using this relative timing.

Due to the effects of storage and travel time within the catchment, the model indicated that the peak flow at the entrance occurred approximately four hours after the peak ocean level. The sensitivity of flood levels to the relative timing of peak flows and ocean levels was analysed in Reference 3.

Flows from the WBNM model for the three design floods were input to the Cell Model set up with existing topographic conditions. The results are shown as longitudinal profiles on Figure 13 and in Table 1.

All development options were compared to these base values.

TABLE 1

Flood Levels in Marshalls Creek

Existing Conditions

FLOODS Locations	1%	2%	5%	1987
Cell 74 - Billinudgel Industrial Est.	3.78	3.66	3.41	3.89
Cell 69 - D/S of Pacific Highway	3.16	3.04	2.84	3.18
Cell 73 - South Golden Beach	2.87	2.76	2.47	2.75
Cell 76 - Capricornia Canal	2.88	2.76	2.53	2.76
Cell 61 - Marshalls Ck-Yelgun Ck Conf.	2.88	2.76	2.50	2.73
Cell 57 - Sth Ocean Sh Golf Course	2.82	2.68	2.40	2.63
Cell 55 - New Brighton	2.74	2.59	2.30	2.52
Cell 54 - D/S of Orana Bridge	2.36	2.18	2.11	1.90

Note: The locations of the Cells are shown on Figure 6.

The base conditions referred to as "Existing Conditions" in Table 1 are different to the conditions which were present in 1987 when the flood occurred. The major difference was that in 1987 a bund was located across the upstream end of Capricornia Canal which limited exchange of water between the Yelgun Creek floodplain and Capricornia Canal. For the "Existing Conditions" and all subsequent conditions modelled, it was assumed that the bund was removed and the full width of the canal available to convey flow. Thus the calibration run for the May 1987 flood and the values given in Table 1 differ slightly.

3.6 The "Greenhouse" Effect

The "Greenhouse" effect has become one of the major environmental concerns in recent years. The Greenhouse effect could impact on the matters covered in this study in two possible ways - by increasing design flood levels and/or by accelerating beach recession.

An increase in design flood levels could result from an increase in design ocean levels or an increase in design rainfalls. Recent advice from the Bureau of Meteorology indicates that there is no intention at present to revise design rainfalls to take account of the Greenhouse effect, as the possible mechanisms are far from clear, and there is no indication that Greenhouse-type changes would in fact increase design rainfalls for major storms. With regard to design ocean levels, these would have to rise substantially to have any significant impact on flood levels within the area being considered for further development. In addition, the normal freeboard required by Council above the designated flood level, would provide some margin if there is any future rise in flood level. To provide a measure of the likely Greenhouse effects on the various development proposals considered in this study, two possible scenarios involving sea level increases of 0.2m and 0.5m have been run through the hydraulic model, and these are reported in Section 4.7.

The other possible manifestation of the Greenhouse effect - accelerated beach erosion - has been considered in the concept design of the eroding flood outlet. This is described in Appendix E.

4. DEVELOPMENT AND FLOOD PROTECTION OPTIONS

4.1 General

During the course of the study various combinations of development and flood protection options were examined for each of the three design floods and the May 1987 flood.

Each option was simulated by altering the model data to represent the physical changes involved in each case. This was achieved in the following manner:

- filling of the floodplain in a backwater area was simulated by reducing the storage dimensions of the relevant cell,
- change in flow area was simulated by altering the shape of the cross-section or if the connection was via a weir:
  - by altering the weir dimensions, or
  - by adding new flow paths by connecting affected cells.

Overall the model is reasonably flexible in its ability to simulate topographic changes.

4.2 Approach Adopted

The study brief required that the following options be examined:

a) The effects of altered hydraulic controls at Billinudgel and Mullumbimby.

b) The impact of filling in North Ocean Shores including bund walls at the northern boundary and the effects of the proposed lake.

c) The impact of filling within the golf course.

d) The impact of filling in the Holiday Village and/or a levee around the site at South Golden Beach.

e) Hard/soft flood outlets at:

   North Ocean Shores,
   Holiday Village,
   Sheltering Palms (as examined in the earlier Floodplain Management Study).

f) 3 runs looking at combinations of some of the above.

Provision was made in the model for further options (should they be required to be examined at a later date) such as - a bund around the golf course, a bund all along the southern limit of Ocean Shores, changes to the entrance rock walls at the mouth of the Brunswick River, changes to the rock walls at the mouth of Marshall's Creek, dredging of Marshalls Creek.

During the course of the investigation additional flood mitigation options were identified and investigated. These tended to be derivatives of the options listed above which were identified at the commencement of the study. Some options were also canvassed as a result of a public meeting held at Byron Bay on 6 May 1988 at which a range of options and combinations of options were presented.

Initially all the options were examined using a Development Plan already submitted to Council for consideration at that time. Using early results from the study, and taking into account environmental and social concerns, the proposed OSDC development was amended in December 1988 to the Development Concept Plan shown in Figure 3, whose floodplain management impacts have been considered herein.

The Committee also considered as part of this study, future development of two parcels of land adjacent to Balemo Drive owned by Mr J Mangleson.

The flood mitigation options which had been found to be practical for the original OSDC plan were re-examined and then modified to suit the new development proposals.

The following sections describe the various development and flood mitigation options and their impacts on existing flood levels. For the more important cases results at key locations are tabulated in Appendix B.

The first section describes the features of the proposed Development Concept Plan and Mangleson developments as far as they affect flood behaviour. The hydraulic model was modified to represent the physical dimensions of the developments by changing the model geometry. The impacts of the developments are then described in the absence of any mitigation options.

The next section (Section 4.4) discusses the full range of individual flood mitigation options examined. Some cases relate to existing conditions rather than with the proposed developments in place. In the more important cases quantitative results are presented. In others, the results are interpretted qualitatively drawing on the known results from other cases.

Section 4.5 draws together the results from the individual cases, to present several combinations of development and flood protection measures, which achieve the objectives of matching development proposals with protection of existing development.

The final two sections discuss the staging of the flood mitigation works (Section 4.6) and the modelling of the Greenhouse impacts (Section 4.7).

Figure 14 shows the locations of the main flood mitigation options which were considered in this investigation.

4.3 Modelling of the Development Proposals

4.3.1 Development Concept Plan

The proposed development of North Ocean Shores and adjoining areas is shown on Figure 3. A detailed description of the development is given in Appendix C, together with the assumptions made in modifying the hydraulic model to determine the impacts of the development.

The proposal encompasses the following features:

- 16ha lake connected to Capricornia Canal,
- residential development surrounding the lake,
- tourist development north of the lake,
- residential development adjoining Capricornia Canal,
- tourist development of the land between New Brighton and South Golden Beach (Holiday Village),

- a residential site within the South Ocean Shores Golf Course,
- a recreation/open space development within the South Ocean Shores Golf Course,
- a recreation/open space development adjoining the northern bank of Marshalls Creek, downstream of the Pacific Highway,
- residential/educational development on Shara Boulevarde,
- town centre site at the junction of Shara Boulevarde and the Pacific Highway,
- a motel site on Shara Boulevarde,
- rural residential development on the hills overlooking Yelgun Creek,
- widening of the creek between Capricornia Canal/Yelgun Creek and Marshalls Creek.

Since the proposed OSDC development is in concept form at this stage, there are no details on the exact extent of any filling or the proposed layout of buildings and structures. In the absence of this detail the maximum practical extent of filling was assumed. Details of the extent of the filling which was assumed in each area are given in Appendix C. The actual development when it is refined further, will inevitably involve less filling.

4.3.2 Development of the Mangleson Land

The land owned by Mr J Mangleson considered in this study consists of two sites shown on Figure 2 as Sites A and B. Site A is adjacent to Balemo Drive immediately downstream of the Pacific Highway. The site is heavily vegetated and is flood liable in a 1% event. Site B is adjacent to Marshalls Creek at the downstream end of the man made lake and is separated from Balemo Drive by the Golf Course.

Preliminary assessment of Site B indicates that development of this site may produce significant hydraulic impacts which cannot readily be mitigated. This is because the land is relatively low-lying and during floods a considerable proportion of the total flow passes across the site and the adjoining Golf Course fairways. Access to the site would also be difficult during floods. For these reasons Site B was not considered further as part of this study.

4.3.3 Impact on Flood Levels

The results (Option A: Note that model results are given in Appendix B) show that, in the absence of any mitigation measures, a development and flood protection package consisting of the Development Concept Plan, Site A of the Mangleson land and a levee around South Golden Beach, would raise flood levels in the adjoining areas by up to 260mm. The actual development will produce less adverse impacts because of the likely reduction in fill volumes.

4.4 Modelling of Individual Flood Mitigation Options

4.4.1 Existing Development with the North Ocean Shores Flood Outlet

This option evaluated the impacts of a flood outlet at North Ocean Shores under topographic conditions prevailing at the time of the 1987 flood. As discussed earlier, the 1987 conditions differ from the "Existing Conditions" used elsewhere in this report because the bund in Capricornia Canal was assumed to be still in place.

Two outlet conditions were evaluated:

- an outlet 200m wide with a hard crest at 2.3m,
- an eroding outlet 200m wide with a hard crest at 1.9m, but filled to 2.3m with sand.

The results show little difference between the two outlets. The change in flood levels at two key locations are given below.

	Hard Crest	Eroding Crest
South Golden Beach	-37mm	-19mm
New Brighton	-21mm	-4mm

The presence of either of these outlets at the time of the May 1987 flood would have therefore had a negligible impact on peak flood levels.

The minor impact on flood levels of either of these entrances can be traced to the present tortuous flow path. The waterway link between Marshalls Creek and the outlet is hydraulically inefficient at the present time. This is because the Yelgun Creek bridge and downstream channel are very restricted and there was an earthen weir restricting flow into or out of Capricornia Canal. The link between the canal and the outlet is also heavily vegetated.

The slight difference in performance of the two types of outlet for this flood can be traced to their different operating performance for different depths of overtopping. For shallow depths, a hard crest is more efficient than a sand crest because of its lower roughness value. However, as the sand erodes from the eroding crest with greater depths and volumes of flow, then it becomes more efficient. In the May 1987 flood the relatively limited amount of flow that could get to the outlet meant that an eroding crest was less efficient.

4.4.2 Levee Options

South Golden Beach

The township of South Golden Beach is divided by Capricornia Canal into eastern and western sections. Eleven single storey dwellings as well as up to 20 two storey dwellings are inundated in a 1% flood. The banks of the canal form levees which give protection up to a 20% flood. There is a higher private levee around a motel in the eastern section.

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There are few engineering or environmental problems associated with constructing additional levees or raising existing ones to protect South Golden Beach. Results from the hydraulic model show that the impact of constructing levees to protect both sections of South Golden Beach is to raise flood levels at South Ocean Shores by 13mm and by 12mm at New Brighton in a 1% flood (Option B).

Construction of levees around South Golden Beach can be carried out independent of the other development and flood mitigation works. A preliminary design is currently being carried out and this will be covered in a separate report by Council.

Levee Around Billinudgel

That part of the township west of the railway is inundated in a 20% flood. In the May 1987 flood there was up to 1.5m of water through some of the buildings. The industrial estate east of the railway line is located on fill at approximately the 1% flood level (3.8m).

The impacts of a constructing a levee around western Billinudgel were assessed assuming existing conditions elsewhere. The model results (Option C) showed that in a 1% flood this would raise levels by 67mm immediately upstream of the township and have no significant impact downstream of the Pacific Highway. Preliminary design of a proposed levee is currently being carried out and this will be covered in a separate report by Council.

Construction of a levee at Billinudgel can be carried out independent of the development and mitigation works proposed downstream of the Pacific Highway in Ocean Shores.

Flood Protection for New Brighton

Up to 60% of the flood liable buildings in the Marshalls Creek floodplain lie within the township of New Brighton. Flood protection for New Brighton could be achieved by one or more of the following measures:

- raising floor levels,

  The raising of floor levels is practical for the majority of non-brick dwellings. It has been carried out successfully in many locations and costs approximately $10-15 000 per house. At New Brighton preliminary assessment indicates that up to 70% of the dwellings flooded in the 1% flood could be raised. The total cost of implementing this measure would be approximately $300 000. Whilst protecting the buildings from flood damages, there would still be damages such as "back yard" damages and vehicle damages. Additional measures would be required for the remaining dwellings.

- reducing flood levels,

  Flood levels in Marshalls Creek can be reduced by various measures such as removing creek constrictions, dredging or by construction of flood outlet(s). These are discussed in later sections. However, although these measures would improve the situation, they would not eliminate the flood hazard from New Brighton.

- levee construction,

  Construction of a levee around New Brighton would eliminate flooding from the township up to the height of the levee. However, there are problems in defining a satisfactory levee alignment. Complete levee protection would involve a levee around Casons Road and along River Street to North Head Road (downstream of Orana Bridge). It would be over 1.5 kilometres in length and is unlikely to be cost-effective. Furthermore there would be accompanying problems of local drainage and access, together with possible adverse hydraulic impacts upstream.

  Preliminary analysis suggests that a levee along Park Street and River Street to Orana Bridge may be practical. This would protect approximately 50% of the houses subject to flooding in New Brighton as well as permitting future development within the levee. The areas excluded upstream (Casons Road) and downstream (south of Strand Avenue) could be evaluated on a house by house basis and other measures such as house raising considered.

  No analyses were carried out in order to assess the hydraulic effects of possible levee construction. If a partial levee were to be constructed in conjunction with the full development of the Holiday Village site, there would be no significant additional hydraulic impact.

4.4.3 Change in Hydraulic Controls at Billinudgel or Mullumbimby

The hydraulic model was used to determine the effects of lowering the railway line at Mullumbimby or widening the Pacific Highway bridge and Railway bridge at Billinudgel. Whilst these options are outside the area proposed for development (see Section 4.3) they were undertaken in order to ensure that the development would not preclude other possible flood mitigation measures outside the immediate area. At Mullumbimby it was assumed that the railway was lowered by 1m from the rail bridge over the Brunswick River to the road crossing in the town. At Billinudgel the waterway areas of the two bridges were doubled.

The results produced in Appendix B (Options D & E) show that the altered hydraulic controls at Mullumbimby and Billinudgel have little impact upon levels in the Marshalls Creek area. Therefore, by undertaking the proposed development of the area, these options could still be employed at a later date and they would not significantly affect the existing development.

4.4.4 Flood Outlets at Various Locations

Three alternative flood outlets were examined (see Figure 14):

- at Sheltering Palms,
- near Holiday Village,
- within North Ocean Shores.

The concept behind a flood outlet is that once the flood level reaches the crest of the outlet, water will escape to the ocean. This will mean that less water will flow down the main creek and thus flood levels will be reduced. The main hydraulic features associated with a flood outlet are its location, the dimensions of the structure and the intake channel leading to the outlet. The outlet should be in such a position that it can readily accept floodwaters from the catchment and be of sufficient size to provide a benefit. This is largely dependent upon the height and width of the weir - the lower the crest the greater the impact. However, there is the risk with such structures that during the occurrence of high ocean levels sea water may enter and adversely affect development or the environment. An appropriate management strategy is thus necessary to minimise such problems.

In the first instance the outlets were considered to be hard-crested structures 200m wide and at 2.3m AHD. The level of 2.3m was determined to be a reasonable level to provide flood mitigation benefit without being so low as to be regularly overtopped with high ocean levels. The design 1% ocean level is 2.6m AHD and the design 5% ocean level is 2.3m AHD. The structure would be overtopped for ocean levels exceeding 5% unless appropriate management strategies were adopted. This matter is addressed in some detail in a separate report.

The Sheltering Palms outlet would not significantly reduce flood levels at or upstream of New Brighton as modelling indicates that insufficient water would flow over the structure. This is because the outlet is near the Brunswick River entrance and flood levels are lower than further upstream. This option was thus not considered any further after the initial assessment.

The Holiday Village outlet lowered flood levels below existing conditions in the flood affected areas of New Brighton and South Ocean Shores with the proposed development and flood protection package in place. However, a major drawback of this outlet was that a channel would have to be constructed across River Road and the playing fields at New Brighton. It would also be costly to construct and maintain. For these reasons this option was also rejected.

A similar hard-crested outlet at North Ocean Shores was found to largely negate the adverse impacts of the development and lower flood levels below the existing levels in parts. A weir crest level of 2.1m was also modelled in order to assess the impact of lowering the crest level while accepting an increase in the frequency of overtopping. The lowering to 2.1m did have significant additional benefits over the 2.3m crest (Options F & G). Nevertheless, it was not considered further at this time because of concerns that it would not be viable given the increased likelihood of overtopping from the ocean.

An eroding weir crest was also simulated (Option H). Under this option it was assumed that a hard-crested weir would be constructed to 1.9m, but, would normally be filled to RL 2.3m with sand. Thus once the crest was overtopped the sand would erode, lowering the crest level and providing further mitigation benefits. The hard crest at 1.9m ensured that the opening would not erode to a lower level and permit salt water inflow after the flood event.

One further drawback of a soft-crested weir is that it would require additional maintenance to re-establish after a flood and may be susceptible to wind-erosion. An eroding weir with an initial crest level of 2.3m does have advantages (40mm at Capricornia Canal) over the fixed crest at 2.3m. However the gains were not as significant as expected as the hard-crested structure, because of its smooth surface, allows a larger flow than a sand-crested structure with the same dimensions.

From an environmental viewpoint, the construction of a flood outlet would require strict controls. These have not been fully evaluated as yet and will be reported separately. The main issues are summarized below:

- possibility that the structure could be outflanked unless appropriate safeguards were implemented,
- maintenance costs,
- visual impact of the structure,
- affect upon coastal behaviour,
- impact on water quality and flora.

More information on most of these issues is given in Appendix E.

4.4.5 Widening of Orana Bridge

Examination of the longitudinal profiles (Figures 9 and 13) shows that Orana Bridge acts as a significant hydraulic control during floods. Flood levels rise steeply upstream of the bridge as the water backs up behind the bridge. Widening of the bridge would lower flood levels immediately upstream but the effect would reduce with distance. Widening of the structure could be accomplished by expanding the main waterway opening and raising of the eastern approaches (with openings underneath).

Widening of the bridge results in significant reductions in flood levels upstream (70 mm at New Brighton reducing to 10mm at the Pacific Highway). However, it would be at considerable cost and its effect is largely local. It could however be considered in combination with other options, particularly if dredging does not proceed.

From an environmental viewpoint the widening of Orana Bridge would appear to have little impact.

4.4.6 Dredging of Marshalls Creek from near its Mouth to the Junction with Yelgun Creek

Currently Marshalls Creek has an average water depth of less than 1m. The bed level of the creek varies between -0.5m and -1m AHD. Dredging of the creek will increase the waterway area thus lowering flood levels. The effects of several depths of dredging to a maximum depth of -2.5m AHD were simulated (Option I). It was found to produce a significant reduction in flood levels of up to 350mm along the dredged reach for the 1987 flood with a benefit of 20 mm at the Pacific Highway. However, for the 1% flood the gain is only 170mm. This is due to the influence of the high ocean level at the downstream end.

Dredging of Marshalls Creek has been proposed in the past, although generally for financial gain rather than for flood mitigation purposes. Suggestions have also been made that the creek is silting up and worsening the existing flood problem.

Examination of historical air photos (dating back to 1947) shows that even at that time there were significant shoals present along the creek. There are few topographic surveys available for Marshalls Creek which can be used to assess the extent of siltation. One of the problems with comparing surveys taken at different times is that the surveys do not take sections at identical locations. However, for the Flood Study (Reference 2) a hydrographic survey was undertaken in 1983 for Marshalls Creek which included 26 cross-sections.

As part of this present investigation, 15 of these sections were re-surveyed in 1988 at precisely the same locations as used previously. Comparison of these surveys showed that there were no appreciable changes in the cross-sections from 1983 to 1988. However this does not mean that in the medium term (40-50 years) there have not been any changes. It does show that any changes that have taken place are likely to be small.

The advantage of dredging is that it could be undertaken at nominal cost to Council or in fact at some financial gain. Preliminary estimates suggest that of the order of 300 000m^3 of material could be obtained. If the dredging were to be undertaken in a short period of time (2-4 years) the local market would not be able to absorb that quantity. However, if major development was taking place at the same time, the surplus could be readily absorbed as fill for the proposed development.

Dredging of the creek would have to be undertaken under strict guidelines set by the relevant authorities. Issues which would require examination include:

- the maximum possible depth of dredging,
- the shape of the proposed channel,
- the effect of dredging on mangroves and aquatic life,
- the pollution effects of the dredging operation,
- the effects of dredging upon the tidal regime and saline intrusion.

As part of this study Professor Warner of the University of Sydney undertook a geomorphological examination of Marshalls Creek. His report is given in Appendix D. A separate report in draft form canvasses some of the issues in relation to the likely impacts of dredging on the tidal regime.

An additional benefit from dredging is that it offers the possibility of an improvement in the local drainage within South Ocean Shores. This issue was raised at the Public Meetings but falls outside the scope of this study. However, if the drainage channels within South Ocean Shores were improved, in conjunction with the improved capacity of the creek due to dredging, it is likely that most of the present drainage problems would be solved.

4.4.7 Separation of the Marshalls Creek from the Yelgun Creek Floodplain

The Yelgun and Marshalls Creek floodplains were originally only connected by a fibrous creek network. This was considerably altered with the construction of Capricornia Canal. The question of separating the two systems was investigated to see if it would have any flood mitigation benefit. This was modelled by assuming that a bund would be used to separate the two creeks. Flow from Yelgun and Billinudgel Creeks would flow northwards or over the proposed flood outlet. Total separation of the two systems was found to be a disbenefit because the Yelgun creek floodplain is filled by floodwaters from Marshalls Creek during the rising stage of a flood. Thus cutting off the Yelgun Creek floodplain decreases the available storage volume and thus increases the flood peak along Marshalls Creek.

A flap-gated system was also investigated which would permit water to flow into Yelgun Creek during the rising stage of the hydrograph and hold it there during the peak of the flood. This proposal had no significant flood mitigation benefit.

An alternative was to look at expanding the present opening to compensate for the construction of a high level bridge and approaches to provide flood free access to Holiday Village. Waterway widths between 20m and 60m were evaluated. It was found that a 20m width was adequate to maintain a good connection between the two systems.

From an environmental viewpoint any major change in the connection between the two river systems would require careful examination. Issues which would arise include:

- the effect on saline intrusion,
- the effect on water quality and stratification of the Capricornia Canal.

4.4.8 Provision of Local Floodways

Floodways can be used as an option to reduce flood levels. These may take the form of an excavated channel parallel to the main channel or as a separate channel which eliminates creek meanders. In Marshalls Creek two floodways were considered (see Figure 14):

- across the South Ocean Shores Golf Course,
- immediately downstream of the Highway.

Other locations for floodways would eliminate significant stands of mangroves and would therefore be undesirable from an environmental viewpoint.

Floodways are not useful as flood mitigation measures on their own in this area but could be considered as part of an overall scheme. The South Ocean Shores floodway was considered as part of one scheme but when further survey was obtained, it was found that little additional lowering of ground levels could be achieved in this area without causing salinity problems. The other floodway downstream of the Highway has been included in the preferred combination of options discussed in Section 4.5.

4.4.9 Entrance Training Walls

The entrance of the Brunswick River and the tributaries that join the river near the entrance - Marshalls Creek and Simpsons Creek - are controlled by training walls. These walls were built in the early 1970's, and are constructed of local crushed rock. Their purpose is to ensure a navigable entrance for the Brunswick River. They have been the subject of some criticism by local residents due to their perceived impact on flood levels. Because of concerns raised at the Public Meetings the hydraulic model has been used to determine the impacts on flood levels of lowering or removing these walls.

The natural width of the mouth of Marshalls Creek is approximately 600m. This has been limited to two relatively narrow entrances at each end by the construction of a 500m training wall at approximately 1.7m AHD. In addition to this wall, which runs east/west, there is a 300m long training wall running off near the eastern end in a north/south direction. This wall was originally higher, but was lowered to 0.4m after it appeared to impede flood flows.

To assess the impacts on flood levels of modifying the walls, the following modifications were made - the north/south wall was removed completely and the east/west wall was lowered to 0.4m. This meant that it could still carry out its main purpose of re-directing tidal flows and thus ensure a stable entrance.

The results are shown as Option J in Appendix B. In the immediate vicinity there is some minor impact on flood levels, but this is reduced to a negligible amount at New Brighton.

4.4.10 Other Options Examined

During the course of the study several other possible flood mitigation options were considered. These arose largely from the regular meetings of the Technical Sub-committee of the Floodplain Management Committee or from the public meeting held in May 1988. These options were not in general analysed using the computer models.

The issue of dams in the upstream catchment was raised as a possible means of reducing the flood peak. Dams are a useful flood mitigation option, but, are not considered a viable option for reducing peak flows at Ocean Shores for the following reasons:

- flood mitigation dams are extremely costly,
- there are no suitable sites which would allow one dam to control the majority of flow in Marshalls Creek,
- there are significant environmental problems arising from the construction of dams.

Stream clearing and re-alignment are again legitimate means of flood mitigation. The principle behind stream cleaning is that the velocity of flow will be increased thus reducing the peak flood levels. With stream re-alignment the water is conveyed to the ocean by a shorter route. Stream re-alignment was consider in part in the previous section under floodways. However these were only overbank re-alignments rather than constructing of a new channel. Marshalls Creek follows an extremely sinuous path downstream of the Pacific

Highway with several 180° loops.

It is also heavily overgrown with several fallen trees and other vegetative debris constricting the flow. However re-alignment and stream clearing were not considered viable options for the following reasons:

- any significant change to the existing channel could have significant environmental consequences,
- it would be expensive to undertake and there would be additional annual maintenance costs.

At the onset of the study the question of removing the rock walls at the mouth of Marshalls Creek was raised. These walls were constructed to ensure a navigable entrance to the Brunswick River and thus any changes may affect the navigability and hence the local fishing industry. Removal of the walls may have a limited flood mitigation benefit along the lower part of Marshalls Creek, but it was considered that the effects would be insignificant upstream of Orana bridge.

One proposal suggested by Mr Hall, who lives upstream of Billinudgel, was to construct a flood channel to bypass the more tortuous parts of Marshalls Creek. This was presented as a design concept rather than a detailed plan. The basis of the concept was that a flood channel would be constructed which would convey floodwaters from Billinudgel direct to the ocean near Holiday Village. It was proposed as a solution to the existing flood problems rather than a solution for the impacts likely to be caused by the proposed development.

The flood channel was not considered a viable flood mitigation measure for the following reasons:

- in order to act as significant flood mitigation measure it would require a lined channel with high velocities. This would present significant environmental concerns as it would destroy a large part of the mangrove stands,
- there would be problems with the fluvial access between Yelgun and Marshalls Creek. A bridge or bridges would also be required for vehicular and pedestrian access,

- as the land within the channel of Marshalls Creek is generally lower than 1m AHD, a levee would be required to confine the floodwaters to the channel. The water level within the channel would be in excess of 2m AHD and if the levee failed or overtopped, there would be considerable damage,
- it would be likely to have significant impacts on the tidal hydraulics and in particular could affect the navigability of the Brunswick River entrance,
- there are significant problems associated with constructing a new ocean entrance particularly if it was to be a low-level opening at say -1m AHD,
- it would be expensive and most unlikely to be cost effective.

4.5 Proposed Developments with Alternative Combinations of Flood Mitigation Options

4.5.1 General

The preceding sections describe a number of different flood mitigation measures. None of them in isolation can meet the objective of this study, this being to reduce flood damages to existing development and mitigate the adverse impacts caused by the filling of the floodplain. Various alternative combinations were therefore examined to satisfy this objective.

As noted earlier, the proposed developments changed during the course of the study and the majority of the analyses were undertaken using an earlier version of the OSDC proposal. However, the results obtained previously were useful in determining which mitigation measures would be viable for the new development concept. The results of the individual measures are summarized in turn below leading to a number of alternative combinations to meet the stated objective.

Of the three possible ocean outlets considered, the most attractive is the North Ocean Shores location. The Sheltering Palms option is not hydraulically effective and the Holiday Village site would involve the removal of the New Brighton playing fields and considerable road works. An eroding crest at 2.3m was found to be more efficient than a hard crest at 2.3m. The North Ocean Shores flood outlet would lower flood levels within North Ocean Shores, but would have little impact at New Brighton. Thus a further mitigation measure must be employed to lower levels at New Brighton. This could be accomplished by widening Orana bridge, by dredging, or possibly by a combination of the two. The dredging option offers the following advantages over widening the bridge:

- dredging benefits extend further upstream,
- dredging could be undertaken more economically than widening Orana bridge.

Further work is required to ensure that dredging will be acceptable on environmental grounds.

In order to reduce the flood impacts at the proposed Town Centre site local floodways were considered. The golf course floodway was rejected as the ground level is currently around 1.3m AHD and has little potential to be lowered further. A floodway on the northern bank of the creek was therefore proposed. The land at this location is also very low, however, hydraulically it can be improved significantly by clearing the overbank vegetation.

Changing of the Yelgun Creek connection between Capricornia Canal and Marshalls Creek was found to have minor hydraulic impacts, except when it was bunded off completely, when it was found to cause a significant adverse impact. The width of the channel was therefore expanded to 20m in order to provide an efficient hydraulic connection between the two creek systems. This was required as the overbank flow link will be much reduced under developed conditions.

Various combinations of flood mitigation works were developed which would reduce or eliminate the existing flooding problem, and would eliminate the adverse effects of the proposed developments. The following mitigation options have been included in each of the alternative combinations:

- **Floodway downstream of the Pacific Highway**

  It was assumed that a floodway would be constructed on the northern overbank area of Marshalls Creek between the limit of the proposed development and the creek. This would be compatible with the recreation/open space zoning. The floodway would extend downstream from the Pacific Highway for approximately 500m and would be constructed at 1.0m AHD. It was assumed that it would be regularly maintained in order to be hydraulically efficient.

- **Widening of Yelgun Creek Bridge**

  Because of the proposed raising of the road across Yelgun Creek to Holiday Village to permit flood free access, it was necessary to expand the waterway area within Yelgun Creek, and at the bridge, to permit a good hydraulic connection between Capricornia Canal and Marshalls Creek. Widening to 20m was found to be adequate.

A number of different combinations of flood mitigation works can be used to achieve the study objective. Three possible combinations are discussed in the following sections. Environmental, economic and social factors should be used to determine the optimal combination.

4.5.2 Combination 1 - Dredging & NOS Flood Outlet (Option G)

The first combination includes the above measures, construction of the Flood Outlet at North Ocean Shores, together with dredging of Marshalls Creek to -2.5m AHD. The dredging was assumed to commence a reasonable distance upstream of the training walls on the Brunswick River to the confluence with Yelgun Creek.

The results for this combination of mitigation options with the full development in place are given as Option G (variations due to differing depths of dredging are given as Options K & L). There is a slight increase in level at Billinudgel in one flood but a reduction in all the other design cases. Downstream of Orana Bridge there are minor increases in level for high ocean conditions. However, a substantial benefit accrues in this area for a 1987-type flood. New Brighton achieves a significant level of benefits for the full range of floods.

4.5.3 Combination 2 - No Dredging, but Orana Bridge Widened & NOS Flood Outlet (Option M)

The second combination consists of the two common measures outlined above, construction of the Flood Outlet at North Ocean Shores, together with widening of Orana Bridge in the manner described in Section 4.4.5. It was found that this combination produced less overall benefit than the one involving dredging, with a reduction in level of 109mm at New Brighton compared with 137mm in the dredging case (see Option M in Appendix B). For the 1987 and 5% floods there are increases in flood levels above existing levels at the Marshalls Creek/Yelgun Creek confluence. However there would be no impact on existing houses in this area. Widening of Orana Bridge does however produce an increase in levels downstream (85mm) in a 1987 type flood. The possible impact of this on existing houses is described in Section 4.5.5.

Other combinations involving lesser dredging (-1.5m and -2m AHD) in combination with widening of Orana Bridge have also been run and the results are given in Appendix B (Options N & O).

4.5.4 Combination 3 - Dredging, Orana Bridge Widened & NO NOS Flood Outlet (Option P)

This third combination consisted of the two common measures together with dredging of Marshalls Creek to -2.5m and widening of Orana Bridge. This option was investigated to determine whether the adverse hydraulic effects of the proposed developments could be mitigated in the absence of a Flood Outlet in North Ocean Shores. It is possible that the flood outlet could be rejected on environmental grounds although it is satisfactory on technical grounds (see Appendix E).

The results are presented in Appendix B for the full range of floods (Option P). For the 1987 flood, levels are 294mm lower than existing conditions at New Brighton and 217mm lower at the Golf Course. For all the floods there are useful reductions in flood levels at New Brighton and in South Ocean Shores. There is a minor increase near Yelgun Creek for the 1% flood. In Capricornia Canal levels are increased, but with protection of South Golden Beach likely, the only adverse effect will be to increase the fill level for the new development in that area. Downstream of Orana Bridge the design flood levels are increased, mainly due to the widening of Orana Bridge, but also to the greater penetration of storm surges after dredging.

Although the results show that the development effects can be satisfactorily mitigated in the absence of a flood outlet at North Ocean Shores, provision of such an outlet is still desirable from a hydraulic viewpoint. This is because it does provide another flow path to the ocean and it does mean that there is less reliance on dredging to solve the flooding problems in the area. It is essential that either dredging or the flood outlet proceed. None of the other measures investigated can compensate for the new development and lower flood levels below existing levels.

4.5.5 Compensatory Works

Each of the three combinations of works described in the preceding sections will produce an increase in flood level downstream of Orana Bridge under certain circumstances (Appendix B, Options G, M & P). For the combinations which include dredging (see Sections 4.5.2 and 4.5.4) increases occur for the design floods but not for 1987-type floods. The increases are caused by the greater upstream penetration of ocean storm surges after dredging. The increase in level can be up to 42mm.

For the combination in which there is no dredging, but Orana Bridge is widened (Section 4.5.3), a 1987-type flood produces an 85mm rise downstream of the bridge. This is because under existing conditions Orana Bridge causes the floodwaters to back up reducing the peak of the flow downstream and hence the level. When the bridge is widened to reduce upstream flooding around New Brighton, the floodwaters get away faster and produce higher peak flows and levels downstream of the bridge.

A detailed assessment of the properties in this area was carried out to determine the impact of this increase in flood level. It can only affect properties downstream of the bridge and Strand Avenue. Within this area there are approximately 30 dwellings, however many of them are of two storey construction. The habitable first floor levels tend to be at least 2m above the 1% flood level. For a 1% flood the ground floor levels would be inundated by approximately 0.5m of floodwaters. These premises are designed for minimisation of flood damages by not having permanent fixtures in the ground floor area. In fact in general their building approvals require that the ground floor area be non-habitable and residents remove property upstairs when there is imminent danger of flooding. The potential effect on these properties in the event of a small increase in flood levels during storm surge events, is considered to be minimal, and has not been considered further.

Of the 30 dwellings in the area, 8 are of single storey construction. In a 1% flood these will be inundated by up to 0.4m. The potential increase in flood level of 30mm in a storm surge event will not significantly increase the amount of flood damages. However, for a lesser flood, which would not have come inside a particular house prior to the works, but would have just come in after, the increase in damages could be significant.

Construction of a levee around New Brighton is discussed in Section 4.4.2. Preliminary indications are that a levee to protect so few houses is most unlikely to be cost-effective. Other alternatives canvassed, such as house raising, appear more attractive. Of the 8 dwellings at least 3 are slab on ground and would be difficult to raise.

All of these dwellings, both single storey and two storey, are better off for the dredging combinations for the more common 1987-type floods. It is considered that, on balance, the dwellings downstream will be a little worse off in storm surge dominated events, but for the more common floods originating from high rainfalls over the catchment, they will be better off. Depending on the finally adopted scheme of flood mitigation works, it may be appropriate to consider some form of flood protection for these properties.

4.6 Staging of the Flood Mitigation Works

As the development proceeds, it will be necessary to ensure that the flood mitigation works are implemented systematically in parallel with the development. If this is not done, there is a danger that situations could develop where flooding was exacerbated in the short term. The following guidelines are given with regard to the staging process:

- filling of the land adjoining Shara Boulevarde or Mangleson Site A must be carried out in conjunction with clearing of the floodway downstream of the Pacific Highway,
- filling of the North Ocean Shores site and excavation of the lake must be carried out in conjunction with construction of the flood outlet and waterway link to Capricornia Canal,
- development of the Holiday Village site (including the Crown Land) should occur with dredging of Marshalls Creek in the vicinity of New Brighton,
- widening of the Yelgun Creek waterway connection must be undertaken in conjunction with any raising of River Road between the Canal and New Brighton,
- limited dredging will be required to mitigate the adverse effects of filling within the South Ocean Shores Golf Course.

The staging set out above applies for the implementation of Combination 1 described in Section 4.5.2. The staging will need to be fine tuned if another combination is to be implemented.

4.7 Modelling of Greenhouse Impacts

Preliminary analyses were undertaken of the impacts on flood levels which would occur if the mean sea level increased due to the "Greenhouse Effect". The following increases in mean sea level were considered:

+0.2m

+0.5m

The results for the 1% and 1987 floods are given for existing conditions and for Combination 1 (see Section 4.5.2) in Appendix B (Options Q,R,S & T).

In summary, for a 0.2m sea level rise, the results show that for both the 1% and 1987 floods, after implementation of Combination 1, there will still be a reduction in flood levels compared to existing conditions except downstream of Orana Bridge. For a 1987-type flood there will still be a reduction of 185mm with a 0.5m increase in sea level. As noted earlier, dredging has the effect of increasing the penetration of high ocean levels. Thus for the 1% flood with a 0.5m greenhouse rise, there is a greater increase in levels in most areas after the dredging than under existing conditions.

5. REFERENCES

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Australian Rainfall and Runoff, 1977.

10. Griffin A G
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23rd Flood Mitigation Conference, Kempsey, 1983.

FIGURES

FIGURE 1

LOCALITY PLAN

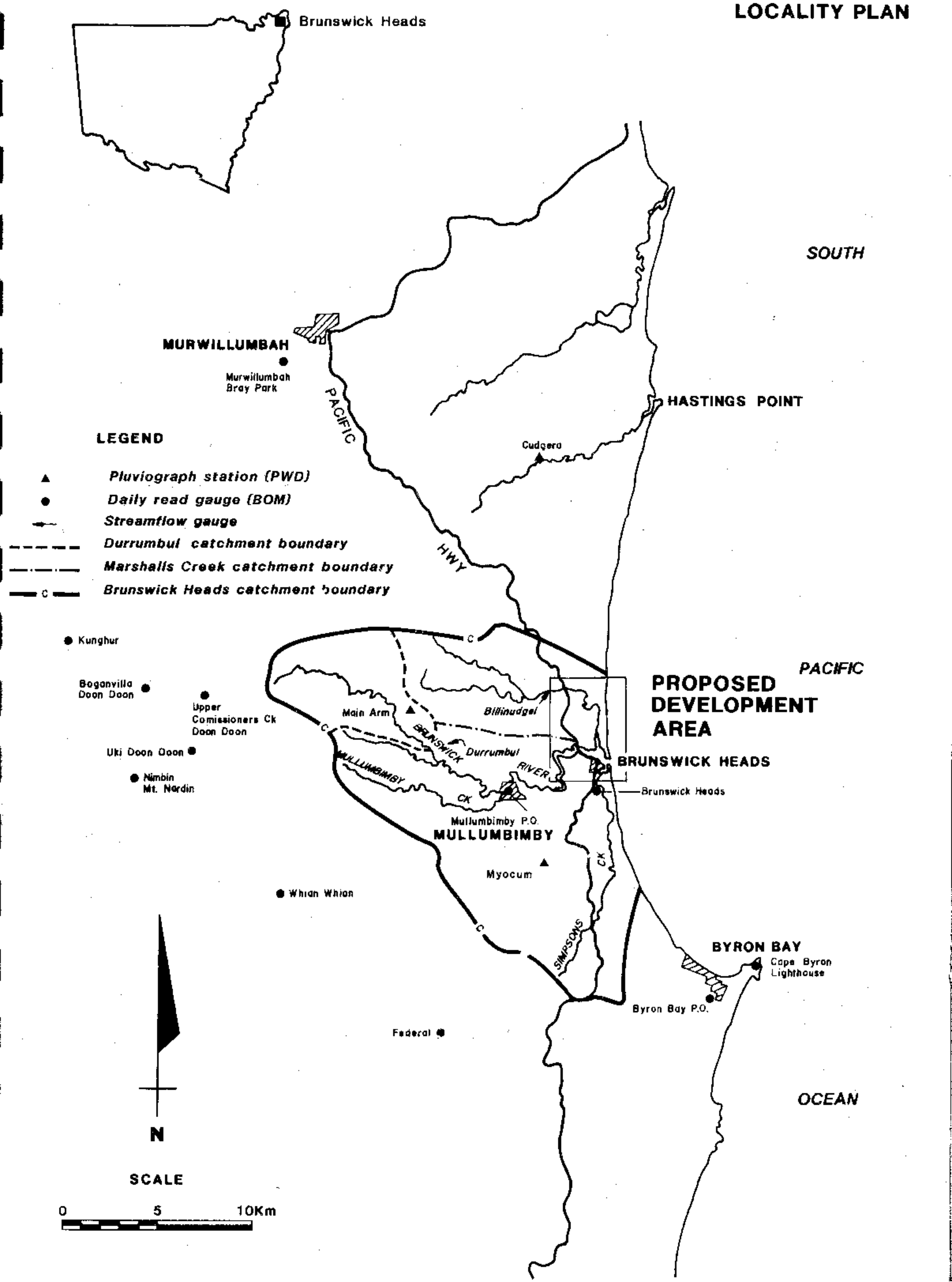


Fig 2

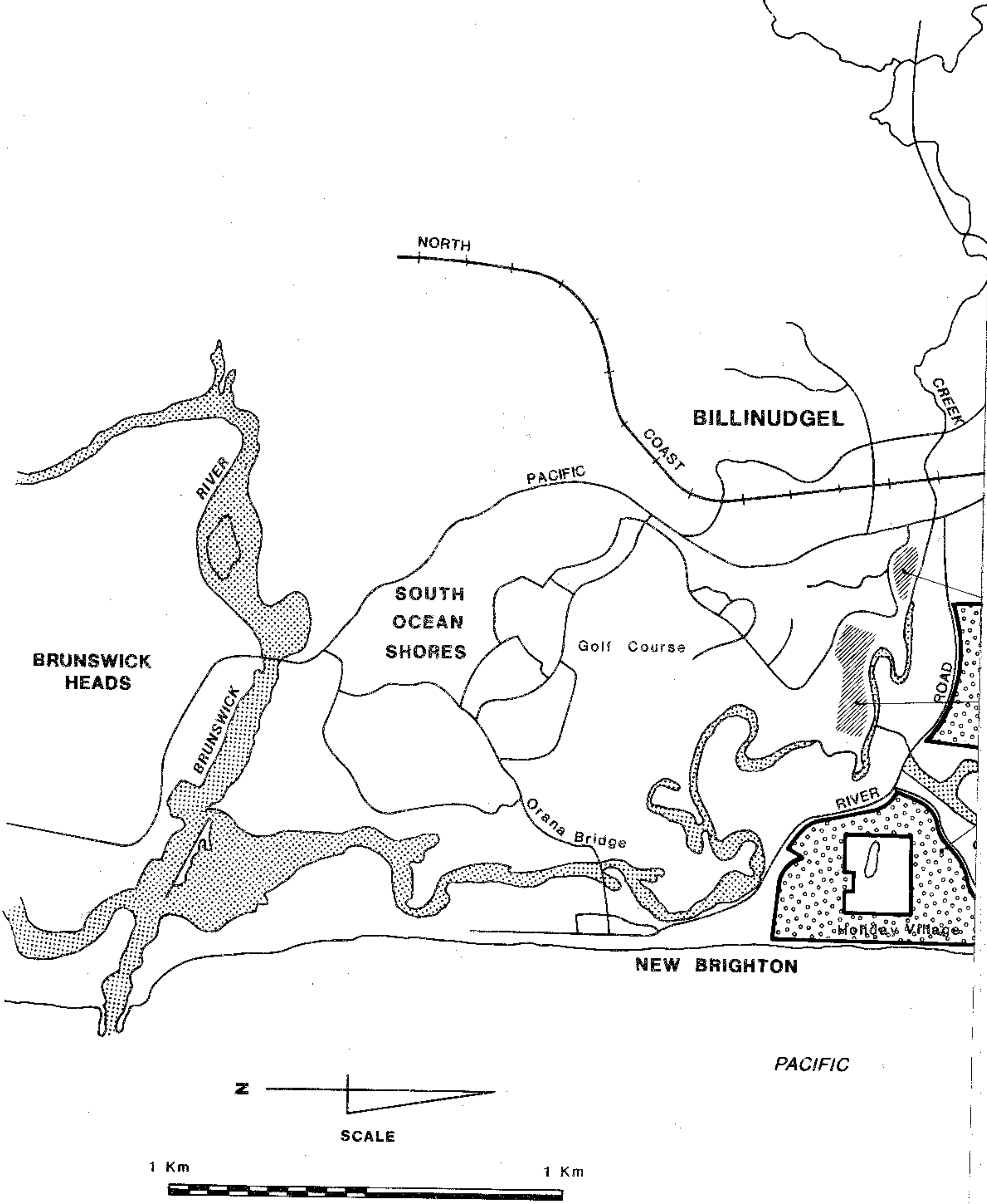

NORTH
COAST
PACIFIC
BILLINUDGEL
CREEK
ROAD
SOUTH
OCEAN
SHORES
Golf Course
BRUNSWICK
HEADS
RIVER
BRUNSWICK
Orana Bridge
RIVER
NEW BRIGHTON
PACIFIC
N
SCALE
1 Km
1 Km

FIGURE 2

STUDY AREA

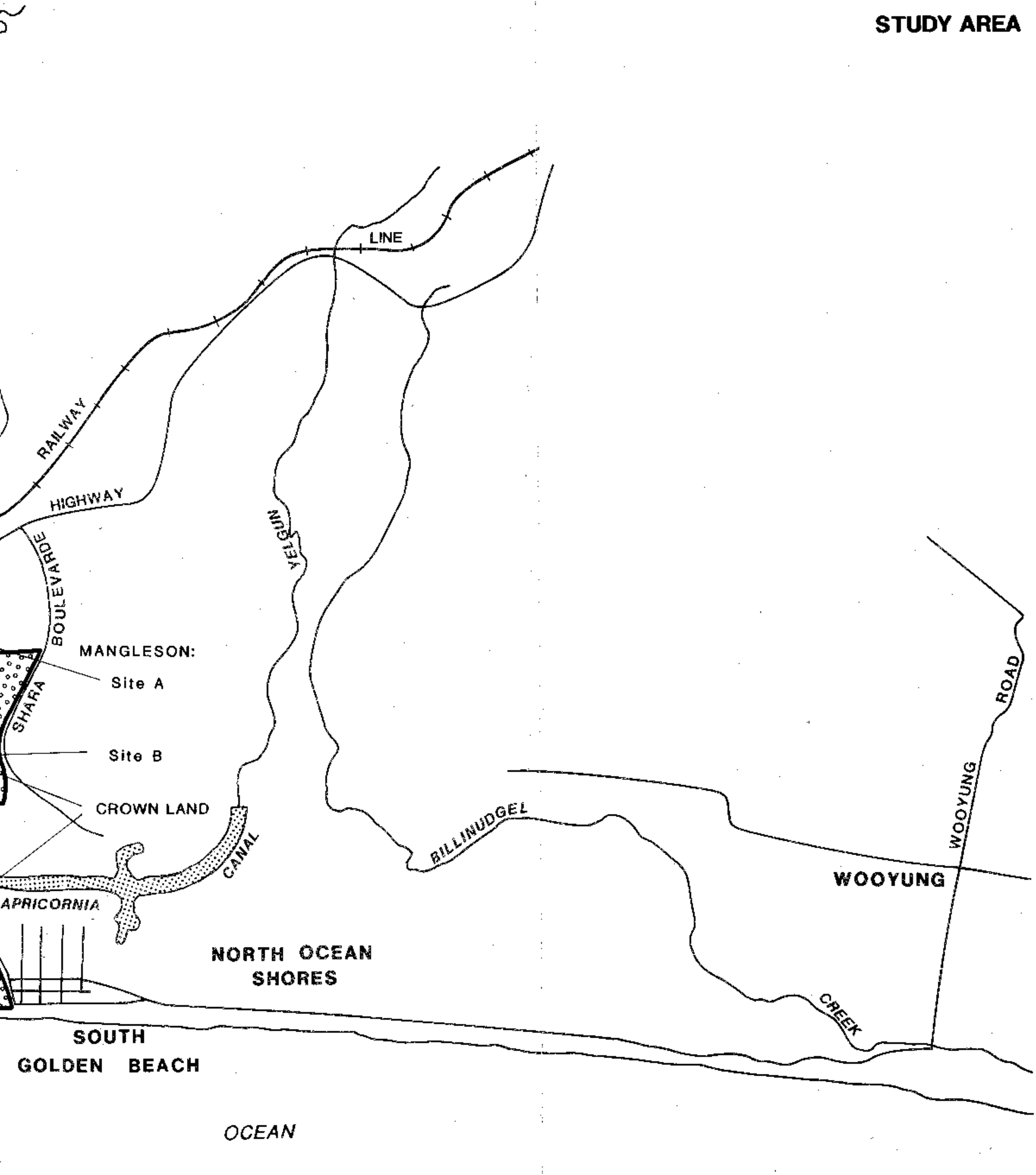
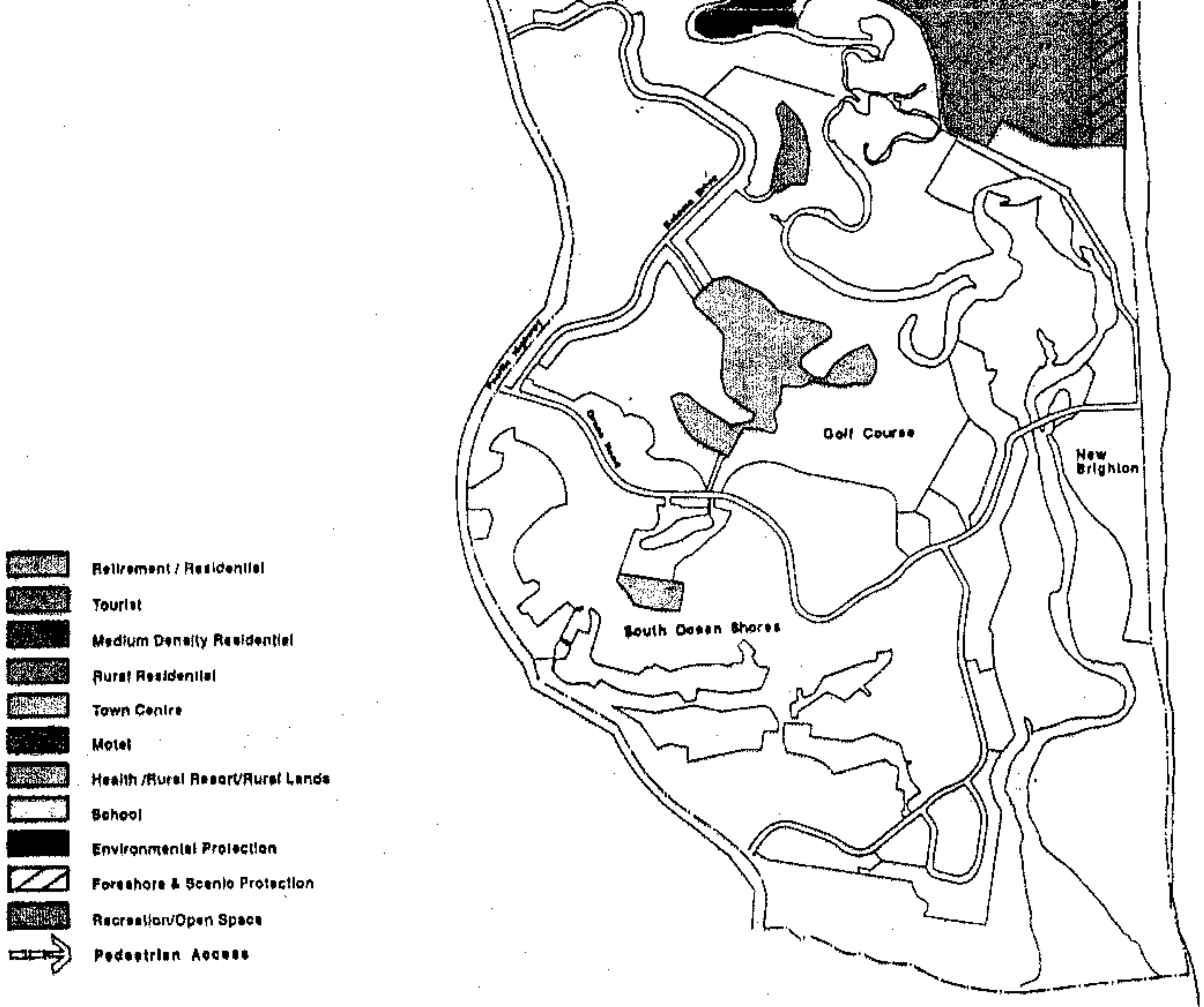


Fig 3



OCEAN SHORES
Proposed Development Concept

Rice Daubney
Urban Planners

December

PROPOSED DEVELOPMENT CONCE

Fig 3

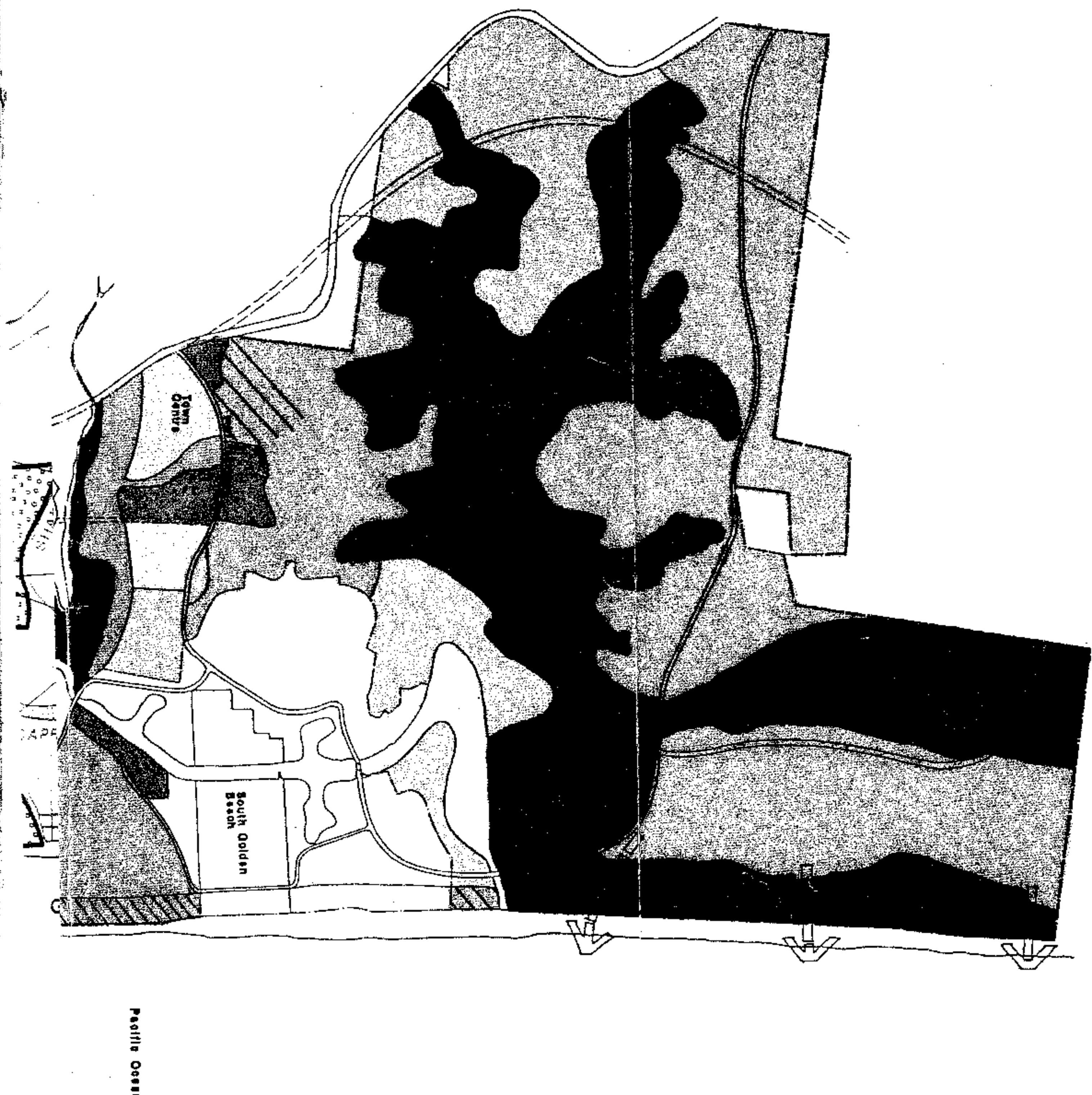


FIGURE 4

WBNM LAYOUT

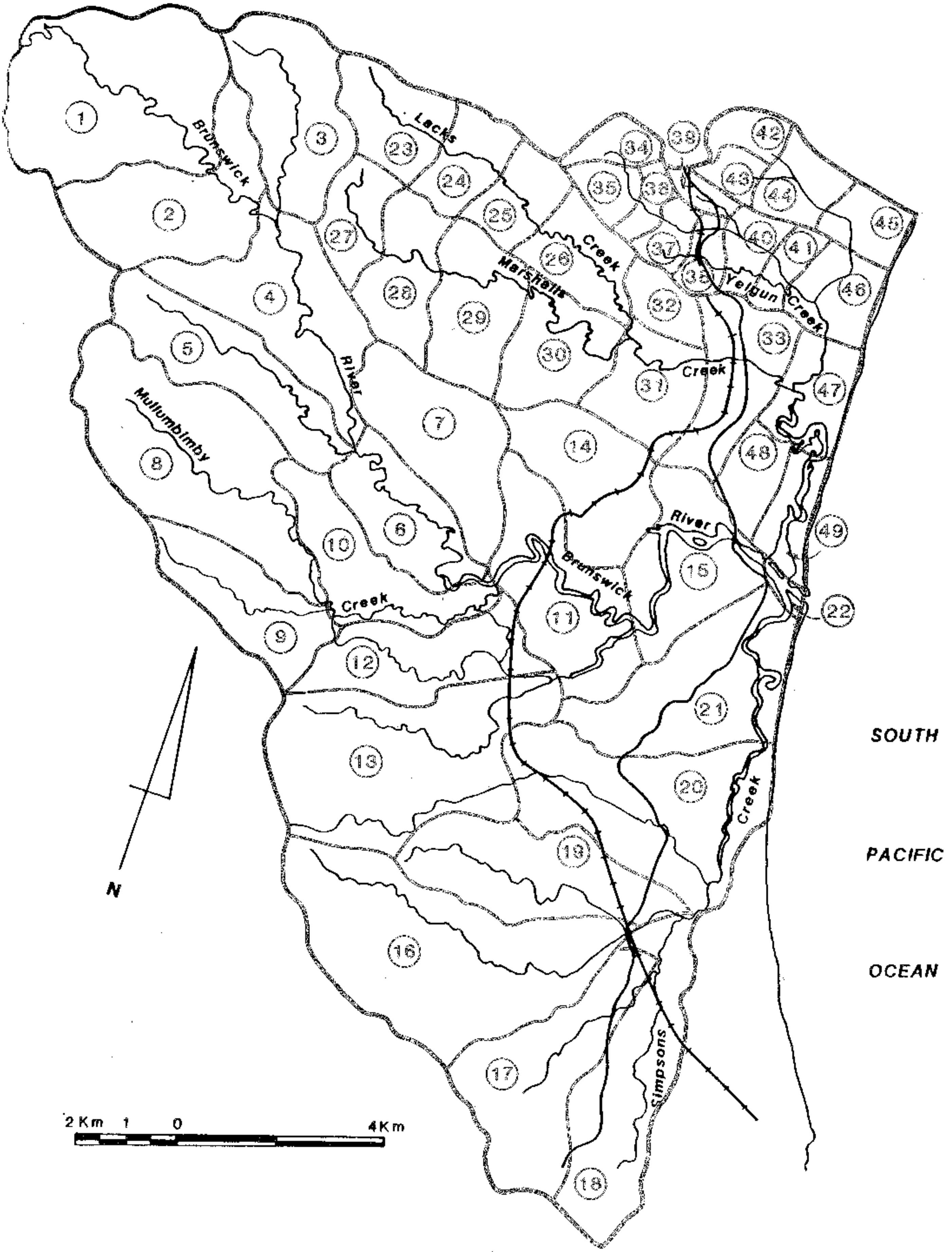


FIGURE 4

WBNM LAYOUT

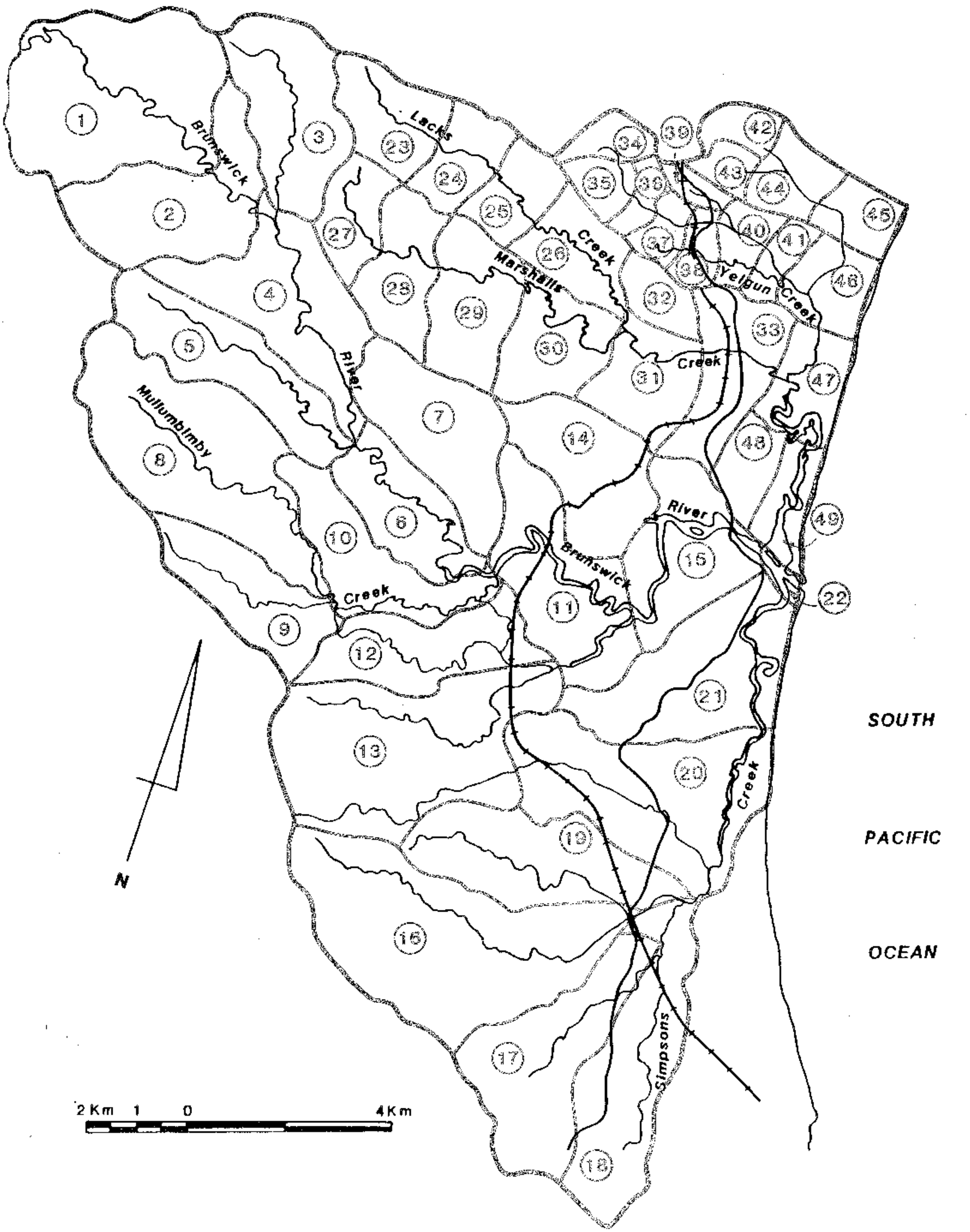


FIGURE 5

FLOW HYDROGRAPHS DURRUMBUL MAY 1987 FLOOD

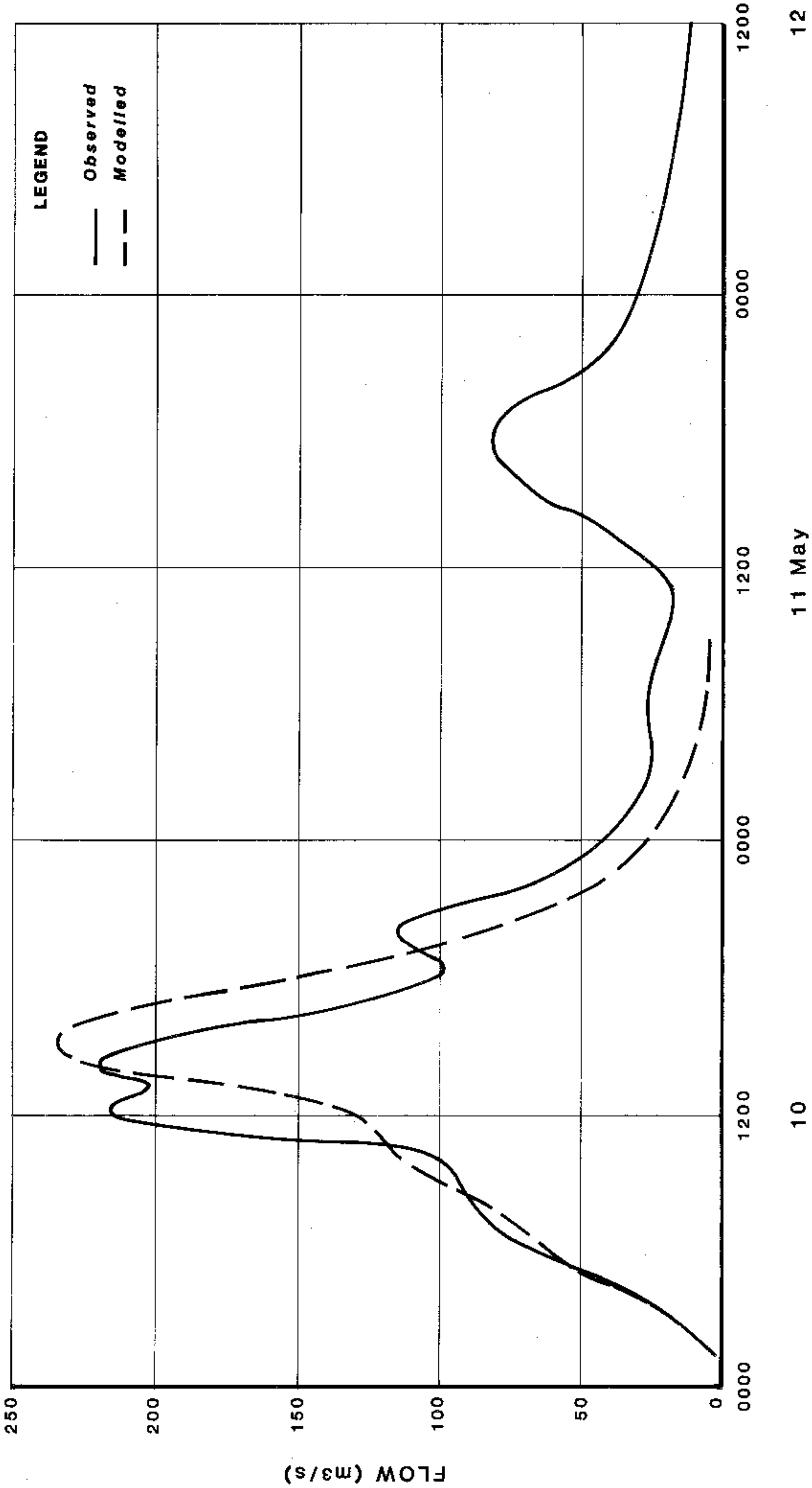
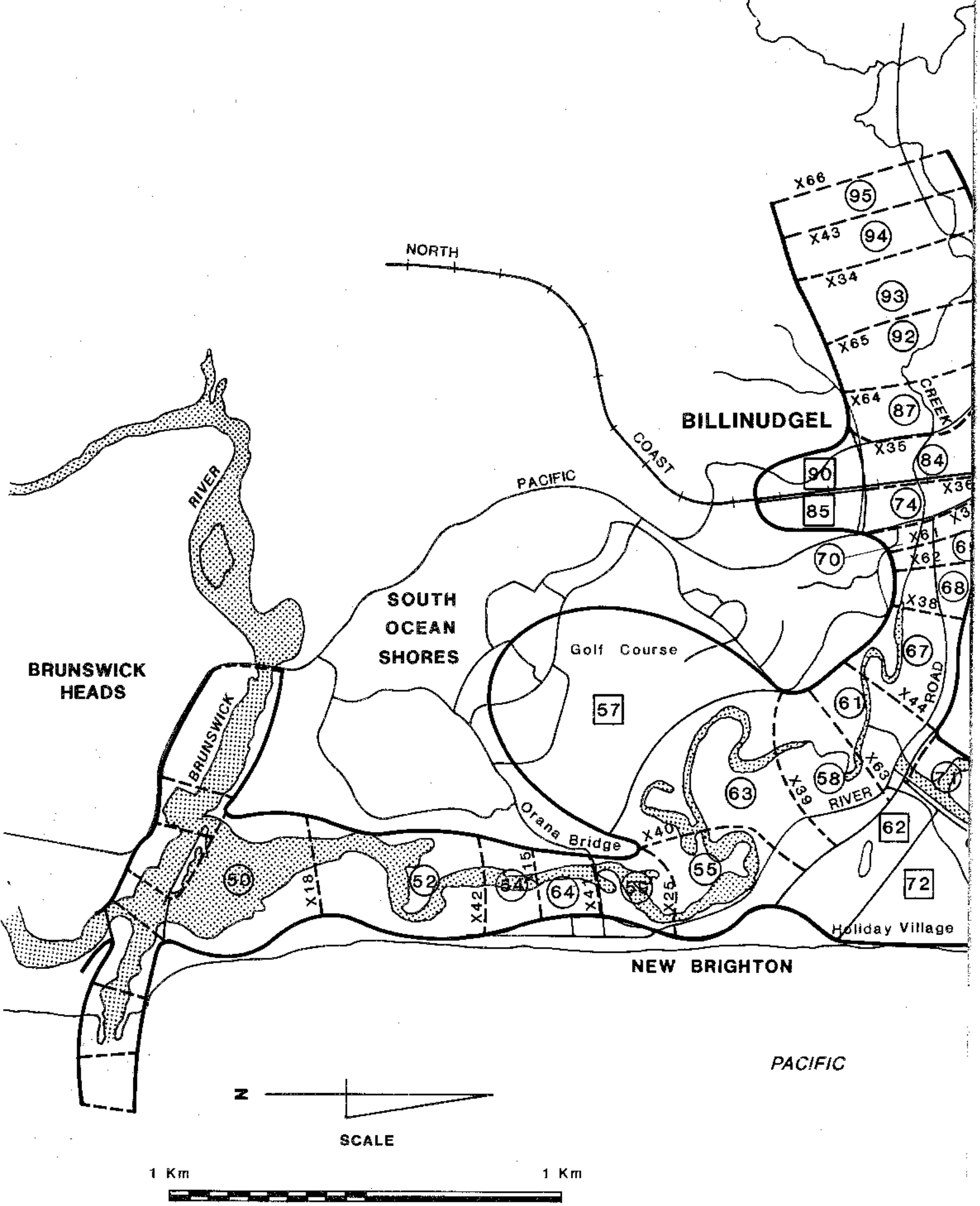


Fig 6



NORTH
PACIFIC
COAST
BILLINUDGEL
CREEK
SOUTH
OCEAN
SHORES
BRUNSWICK
HEADS
RIVER
BRUNSWICK
Golf Course
Orana Bridge
RIVER
ROAD
Holiday Village
NEW BRIGHTON
PACIFIC
X66
X43
X34
X65
X64
X35
X61
X62
X38
X44
X63
X39
X40
X18
X42
X41
X25
15
95
94
93
92
87
84
90
85
74
70
68
67
61
58
63
57
62
72
55
50
52
64
N
SCALE
1 Km
1 Km

FIGURE 6

CELL MODEL LAYOUT MARSHALLS CREEK

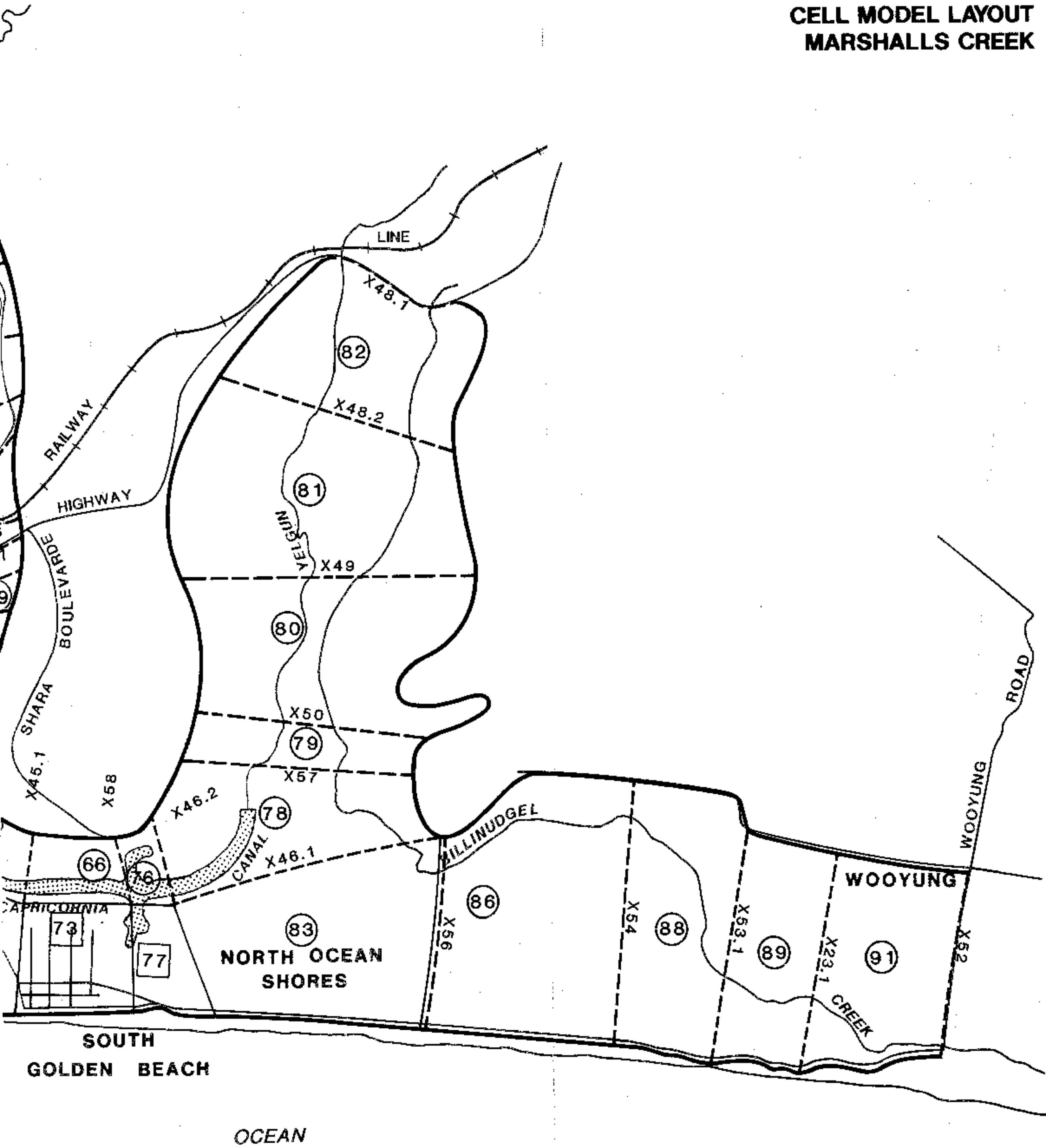


Fig 7

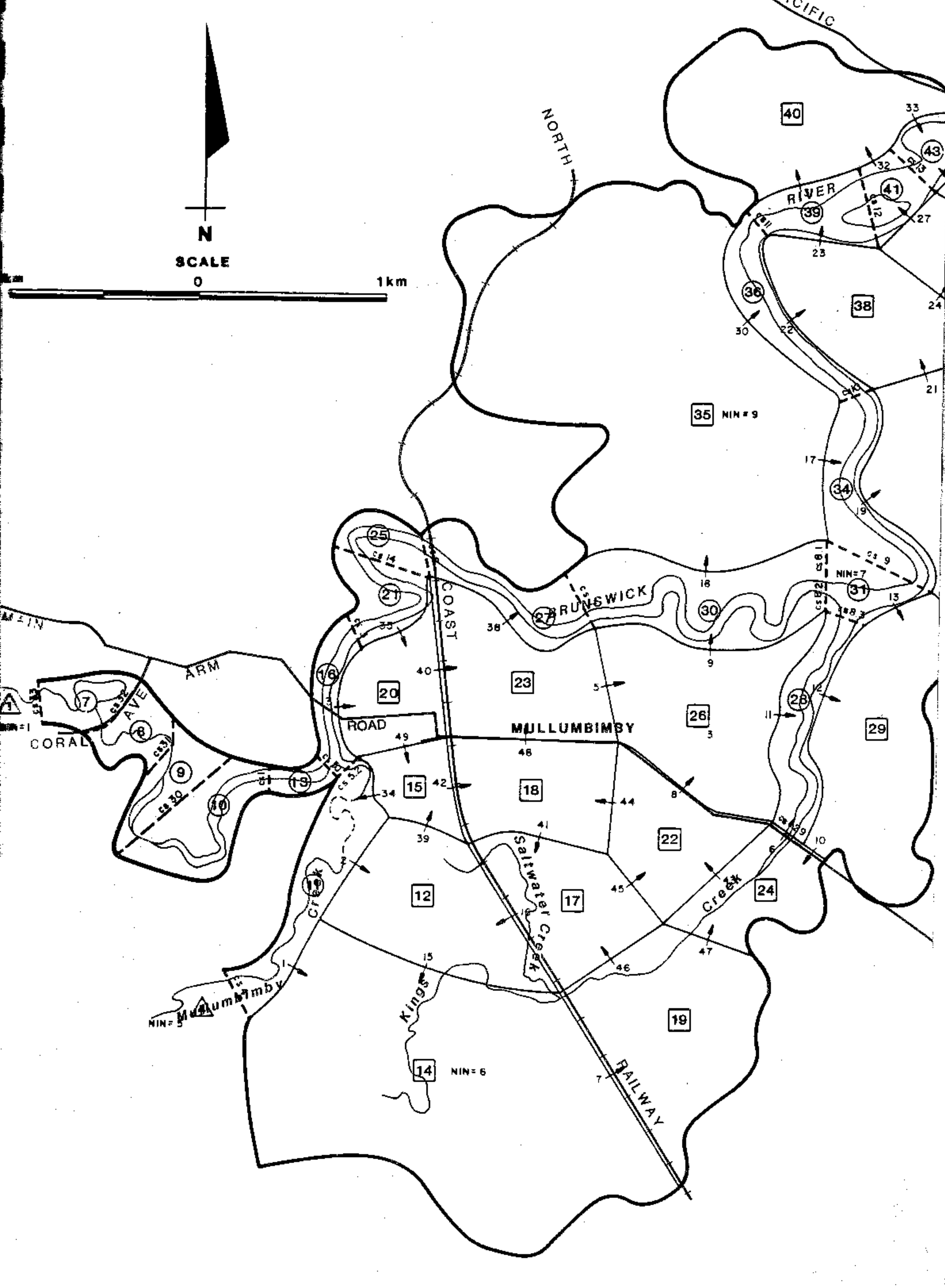
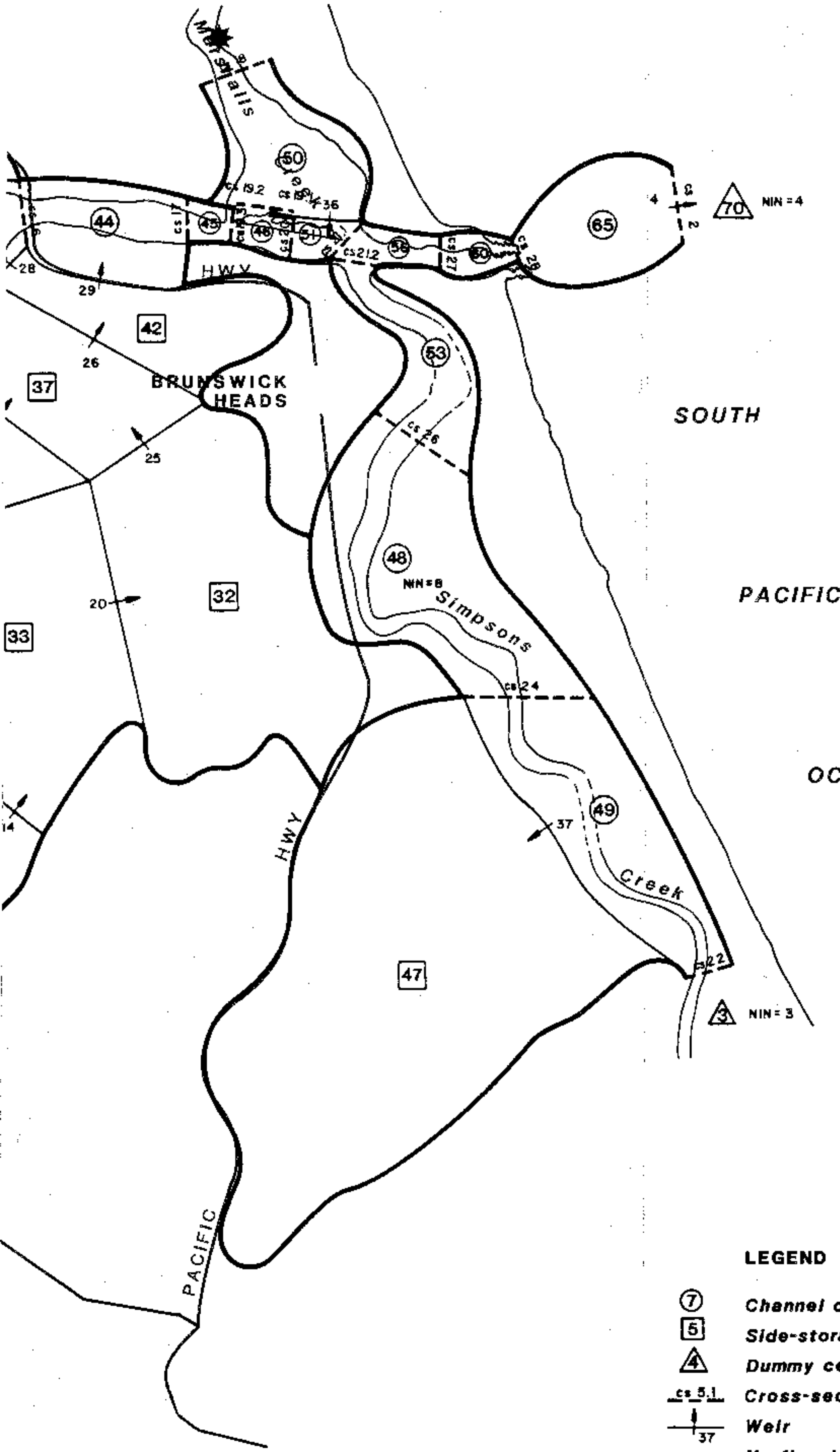

N
SCALE
0
1km
PACIFIC
NORTH
RIVER
BRUNSWICK
COAST
ROAD
MULLUMBIMBY
ARM
AVE
CORAL
Saltwater Creek
Creek
Kings
RAILWAY
NIN=9
NIN=7
NIN=6
40
38
35
23
26
29
20
15
18
22
12
17
24
19
14

FIGURE 7

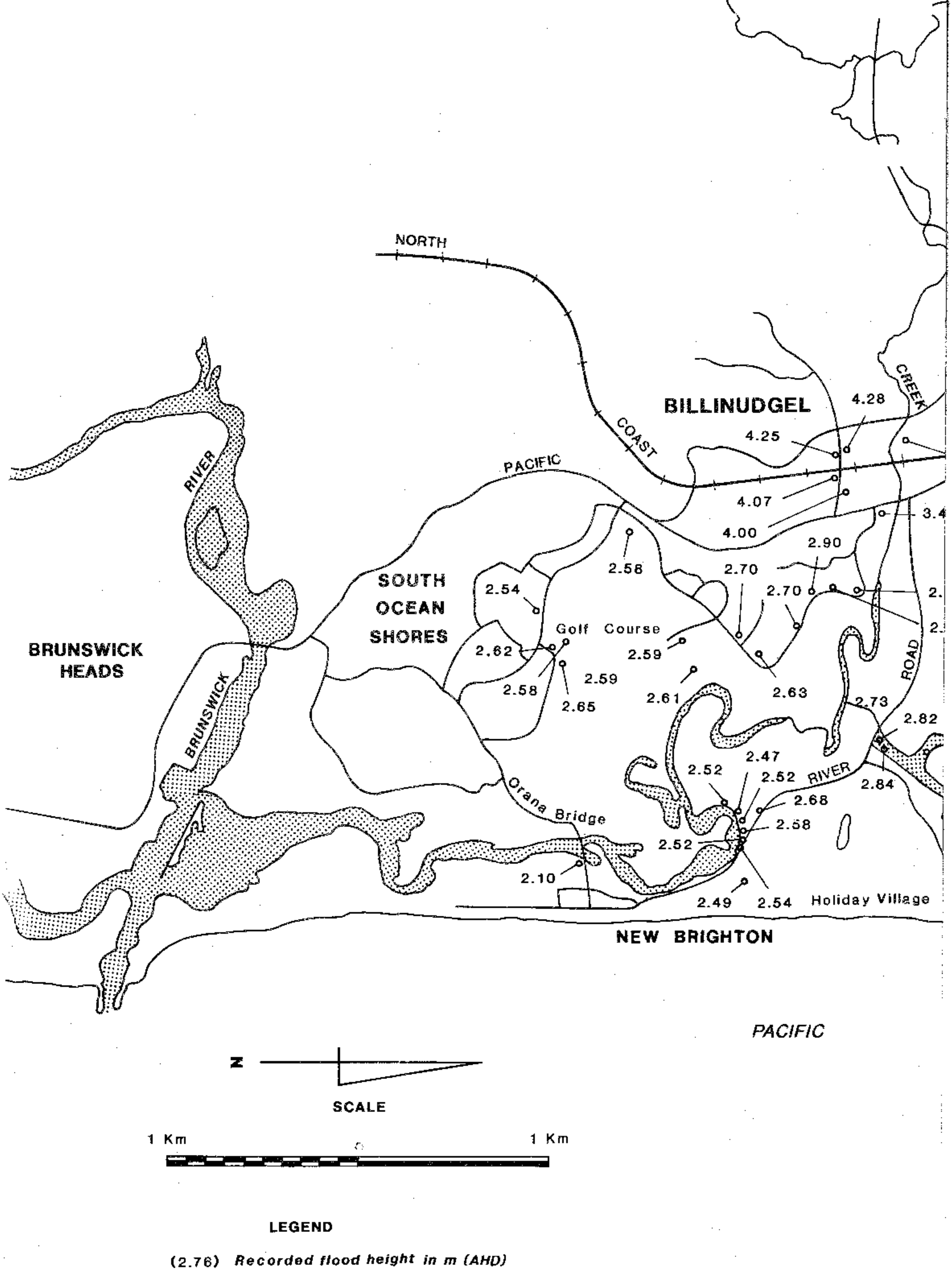
CELL MODEL LAYOUT
BRUNSWICK RIVER



LEGEND

- Channel cell
- Side-storage cell
- Dummy cell
- cs 5.1 Cross-section
- 37 Weir
- No flow boundary
- NIN = Inflow or boundary condition
- For continuation of model see Figure 6

Fig 8


BILLINUDGEL
NORTH
COAST
PACIFIC
CREEK
ROAD
RIVER
BRUNSWICK
RIVER
BRUNSWICK HEADS
SOUTH OCEAN SHORES
Golf Course
Orana Bridge
Holiday Village
NEW BRIGHTON
PACIFIC
4.28
4.25
4.07
4.00
2.90
2.70
2.70
2.58
2.54
2.62
2.58
2.65
2.59
2.59
2.61
2.63
2.73
2.82
2.84
2.47
2.52
2.52
2.68
2.58
2.52
2.10
2.49
2.54
N
SCALE
1 Km
1 Km
LEGEND
(2.76) Recorded flood height in m (AHD)

MAY 1987 FLOOD LEVELS

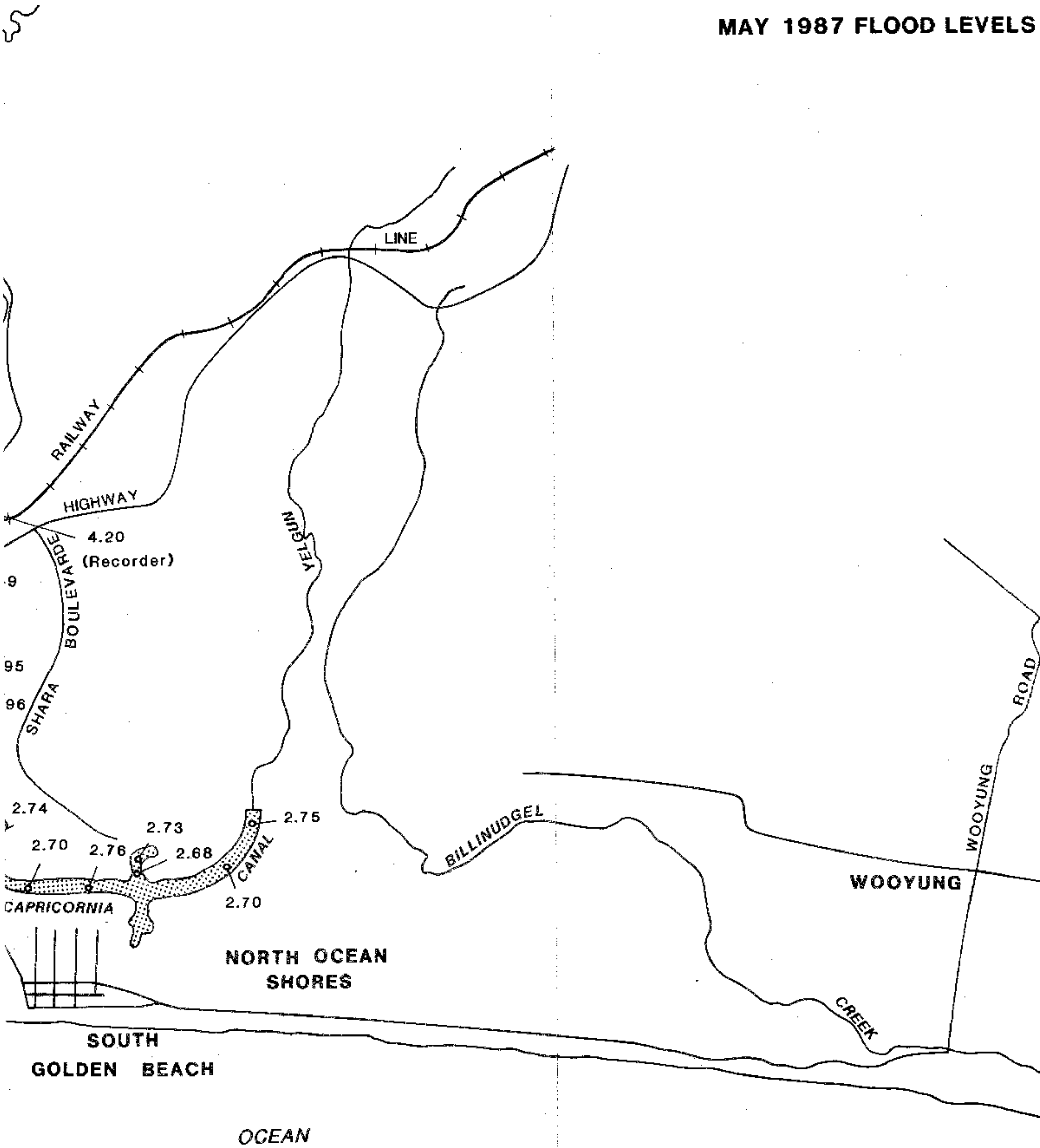


FIGURE 9

LONGITUDINAL PROFILE MARSHALLS CREEK MAY 1987 FLOOD

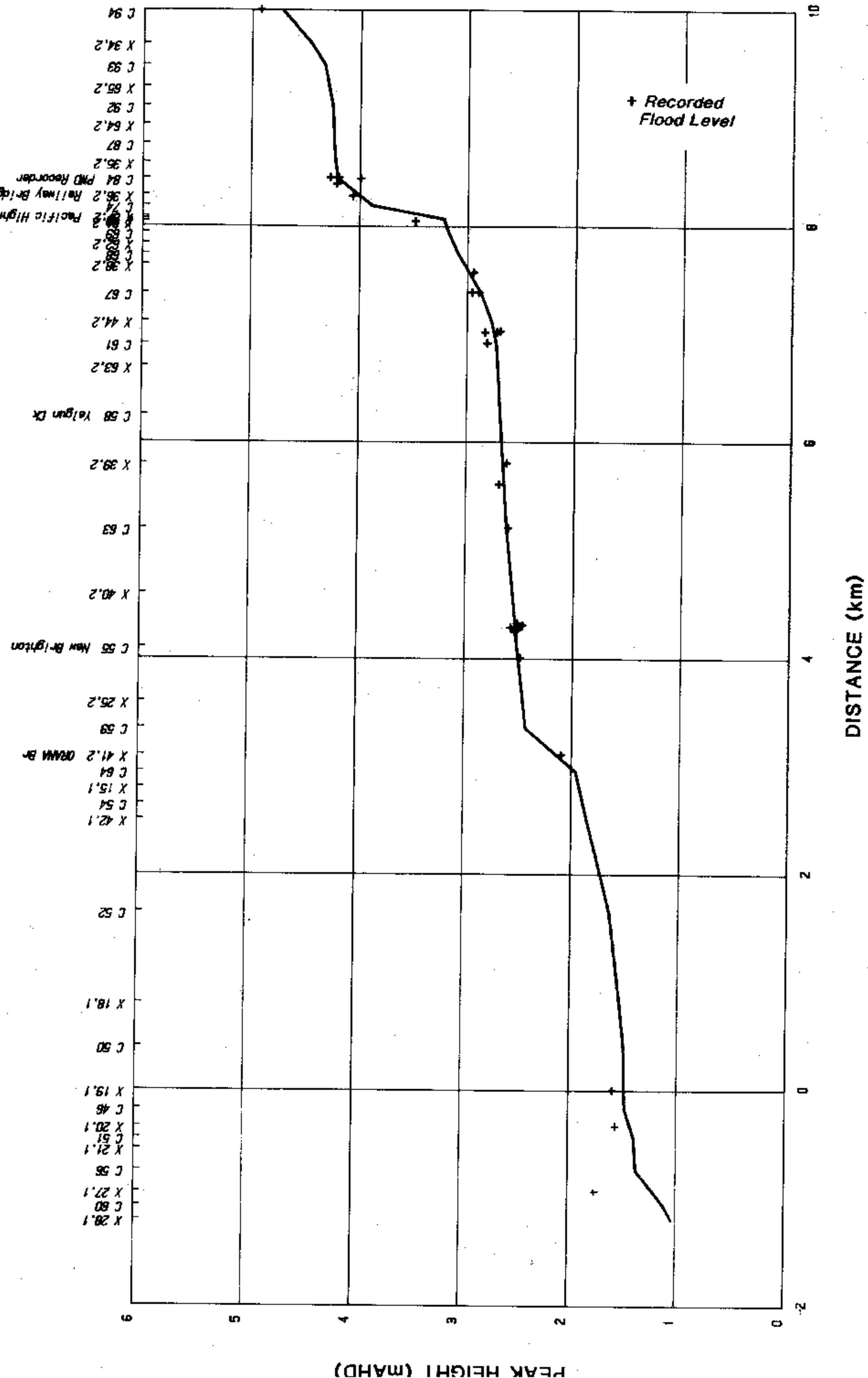


FIGURE 10

LONGITUDINAL PROFILE BRUNSWICK RIVER MAY 1987 FLOOD

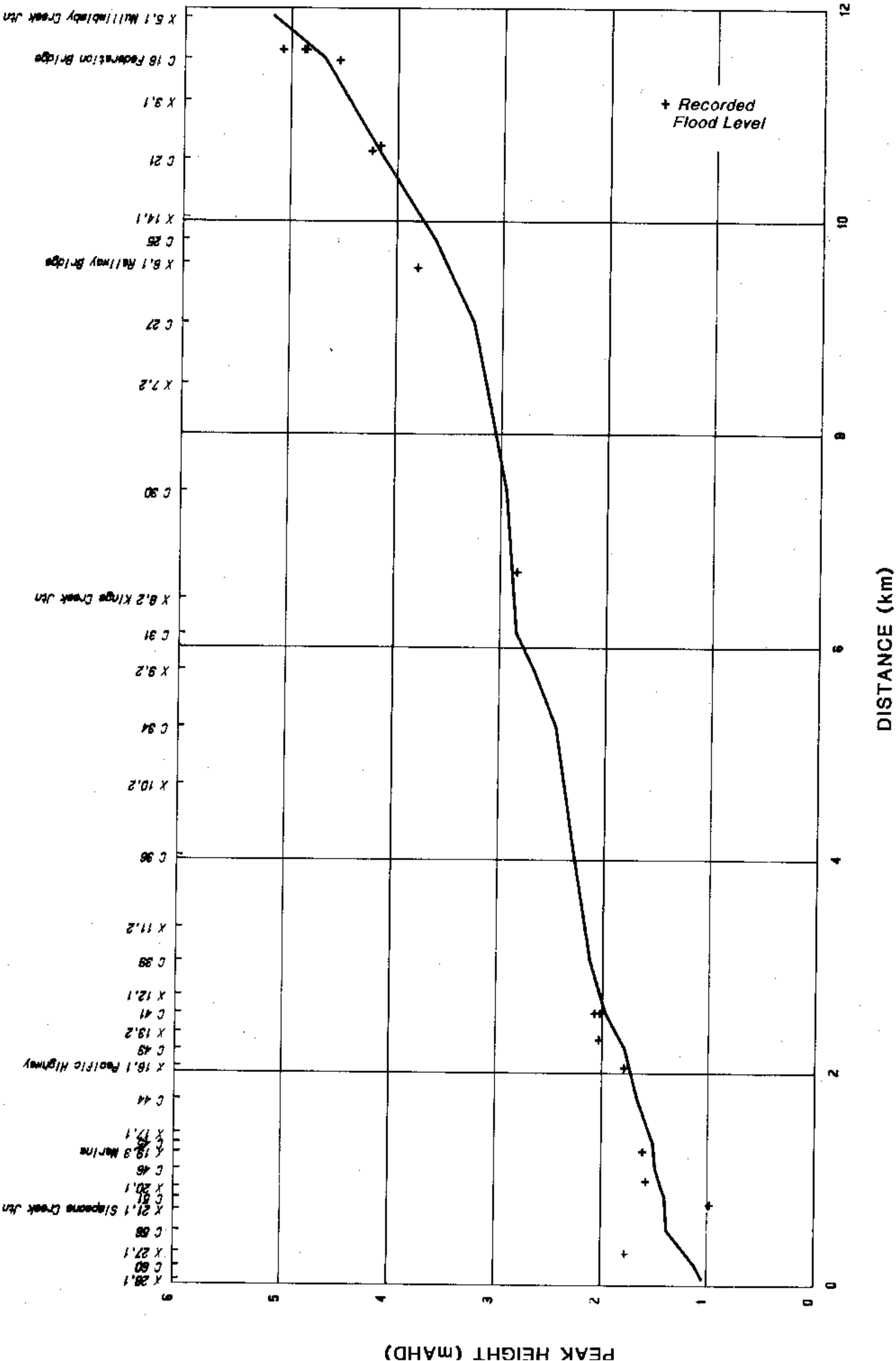


FIGURE 11

LONGITUDINAL PROFILE CAPRICORNIA CANAL MAY 1987 FLOOD

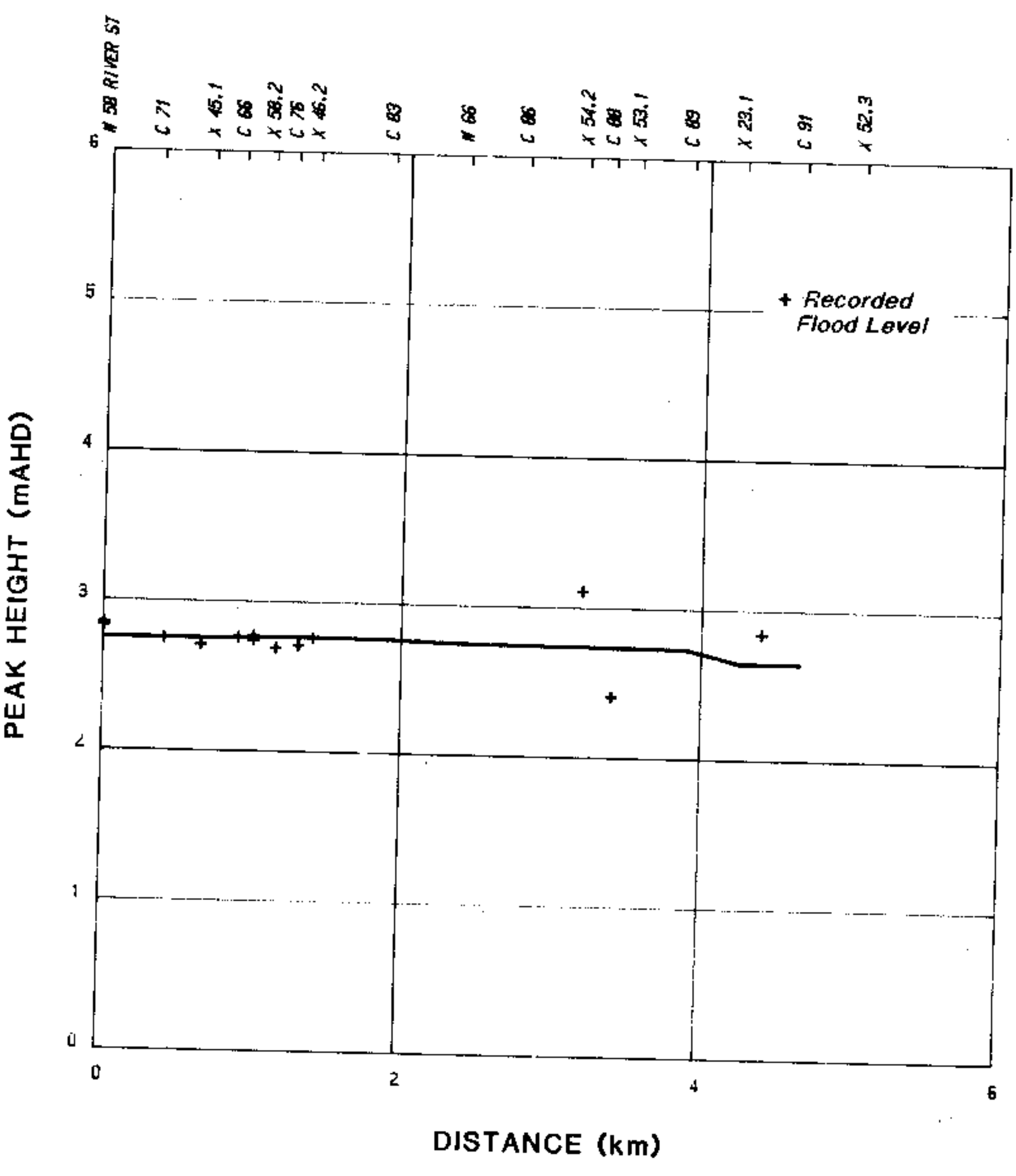


FIGURE 12

STAGE HYDROGRAPHS
MAY 1987 FLOOD

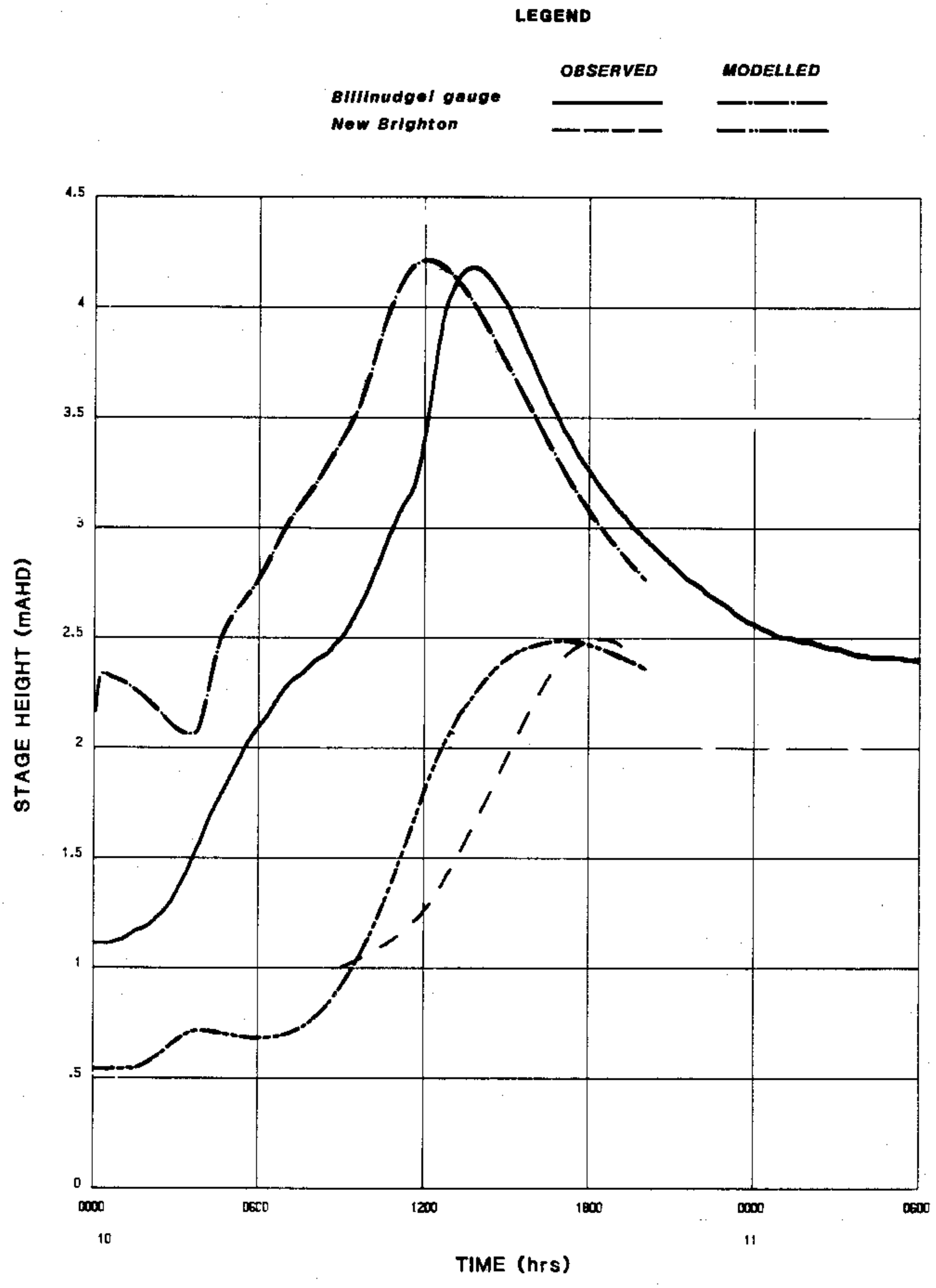


FIGURE 13

LONGITUDINAL PROFILES MARSHALLS CREEK DESIGN FLOODS

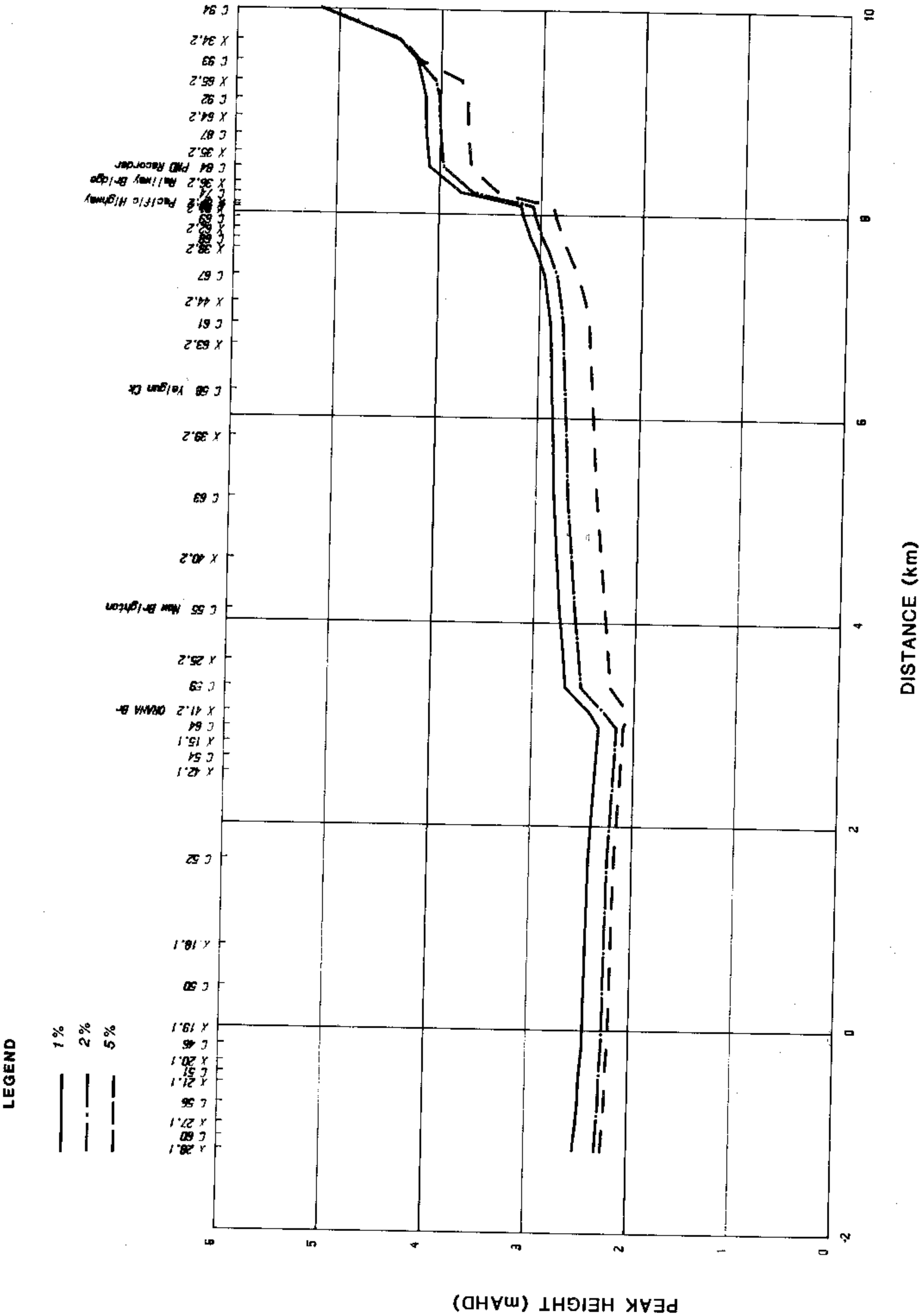
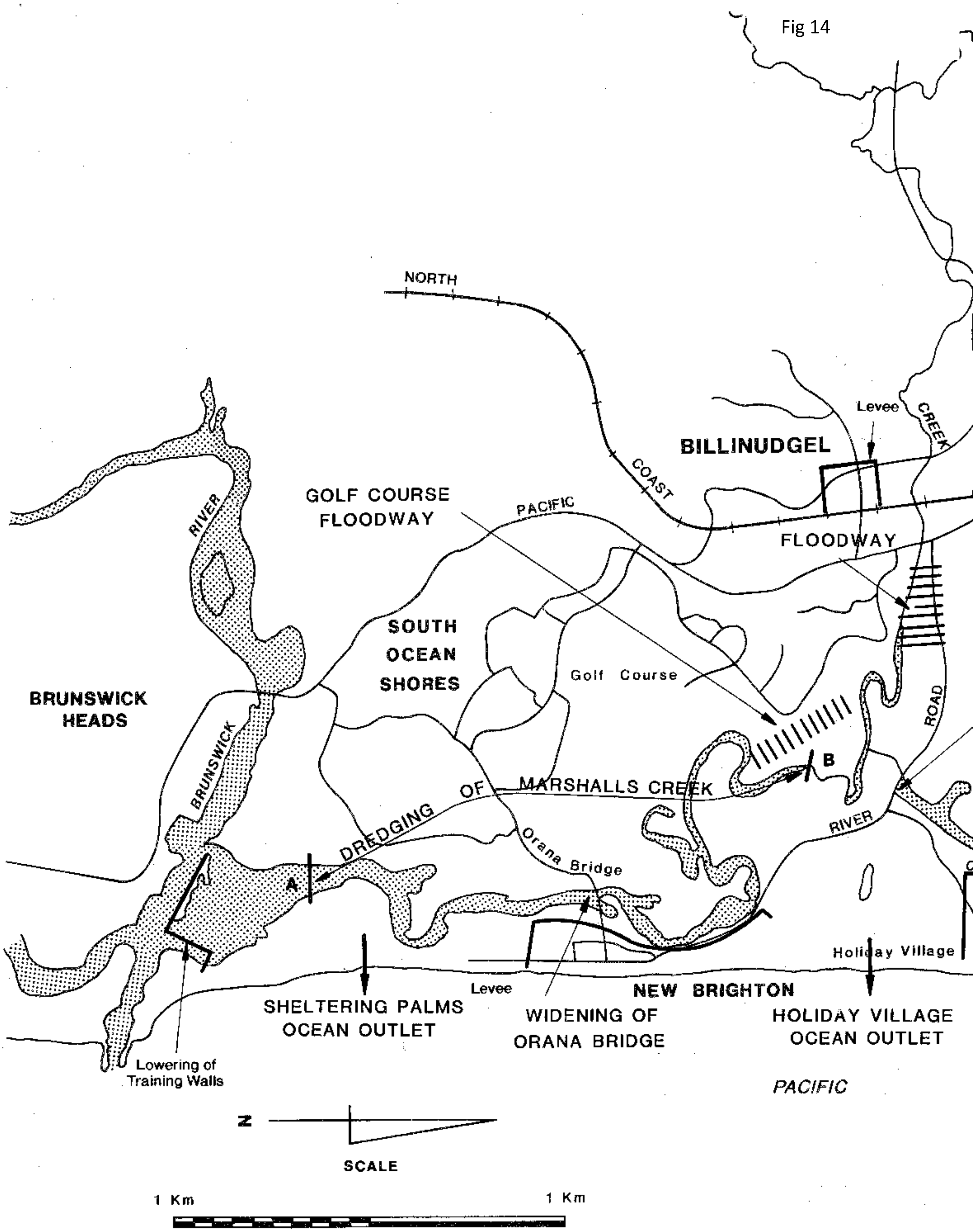


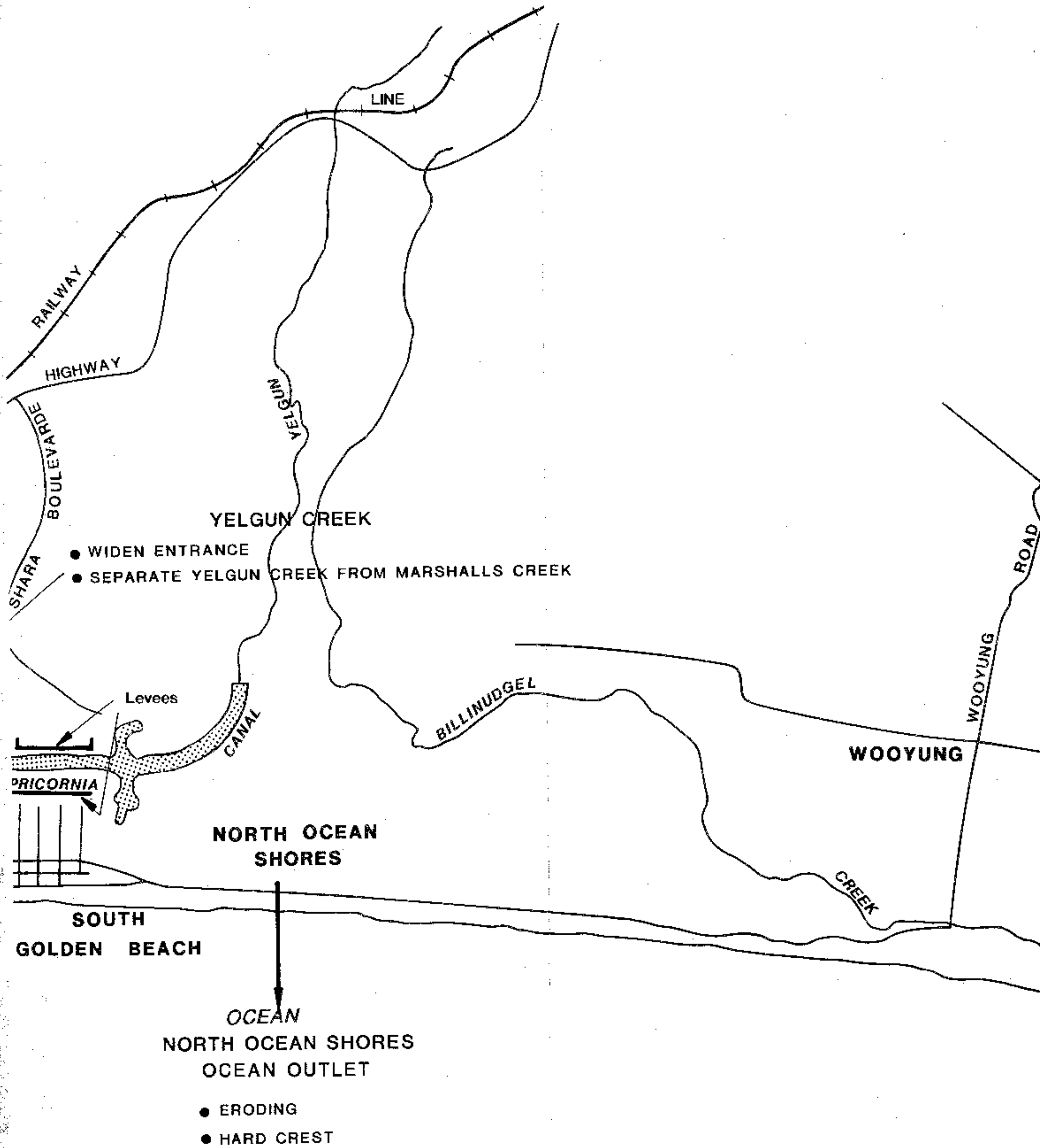
Fig 14



NORTH
COAST
PACIFIC
BILLINUDGEL
Levee
CREEK
GOLF COURSE
FLOODWAY
FLOODWAY
SOUTH
OCEAN
SHORES
Golf Course
ROAD
RIVER
BRUNSWICK
HEADS
BRUNSWICK
B
DREDGING OF MARSHALLS CREEK
RIVER
A
Orana Bridge
Holiday Village
Levee
NEW BRIGHTON
SHELTERING PALMS
OCEAN OUTLET
WIDENING OF
ORANA BRIDGE
HOLIDAY VILLAGE
OCEAN OUTLET
Lowering of
Training Walls
PACIFIC
N
SCALE
1 Km
1 Km

FIGURE 14

RANGE OF FLOOD MITIGATION OPTIONS CONSIDERED



APPENDIX A

APPENDIX A

FLOOD HEIGHTS - MAY 10 1987

Source	Height m (AHD)		Location
Billinudgel Area			
	4.28	Billinudgel Hotel - Recorder	PWD
	4.25	Wilfred st - debris mark	Council
2.30 p.m.	4.20	Billinudgel gauge	PWD
	4.07	Mogo Place - Recorder	PWD
	4.0	Mogo Place - debris	OSDC
	3.49	D/S Pacific Hwy - debris	OSDC
South Ocean Shores			
	2.96	No 93 Balemo Drive - recorder	PWD
	2.95	Balemo Drive - debris	OSDC
	2.90	Balemo Drive - debris	Council
	2.84	Yelgun Creek Bridge - debris	Council
	2.82	Yelgun Creek Bridge - recorder	PWD
	2.73	D/S Yelgun Creek Bridge-debris	OSDC
	2.70	Balemo Drive - debris	PWD
	2.70	Balemo Drive - debris	OSDC
	2.68	River Road - debris	Council
	2.65	Orana Road - debris	Council
	2.63	Balemo Drive - debris	OSDC
	2.62	Orana Road - debris	OSDC
	2.61	Golf Course - recorder	PWD
	2.59	Golf Course - debris	OSDC
	2.58	Balemo Drive - debris	OSDC
	2.58	Orana Road - debris	OSDC
	2.54	Kooringa Crescent - debris	Council

New Brighton

	2.58	Casons Road	Council
	2.54	Casons Road	Council
	2.54	Kooringa Crescent - debris	Council
	2.52	Casons Road	Council
	2.52	Casons Road	Council
	2.52	Casons Road	Council
6.20 p.m.	2.49	New Brighton Store - recorder	PWD
	2.47	Casons Road - debris	Council
	2.10	Orana Bridge - debris	OSDC

North Ocean Shores

2.76	Gloria Street - recorder	PWD
2.75	Capricornia Canal debris	OSDC
2.74	Capricornia Canal debris	OSDC
2.73	Capricornia Canal debris	OSDC
2.70	Capricornia Canal debris	OSDC
2.70	Capricornia Canal debris	OSDC
2.68	Capricornia Canal debris	OSDC

Note: Council - Byron Shire Council
PWD - Public Works Department
OSDC - Ocean Shores Development Corporation Pty Ltd

APPENDIX B

APPENDIX B

COMPUTER RUNS

NOTE: Values shown as the change in peak flood levels in millimetres compared to Existing Conditions.

	1%	2%	5%	1987
EXISTING CONDITIONS				
Existing Conditions				
CELL 74 Billinudgel Indust. Est.	3.78	3.66	3.41	3.89
CELL 69 D/S of Pacific Highway	3.16	3.04	2.84	3.18
CELL 73 South Golden Beach	2.87	2.76	2.47	2.75
CELL 76 Capricornia Canal	2.88	2.76	2.53	2.76
CELL 61 Marshalls Ck-Yelgun Ck	2.88	2.76	2.50	2.73
CELL 57 Sth Ocean Sh Golf Course	2.82	2.68	2.40	2.63
CELL 55 New Brighton	2.74	2.59	2.30	2.52
CELL 54 D/s of Orana Bridge	2.36	2.18	2.20	1.90
OPTION A				
Proposed Development + NO Mitigation				
CELL 74 Billinudgel Indust. Est.	47	27	16	41
CELL 69 D/S of Pacific Highway	225	205	140	237
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	264	230	171	254
CELL 61 Marshalls Ck-Yelgun Ck	234	212	179	239
CELL 57 Sth Ocean Sh Golf Course	224	209	180	235
CELL 55 New Brighton	213	202	176	237
CELL 54 D/s of Orana Bridge	7	21	10	141
OPTION B				
Existing Conditions + Levee at Sth Golden Beach				
CELL 74 Billinudgel Indust. Est.	0	0	0	0
CELL 69 D/S of Pacific Highway	4	0	0	0
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	17	14	27	17
CELL 61 Marshalls Ck-Yelgun Ck	12	15	17	11
CELL 57 Sth Ocean Sh Golf Course	13	16	16	13
CELL 55 New Brighton	12	16	15	12
CELL 54 D/s of Orana Bridge	0	0	0	6

OPTION C

Existing Conditions + Levee at Billinudgel

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	2			
CELL 69 D/S of Pacific Highway	-2			
CELL 73 South Golden Beach	-3			
CELL 76 Capricornia Canal	-3			
CELL 61 Marshalls Ck-Yelgun Ck	-6			
CELL 57 Sth Ocean Sh Golf Course	-5			
CELL 55 New Brighton	-5			
CELL 54 D/s of Orana Bridge	0			

OPTION D

Existing Conditions + Lowering railway line at Mullumbimby

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	0	0	0	0
CELL 69 D/S of Pacific Highway	1	2	0	0
CELL 73 South Golden Beach	0	-2	29	0
CELL 76 Capricornia Canal	1	-2	2	0
CELL 61 Marshalls Ck-Yelgun Ck	0	-2	1	1
CELL 57 Sth Ocean Sh Golf Course	2	-1	0	1
CELL 55 New Brighton	0	-1	0	0
CELL 54 D/s of Orana Bridge	0	0	0	1

OPTION E

Existing Conditions + Doubling of waterway area of bridges at Billinudgel

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-433	-443	-377	-471
CELL 69 D/S of Pacific Highway	3	-12	11	14
CELL 73 South Golden Beach	-4	-2	-26	1
CELL 76 Capricornia Canal	-3	-2	4	2
CELL 61 Marshalls Ck-Yelgun Ck	-6	-3	2	-5
CELL 57 Sth Ocean Sh Golf Course	-3	-1	4	-2
CELL 55 New Brighton	-4	-1	4	-1
CELL 54 D/s of Orana Bridge	0	0	0	8

	1%	2%	5%	1987

OPTION F

Proposed Development +

- Fixed Entrance 2.3m
- Floodway d/s Highway
- Dredging to -2.5m

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-11	-8	-9	30
CELL 69 D/S of Pacific Highway	-84	-89	-103	-132
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-90	-93	-28	-113
CELL 61 Marshalls Ck-Yelgun Ck	-97	-90	-14	-92
CELL 57 Sth Ocean Sh Golf Course	-139	-157	-107	-246
CELL 55 New Brighton	-134	-166	-126	-269
CELL 54 D/s of Orana Bridge	25	31	32	-164

OPTION G

Proposed Development +

- Fixed Entrance 2.1m
- Floodway d/s Highway
- Dredging to -2.5m

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-12	-9	-10	31
CELL 69 D/S of Pacific Highway	-105	-119	-128	-149
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-181	-173	-112	-183
CELL 61 Marshalls Ck-Yelgun Ck	-148	-155	-83	-156
CELL 57 Sth Ocean Sh Golf Course	-173	-209	-175	-310
CELL 55 New Brighton	-163	-213	-184	-335
CELL 54 D/s of Orana Bridge	27	31	32	-195

	1%	2%	5%	1987
OPTION H				
Proposed Development +				
- Eroding Entrance				
- Floodway d/s Highway				
- Dredging to -2.5m				
CELL 74 Billinudgel Indust. Est.	-11	-8	-13	30
CELL 69 D/S of Pacific Highway	-84	-88	-71	-131
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-132	-108	-35	-118
CELL 61 Marshalls Ck-Yelgun Ck	-115	-99	-16	-97
CELL 57 Sth Ocean Sh Golf Course	-143	-160	-111	-261
CELL 55 New Brighton	-136	-168	-132	-285
CELL 54 D/s of Orana Bridge	25	31	32	-170
OPTION I				
Existing Conditions + Dredging to -2.5m				
CELL 74 Billinudgel Indust. Est.	-8	-12	-15	6
CELL 69 D/S of Pacific Highway	-6	-13	-22	-17
CELL 73 South Golden Beach	-89	-111	-273	-161
CELL 76 Capricornia Canal	-85	-107	-75	-144
CELL 61 Marshalls Ck-Yelgun Ck	-112	-148	-148	-193
CELL 57 Sth Ocean Sh Golf Course	-156	-205	-233	-322
CELL 55 New Brighton	-169	-220	-218	-347
CELL 54 D/s of Orana Bridge	20	27	28	-221
OPTION J				
Existing Conditions + Entrance Training Walls Lowered				
CELL 74 Billinudgel Indust. Est.	-1			0
CELL 69 D/S of Pacific Highway	0			0
CELL 73 South Golden Beach	-7			20
CELL 76 Capricornia Canal	-6			15
CELL 61 Marshalls Ck-Yelgun Ck	-7			15
CELL 57 Sth Ocean Sh Golf Course	-10			13
CELL 55 New Brighton	-14			13
CELL 54 D/s of Orana Bridge	34			-15

OPTION K

Proposed Development +

- Eroding Entrance
- Floodway d/s Highway
- Dredging to -2.0m

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-9	-6	-7	29
CELL 69 D/S of Pacific Highway	-78	-82	-95	-123
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-119	-95	-17	-95
CELL 61 Marshalls Ck-Yelgun Ck	-100	-83	4	-72
CELL 57 Sth Ocean Sh Golf Course	-121	-128	-62	-195
CELL 55 New Brighton	-115	-136	-79	-213
CELL 54 D/s of Orana Bridge	25	29	30	-147

OPTION L

Proposed Development +

- Eroding Entrance
- Floodway d/s Highway
- Dredging to -1.5m

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-8	-3	-3	27
CELL 69 D/S of Pacific Highway	-72	-77	-90	-114
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-105	-85	0	-71
CELL 61 Marshalls Ck-Yelgun Ck	-82	-69	25	-47
CELL 57 Sth Ocean Sh Golf Course	-95	-97	-12	-124
CELL 55 New Brighton	-88	-101	-21	-134
CELL 54 D/s of Orana Bridge	24	27	29	-118

OPTION M

Proposed Development +

- Eroding Entrance
- Floodway d/s Highway
- ORANA Bridge Widened

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-4	3	6	17
CELL 69 D/S of Pacific Highway	-64	-66	-77	-82
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-87	-57	31	5
CELL 61 Marshalls Ck-Yelgun Ck	-65	-45	55	28
CELL 57 Sth Ocean Sh Golf Course	-83	-73	30	-28
CELL 55 New Brighton	-109	-113	-16	-80
CELL 54 D/s of Orana Bridge	23	-9	-10	85

OPTION N

Proposed Development +

- Eroding Entrance
- Floodway d/s Highway
- Dredging to -2.0m
- ORANA Bridge Widened

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-11	-7	-9	29
CELL 69 D/S of Pacific Highway	-90	-91	-101	-124
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-144	-112	-33	-105
CELL 61 Marshalls Ck-Yelgun Ck	-128	-105	-18	-82
CELL 57 Sth Ocean Sh Golf Course	-171	-185	-133	-270
CELL 55 New Brighton	-186	-225	-195	-341
CELL 54 D/s of Orana Bridge	28	17	17	-102

OPTION O

Proposed Development +

- Eroding Entrance
- Floodway d/s Highway
- Dredging to -1.5m
- ORANA Bridge Widened

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-9	-5	-6	27
CELL 69 D/S of Pacific Highway	-84	-86	-96	-115
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	-134	-101	-20	-76
CELL 61 Marshalls Ck-Yelgun Ck	-114	-92	1	-60
CELL 57 Sth Ocean Sh Golf Course	-150	-158	-87	-206
CELL 55 New Brighton	-168	-199	-145	-274
CELL 54 D/s of Orana Bridge	25	12	12	-60

OPTION P

Proposed Development +

- Floodway d/s Highway
- Dredging to -2.5m
- ORANA Bridge Widened

	1%	2%	5%	1987
CELL 74 Billinudgel Indust. Est.	-11	-9	-11	30
CELL 69 D/S of Pacific Highway	-46	-62	-100	-120
CELL 73 South Golden Beach		FLOOD FREE		
CELL 76 Capricornia Canal	81	19	6	-6
CELL 61 Marshalls Ck-Yelgun Ck	24	-27	0	-26
CELL 57 Sth Ocean Sh Golf Course	-77	-142	-145	-217
CELL 55 New Brighton	-129	-202	-202	-294
CELL 54 D/s of Orana Bridge	42	22	22	-77

OPTION T

Proposed Development + Greenhouse 0.5m sea level rise
- Eroding Entrance
- Floodway d/s Highway
- Dredging to -2.5m

		1%	2%	5%	1987
CELL	74 Billinudgel Indust. Est.	17			23
CELL	69 D/S of Pacific Highway	91			-130
CELL	73 South Golden Beach		FLOOD FREE		
CELL	76 Capricornia Canal	86			-105
CELL	61 Marshalls Ck-Yelgun Ck	149			-81
CELL	57 Sth Ocean Sh Golf Course	173			-185
CELL	55 New Brighton	209			-178
CELL	54 D/s of Orana Bridge	466			81

APPENDIX C

APPENDIX C

Modelling of Development Proposals

C1. Development Proposal Concept Plan

The following is a full description of the proposed Development, as outlined in Section 4, and how it was simulated using the hydraulic model.

16 ha Lake Within North Ocean Shores

This was assumed to have an invert of -6m with 1.6 batters to 2.0m. It was simulated by adjusting cross-sections 57, 46.1 and 59.1.

Residential Development Surrounding the Lake

The 11 ha site east of Capricornia Canal was accounted for by reducing cross-sections 46.1 and 59.1 to simulate the loss of floodplain storage and flow area. Weir 65 which connects Cell 85 to Cell 77 was also eliminated to account for the effective barrier to flow.

The 31 ha site west of Capricornia Canal was accounted for by reducing cross-sections 57 and 46.1.

Both sites were considered to be entirely filled.

Tourist Development North of the Lake

It was assumed that 50% of the 100 ha site would be filled and that the northern boundary of the site near Wooyung Road would act as a barrier to the northwards flow of water. This was simulated by reducing the lengths of cross-sections 56, 54, 53.1, 23.1 and 52 and eliminating Weir 68.

Residential Development Adjoining Capricornia Canal

The development of this 5 ha site was simulated by reducing the storage area of Cell 62. The site was considered to be entirely filled.

Tourist Development - Holiday Village Site (including Crown Lands)

Development of the 60 ha site was simulated by reducing the storage area of Cell 62 and 72. This was effectively accomplished in conjunction with the previous proposal by eliminating Weirs 56, 57, 59, 61 and 62. The site was considered to be entirely filled.

Recreation/Open Space Development - Golf Course and on Northern Bank of Marshalls Creek

It was assumed that there would be only minor filling and landscaping in both areas which would have no significant hydraulic impacts.

Residential Development - Golf Course

Development of a 16 ha site with the Golf Course was accounted for by reducing the storage area of Cell 57. It was assumed that 75% (12ha) of the site would be filled.

Residential, School and Town Centre Development Adjacent to Shara Boulevarde (including Crown Lands)

Development of these sites were accounted for by reducing cross-sections 44, 38, 62 and 61.

Widen Yelgun Creek Bridge

It was assumed that the present River Road bridge was replaced by a 20m span high level bridge. This was simulated by adjusting Weir 58.

South Golden Beach Levee

A levee around South Golden Beach was simulated by eliminating Weirs 62, 63 and 64. The storage area of the area west of the canal was included within Cell 73 on the east of the canal.

Billinudgel Levee

A levee around Billinudgel was simulated by eliminating Weir 51 and reducing the storage of Cell 90.

Mangleson Site A

Development of this site was accounted for by adjusting cross-sections 61 and 62.

C2. Mitigation Options

Eroding entrance at North Ocean Shores

A weir outlet was connected to Cell 83 which permitted outflow to the ocean. The eroding mechanism was modelled using the Ackers & White sediment transport formulae (Reference C1). At each time step the crest of the weir was lowered by the equivalent volume of sand which was transported to the ocean.

Dredging of Marshalls Creek

The effect of dredging was simulated by altering the cross-section from the mouth of Marshalls Creek to the Yelgun Creek confluence. The sections modified were 18, 42, 15, 41, 25, 40, 39 and 63.

Floodway downstream of Marshalls Creek

The effect of a floodway on the northern bank of Marshalls Creek was simulated by adjusting sections 61, 62 and 38. The overbank was lowered to 1.0m. The Manning's 'n' of the overbank was also adjusted to account for the clearing of the land.

C3. References

C1 Ackers, P and White, W P
Sediment Transport: New Approach and Analysis
Journal of the Hydraulics Division, ASCE, November 1973.

APPENDIX D

MARSHALLS CREEK NEAR BRUNSWICK HEADS, NORTHERN NEW SOUTH WALES: A PRELIMINARY STUDY OF BED SEDIMENTS AND STABILITY

Robin F. Warner

Report for Webb, Mckeown and Associates

October 1988

SUMMARY

1. Three distinct channel types occur in Marshalls Creek estuary: an upper, straight, high-bank fluvially-dominant reach (900m); a tightly-meandering, low-gradient, low-energy, flanked by wetlands, reach, dominated by reworked marine-sand bedload (4000m); and a straighter, mangrove-flanked, sandbed channel, with steeper tidal gradients (more tidally dominant) (3200m). The latter is characterised by more mobile marine sand and reworked marine sand.

2. With the exception of fluvial materials and the lag gravels at site 13, both of which contain lithics derived from the catchment, all the bedload is potentially very mobile. Most of it is very fine to medium sand from marine sand and reworked marine sand.

3. Present stability is evident west of New Brighton from the wide plane bed, mud and bioturbation.

4. Present instability south of New Brighton is seen in micro and meso bedforms, due both to higher tidal energy and to more confined flood runout.

5. Dredging will modify both stable and unstable environments through the increased hydraulic efficiency of a deeper channel, a higher tidal range and a larger tidal prism, as well as more effective flood drainage.

6. In such potentially unstable bed sediments, there will be stability problems which will need to be carefully considered in the planning and design stages of any proposed works.

7. With more time and resources, more detailed work could be undertaken on sediment ages, the depth of scour and fill and problems associated with transient changes.

1. INTRODUCTION

Information on the sources of present channel-bed material for the lower (tidal) part of Marshalls Creek (formerly known as the North Arm of the Brunswick River) was required to assess the feasibility of dredging to alleviate flood impacts at New Brighton. It was also necessary to have some idea of the age and stability of the channel-floor deposits.

This report addresses the problems of sources and stability mainly (there is no indication of age without more work and the costs of dating). It includes sections on the methods used, the sources of material, their relative stability at present, in the immediate past, and in the future (without and with dredging), and finally it presents some discussion of the problems likely to be encountered as conclusions.

2. METHODS

Fieldwork was carried out on 10th July 1988. This consisted mainly of a boat trip from near Grassy Point in the lower estuary to the Pacific Highway bridge and back, and then a shorter traverse to the lower estuary north training wall and back. During these 23 bed samples were collected (Fig.1) and observations were made on channel bedforms, banks, other channels and the estuarine environment generally.

The samples were dried and examined without magnification, prior to being passed on to Darren Skene. He examined all samples under the mircoscope to determine lithology, colour, general grain size, sorting, angularity/ roundness, shell content, iron-oxide staining and estimates of percentages of mud, quartz and lithics. His general comments and table are presented as Appendix A.

Old surveys were examined at the NSW Department of Public Works. It was possible to obtain some details of earlier conditions in the lower estuary. Air photomaps revealed some information on bedforms. Present and past sinuosity was computed 'rom this source. Topographic maps were examined to assess catchment changes and areas.

3. SOURCES OF BED MATERIAL

As indicated in Appendix A, three sources of bed material were revealed in the samples. These were: fluvial sediments, reworked coastal sands and marine sands.

3.1 Fluvial Sediments

The upper part of the estuary, between the Pacific Highway bridge and the entrance to the lake, east of Gooloo Close, revealed fluvial sediments at sample sites 19, 18 and 20. These had a lithic content of about 90% derived from the transport of bedrock fragments from the upstream catchment. Sample 17, with only 5% lithics, was obviously a marginal location for the influence of river-transported bedload. These samples have been described as sandy gravel because of the inclusion of riverrolled coarser sediments. They represent the limits of estuary penetration by purely alluvial material of bedload size. They are located in a nearly straight channel (sinuosity P= 1.1) which is narrow (less than 10m) and whose banks are 1 - 2m high. In a sense it is a confined reach where probably gradient and flow velocities are higher than in the highly sinuous reach downstream of the entrance to the lake.

3.2 Reworked Coastal Sand

Downstream from the lake to about the Orana Bridge, the bed material has been described as reworked coastal sand. This material has been derived from eroded barriers systems and dunes associated with post-transgression Holocene deposition.

When sea level returned to near its present level about 6500 yr BP from low Pleistocene stages, lower estuaries were flooded / the sea. Barrier sands were moved onshore by marine processes to form the extensive sand barriers which dammed smaller rivers from reaching the sea directly. Frequently their lower courses were either directed south (Marshalls Creek) or north (Simpsons Creek) to join more powerful rivers (Brunswick River) maintaining a seaward outlet. This exits to the sea just south of what would have been a rocky island near the present Heads.

So the bed material for much of Marshalls Creek has been derived from a sandy barrier system post-dating 6500 yr BP. Weathering processes in these sand bodies accounts for the paler fawn-grey colour of this well-sorted, well-rounded material. Textures are mainly very fine to medium-grained sands which have been well rounded by marine processes. Their size makes them the least difficult to move by flowing water.

An exception was noted at site 13 where, as well as reworked marine sand, the bed was floored with well-rounded coarse gravel and boulders of local metamorphic rocks. This deposit covers the channel's floor and extends over 20 to 30m of channel length. While the finer reworked sands may pass through this site, the other materials represent some kind of lag deposit, the origins of which are not easily explained. It could represent a basal deposit of some much older flood plain or terrace grading to some lower sea level of the past. It might perhaps be part of a boulder beach but there are no nearby obvious fossil cliffs, which might have provided the source, but the orientation was right in pre-barrier times.

More important perhaps is the fact that this may create some problems for dredging and it may form some sort of long-profile control. This could not be revealed without survey.

3.3 Marine Sand

Although the marine sand is similar to the reworked material in terms of composition and texture, it is different. It is pale orange (iron staining) and contains more shell fragments (both marine and estuarine). Such sands are found downstream of Orana Bridge where they have been deposited in what might be regarded as a flood-tide delta.

Much of the sand in the lower estuary and even up to New Brighton is very mobile in the upper layers, as is evident from low amplitude, ebb-dominated dunes (0.3m high and up to 10m long) and ripples which can be influenced by both flood and ebb currents. Much of the marine sand has probably been introduced since the near closure of the estuary by the north training wall in 1969-1970. Breaks at western and eastern ends allow sea water and sediments in and out at higher velocities than are found elsewhere in the very wide channel. They do not facilitate much scour or removal, except in the immediate vicinity of the openings.

4. STABILITY OF MATERIAL

Sand-sized material in estuaries is seldom stable. Estuaries accommodate the coarse fluvial load being added from the inland catchment and they also allow the accumulation of marine materials coming through the heads or being reworked from the barriers. In between in the low-energy environments, there may be a mud basin floored by fines and reworked materials.

This pattern of mixing or transition from fluvial to marine has been described by Roy and Crawford (1977) in their attempt to explain a sediment deficiency at the coast. Because currently rivers are not supplying coarse loads to beaches, they claimed that there was a sediment budget deficiency and coastal degradation. They also thought that North Coast rivers could be seen to be in Stage 3 of a four-stage model proposed by Davies (1974). The final stage corrects this negative balance of coastal recession as rivers provide a more continuous load to the shore line.

This seems to be the case in this very small estuary. Only suspended fluvial loads reach the ocean. The coarser sands and gravels are held up some 7.2km channel distance from the training wall. So estuaries are slowly being filled in by various kinds of material. This is essentially the geological view of progressive sedimentation into the sink created by the transgression and the subsequent barriers. Apparent stability involves a shorter (engineering) span of time and this may be examined in three ways: the present, the immediate past and the future (without or with modifications).

Present-day stability seems to be indicated by a comparison of 1983 and 1988 cross-section surveys. This work has already been carried out by Webb McKeown. However, the stability noted in just two surveys six years apart may be more apparent than real. For instance, in a 12.5 month period 1970-71, just south of the north training wall in the Brunswick River, vertical variations were recorded along 5 traverses some 2000m long in five surveys. Maximum scour was up to 2.5m and, in some cases, the end survey was not greatly different from the first. Given the smaller estuary of Marshalls Creek and a more protected location with lower flows, the variations may well be much lower but they could still perhaps involve 1m or more. Much depends on duration and magnitudes of above threshold velocities, as well as the general state of the tides, in high runoff events leaving the flatter wetland environments west of New Brighton.

Immmediate-past stability is harder to understand without survey evidence. Three things have operated through the last 100 or so years to affect stability: changing catchment conditions, changing channel conditions and changing natural regimes.

Changing catchments conditions have probably increased runoff and suspended sediment loads, following deforestation, farming and the creation of some urban areas. Any bedload increases would have resulted mainly from channel erosion and reworking of coarse bank materials.

Changing channel conditions have resulted from human action and natural events. The addition of training walls, initially near the Heads in the late 19th Century and then in the north wall to improve navigation in the Brunswick River, has certainly altered the lower estuary (PWD charts).

The latter work has undoubtedly increased accretion with much sand having been added to the lower estuary since the late 1960s. Not all the PWD sources have yet been found and examined.

Channel changes may also have occurred as a result of variations in the natural regime. The late 19th Century is known to have been flood dominant; the first five decades of this century were much drier with few big floods and the period from 1949 onwards has again been flood dominant. Frequent floods of high magnitude in flood-dominated regimes (FDRs) have increased channel dimensions and have caused channel location changes. There is some evidence for this perhaps in the cutoff meanders in the wetlands area between the lake and New Brighton. For instance, at some earlier stage (probably not that long ago), the inclusion of the old meanders would have made the channel length from Site 20 to the Orana Bridge 5.8km. Sinuosity (channel length/valley length) was then very high at 2.3. Now the distance is 4.7km and the sinuosity has been reduced to 1.9. The loss of more than 1km has been effected by greater discharge (the former meanders are smaller in wavelength and channels were narrower) and some coarsening of the load. When this occurred is not evident as yet (may have been post-1949 but perhaps it was in the later half of the 19th Century). Below the Orana Bridge the channel length is 2.5km and sinuosity much lower at 1.2, indicating a coarser sand-perimeter channel and perhaps higher energy.

Future changes in stability may be of two kinds: those associated with no modification to the present channel and those with dredging for flood mitigation (or navigation as in PWD, 1984). In the former there would probably be a slow loss of capacity with the addition of lithics from the catchment and more marine material in the lower estuary. Some kind of channel would be maintained for the evacuation of water and fine sediments. Such slow changes might be accelerated by more catchment development and the maintenance of a flood-dominated regime.

The latter case cannot really be commented on in detail because data on location, size, shape and so on for the dredged channel are not available. The following speculations are apparent:-

(a) much of the lower part of any dredged channel will be cut in mobile sands, which are subject to reworking at fairly low velocities;

(b) west of New Brighton the tidal range will be increased, as will the prism volume (these are to be offset by increases in cross-section area);

(c) boundaries upstream and downstream of dredged reaches will need careful attention (NB PWD 1969 plan for western entrance through training wall);

(d) upstream there will be some potential for rejuvenation of more confined reaches of undredged channel.

These problems are being addressed by Webb McKeown and need no further comment here.

5. CONCLUSIONS

Samples have revealed that the main source of bed material in Marshalls Creek is reworked coastal sand. This has been derived by erosion of Holocene barriers deposited by marine and aeolian processes since the transgression. At present bed deposits seem to be fairly stable in the shallow, very low gradient, low energy, tight meandering environment west of New Brighton, where floods spread out over the wetlands and into adjacent settled areas. Tidal penetration to this area is presently less important than in the lower estuary (PWD, 1984).

The reworked coastal sand and the more iron-stained, shell-rich marine sands south of New Brighton are much more mobile in a straighter channel, with somewhat lower overbank storage and a steeper tidal gradient. Mobility is evident from ripples and low amplitude dunes. There were no obvious differences in material recovered from 1m below a sand bar near the village. It is probable that floods up to 2m deep may scour the bed for up to 1m but it is still necessary to prove this.

Riverine bed material is limited to the uppermost part of the tidal reach in a confined, high-bank, nearly straight channel east of the Pacific Highway bridge. On the limited observations made, this material appears to be advancing only slowly into the main estuary. There is insufficient energy to move material of gravel size except in some of the larger floods.

6. REFERENCES

Davies, J.L. (1974) The coastal sediment compartment. Australian Geographic Studies, 12, 139-151.

New South Wales Public Works Department (1984) Brunswick River tidal discharge - September 1983. PWD Report No.83040 (Rept.No.385), Manly Hydraulics Laboratory.

Roy, P.S. and Crawford, E.A. (1977) Significance of sediment distribution in major rivers, northern NSW. 3rd Australian Conference on Coastal and Ocean Engineering, Melbourne, (Inst. of Eng. Aust.) 177-184.

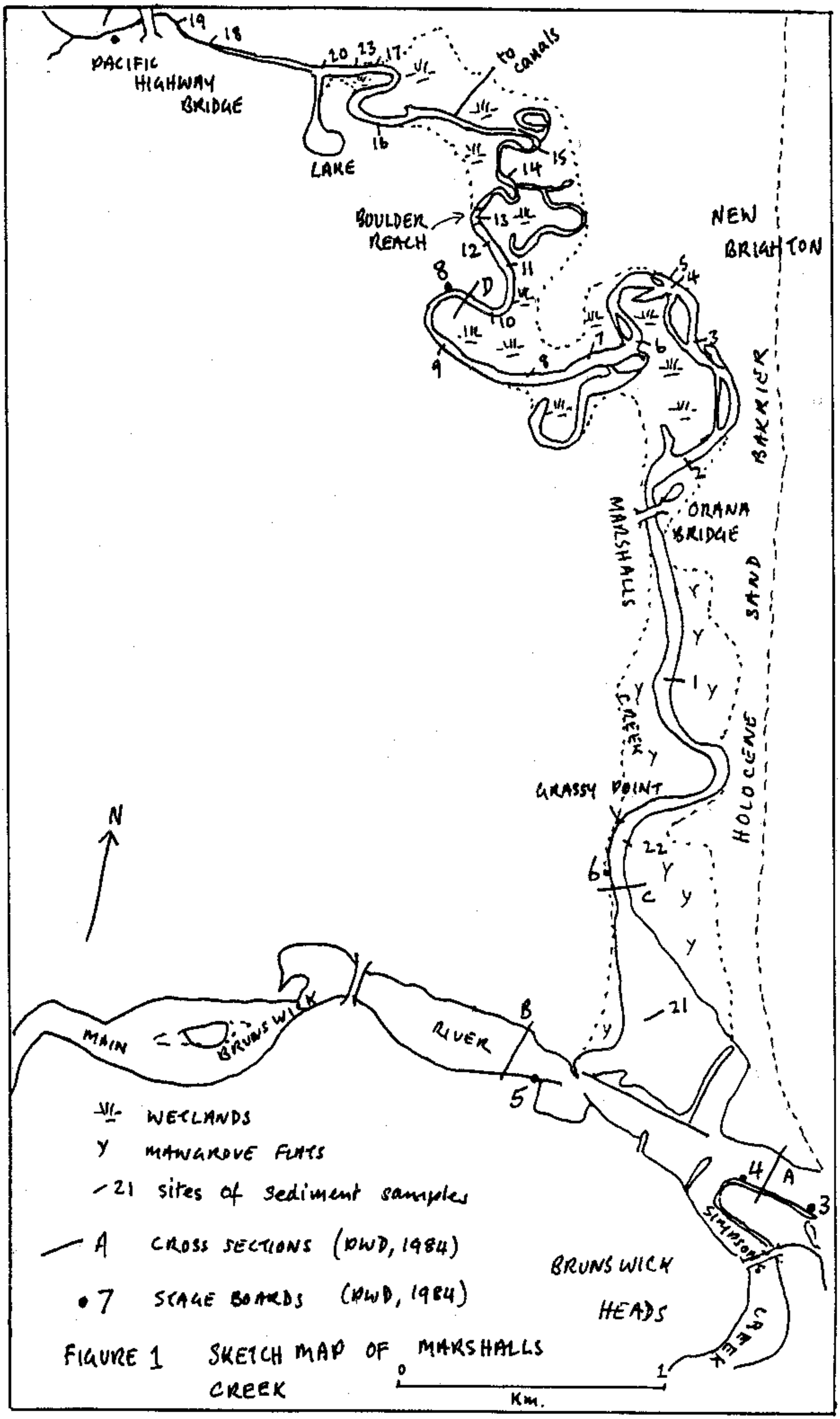


FIGURE 1 SKETCH MAP OF MARSHALLS CREEK

APPENDIX A

ANALYSIS OF RIVER SAMPLES FROM MARSHALLS CREEK, BRUNSWICK HEADS, NORTHERN NEW SOUTH WALES.

Darren Skene

A total of 23 samples collected from the channel of Marshalls Creek were analysed using a binocular microscope. The results are summarised below and detailed analyses are given in Table 1.

Three distinct sediment types are recognised:

(a) fluvial sediment
(b) reworked coastal sand
(c) marine sand

Fluvial Sediment

This is derived from terrestrial erosion of the country rocks in the hinterland and, as a result, they are rich in lithics. The sediment is grey to dark grey, very fine to very coarse, poorly sorted, subangular to angular and contains no shell. The sediment composition reflects the rock types in the river catchment. In Marshalls Creek these are Palaeozoic metasediments.

Reworked Coastal Sand

These sands are reworked from coastal barrier/dune systems intersected by the creek. They are predominantly quartzose in composition with a trace of lithics. The sand is typically pale fawn grey to pale grey, very fine to fine grained, well to very well sorted and subrounded to subangular. Larger fragments of esturaine shell occur occasionally.

Marine Sand

This is texturally and compositionally similar to the reworked coastal sand, but it can be distinguished from the latter mainly by its pale orange-fawn colour and the occurrence of marine (as well as estuarine) shell fragments. The distinctive colouring is due to iron staining of the sand grains. These sands are transported and deposited by ocean waves and tidal currents.

TABLE 1: MARSHALLS CREEK BED SEDIMENT ANALYSES

SAMPLE NO.	LITHOLOGY	COLOUR	GRAINSIZE	SORTING	ANGULARITY AND ROUNDNESS	% MUD	% QUARTZ	% LITHICS FELDSPAR HEAVY MINERALS	% SHELL	IRON OXIDE STAINING	COMMENTS
1	Sand	Pale Orange	f-mg	Well	SR-SA	-	99	Trace	Trace	sl-mod on 60% of grains	Marine sand
2	Sand	Pale-fawn Grey	f-(m)g	Well	SR-SA	-	99	Trace	Trace-frags of estuarine shell	sl on 10%	Reworked
3	Sand	Pale-fawn Grey	f-(m)g	Well	SR-SA	-	99	Trace	Trace-frags of estuarine shell	-	Reworked
4.	Sand	Pale Grey	f-mg	Well	SR-SA	-	99	Trace	Trace	-	Reworked
5	Sand	Pale-fawn Grey	f-(m)g	Well	R-SA	-	99	Trace	-	-	Reworked
6	Sand	Pale-fawn Grey	f-mg	Well	R-SA	-	98	2	-	-	Reworked
7	Sand	Pale-fawn Grey	f-(m)g	Well	R-SA	-	99	Trace	-	-	Reworked
8	Sand	Pale Grey	f-(m)g	Very Well	R-SA	-	99	Trace	-	-	Reworked
9	Sand	Pale-fawn Grey	f-(m)g	Well	R-SA	-	99	Trace	-	-	Reworked
10	Sand	Pale-fawn Grey	f-(vf)g	Very Well	SR-R	-	99	Trace	-	-	Reworked
11	Sand	Light Brown Grey	vf-fg	Very Well	SR-R	Trace	99	Trace	-	-	Reworked
12	Sand	Light Brown Grey	vf-fg	Very Well	SR-R	Trace	99	Trace	-	-	Reworked
13	Gravelly Sand	Pale-fawn Grey	f-(vc)g and gravel	Poor	SR-SA	-	99	Trace	-	-	Reworked Sand Gravel Eroded Metasediments
14	Sand	Pale-fawn Grey	cf-fg	Poor	SR-SA	Trace	99	Trace	-	-	Reworked
15	Sand	Pale Grey	vf-fg	Very Well	SR-SA	-	99	Trace	-	-	Reworked
16	Sand	Light Grey	vf-fg -(m-cg)	Mod-Well	SR-SA	-	98	2	-	-	Reworked
17	Sand	Light Grey	vf-fg-(cg)	Well-Very Well	SR-A	Trace	95	5	Trace-frags. estuarine shell	-	Reworked (Fluvial)
18	Sandy Gravel	Grey-Dark Grey	vf-vc and gravel	Very Poor	SA-A	Trace	10	90	-	-	Fluvial

SAMPLE NO.	LITHOLOGY	COLOUR	GRAINSIZE	SORTING	ANGULARITY AND ROUNDNESS	% MUD	% QUARTZ	% LITHICS FELDSPAR HEAVY MINERALS	% SHELL	IRON OXIDE STAINING	COMMENTS
19	Sandy Gravel	Grey-dark Grey	vf-vc and gravel	Very Poor	SA-A	Trace	10	90	-	-	Fluvial
20	Sandy	Grey-dark Grey	vf-vc and gravel	Very Poor	SA-A	Trace	10	90	-	-	Fluvial
21	Sand	Pale-fawn Orange	vf-fg	Well	SR-SA	-	98	Trace	Trace-frags. estuarine and marine shell	sl-mod. on 60% of grains	Marine
22	Sand	Pale-fawn	vf-fg	Well	SR-SA	-	98	Trace	Trace-frags. estuarine and marine shell	sl-mod 60% of grains	Marine
23	Sand	Pale-fawn Grey	vf-fg (cg)	Well	SR-SA	Trace	99	Trace	-	-	Reworked

KEY:

GRAINSIZE

- vf very fine
- f fine
- m medium
- c coarse
- vc very coarse

ANGULARITY

- R rounded
- SR subrounded
- SA subangular
- A angular

IRON STAINING

- sl slight
- mod moderate

trace = <1%
( ) = less common

APPENDIX E

FEASIBILITY STUDY OF THE

NORTH OCEAN SHORES FLOOD OUTLET

APPENDIX E

FEASIBILITY STUDY OF THE
NORTH OCEAN SHORES FLOOD OUTLET

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LIST OF FIGURES

1. INTRODUCTION

The Brunswick River Floodplain Management Investigation examines a number of flood mitigation measures. One such measure is the construction of flood outlet weirs to provide shorter flow paths to the ocean for floodwaters (see Figure 14 in main text). Arising from the Investigation is an option which includes a flood outlet, the North Ocean Shores flood outlet, which crosses South Golden Beach about 6km north of Brunswick Heads. With this outlet in place, flood flows would be directed from the upper catchments across the coast road and through the dunes to the ocean.

This appendix examines the feasibility of a flood outlet at North Ocean Shores in terms of its coastal processes. A preliminary concept design for the outlet is proposed. The recommended concept is for a fixed crest outlet some 200m wide located at the back of the dune system. A level for the weir crest has been assessed at 1.9m AHD from the Floodplain Management Study. In the area seaward of the weir, it is proposed to remove the dune to a level of 2.3m AHD, and maintain the dune at this level by a regular maintenance clearing operation. However, to provide a barrier to wave overtopping and ocean inflow, the beach berm across the outlet would be maintained at around 4.0m AHD. Event activated berm clearing (to below 2.3m AHD) would therefore be required to ensure operation of the outlet during floods.

As part of the preliminary assessment of coastal processes the report examines possible ocean inflows, sand losses, storm erosion, long term recession and future maintenance requirements for the outlet. The implications of the "greenhouse" effect are also examined.

2. EXISTING COASTAL PROCESSES

2.1 General

South Golden Beach is contained within a zeta-shaped embayment which extends for about 35km from Cape Byron to Hastings Point (Figure 1). The coast north of Brunswick Heads consists of a narrow frontal dune, usually less than 100m wide, backed by low lying coastal plains. The dune crest height generally increases north of Brunswick Heads ranging from RL 4m AHD to a relatively uniform height of RL 6m AHD at the South Golden Beach township. During past storms, the dune system has been overtopped at various locations between Brunswick Heads and New Brighton (Figure 14).

2.2 Beach Recession

In 1978 the embayment was considered by the PWD (Reference 1) to be undergoing recession at an overall average rate of 0.6m/year with an identified rate north and south of the outlet at 0.6m/year and 1.1m/year respectively and localised rate at the outlet location of about 0.9m/year. However, this figure was based on the limited period of photogrammetric information available at that time. The inclusion of more recent data indicates that these rates could overestimate recession by as much as 100% (see Figure E1). On the basis of the additional information, a recession rate of 0.5m/year has been adopted at the outlet site.

2.3 Sand Transport

The gross rates of sand transport at the outlet location were derived by the PWD (Reference 1) for various wave directions using a numerical model. The model incorporated the CERC longshore sediment transport formula (Reference 2). These gross rates in both the north and south directions were then added together to give the net transport rate.

From photogrammetric analysis of the available data the net transport rate was found by the PWD to be 170 000m^3/year to the north. However, by including more recent information, a net transport rate of around 100 000m^3/year to the north is considered more accurate.

The CERC equation incorporates a coefficient to allow calibration to measured rates. By updating the calibration factor, gross sand transport rates for the wave directions were adopted as indicated in the table below.

Generalised Wave Direction	Sand Transport Direction	Gross Transport Rate m^3/year
ENE	South	40 000
ESE	North	45 000
SSE	North	90 000
	Total	175 000

2.4 Storm Erosion

Superimposed on the long term recession of the shoreline are short term erosion and accretion phases in response to storms. This storm erosion can present an immediate hazard to development behind the beach which has not been designed to withstand wave attack.

Storm erosion quantities were not specifically estimated by the PWD (Reference 1) although from the photogrammetric analysis there were short term variations in the mean sea level position of up to 50m in the worst affected areas of the beach. More recent assessment of storm erosion volumes for this embayment by the PWD (Moratti, pers. comm.) has led to the adoption of a design value of 200m^3/m length of beach for the New Brighton area. This volume is measured from mean sea level and is based on an assumed erosion profile.

2.5 Elevated Ocean Level

A report on extreme still water ocean levels in the vicinity of Ocean Shores prepared by Consulting Engineers, Blain, Bremner and Williams (Reference 3) determined a 5% level of 2.1m AHD, a 2% level of 2.4m AHD and a 1% level of 2.6m AHD. More recently, the PWD (Reference 4) estimated a 1% elevated ocean level of 2.6m AHD for the Coffs Harbour region. For flood studies on the Tweed and for this study area the PWD have adopted a design 1% storm ocean level of 2.6m AHD. Based on the above, a 1% value of 2.6m AHD and 5% value of 2.3m AHD have been adopted for North Ocean Shores (See Figure E2).

2.6 Berm Level

The dunes in the vicinity of the proposed outlet were breached during the 1974 storms. From a May 1987 survey of the outlet location (Figure E3) the berm level in the area ranges from 3m to 4m AHD, while the dune height is around 6m AHD.

It is common for the berm level on NSW beaches to be in this range which is just above the wash-zone under normal wave conditions. An examination of unvegetated beach berms at a number of lagoons which have built up after a break out, is given in the following table.

TABLE 1
Beach Berm Levels (Short Term)

Avoca Beach - Lagoon	2.5m to 3.5m AHD
Lake Cathie	2.5m AHD
Shoalhaven River Entrance	2.5m AHD
Dee Why Lagoon	1.9m to 2.6m AHD
Lake Illawarra	1.5m AHD

From the above, and given the relatively high (175,000m^3/yr) gross sand transport rate, sand accumulation on the beach berm at the outlet location could be expected to occur at least to the 2.5m AHD level within a matter of weeks under normal wave activity. Accretion above this height would be a longer term process, dependent on wind as well as wave conditions.

3. OUTLET DESCRIPTION

3.1 General

The Floodplain Management Investigation has examined a range of weir options including:

- a 200m wide fixed crested weir at 2.3m AHD,
- a 200m wide fixed crested weir at 2.1m AHD,
- a 200m wide erodable weir crest at 2.3m AHD with a fixed crest at 1.9m AHD.

The investigation found that these weirs provided significant flood benefits, particularly the latter 2 options. Flood outlets above the 2.3m AHD level and less than 200m wide did not lower flood levels significantly and so are not examined in this report.

3.2 Concept Design

From an assessment of the flooding requirements and coastal processes, a preliminary concept design for the outlet structure has been developed (see Figures E4 & E5). The design criteria used are as follows:

- structure design life	- 50 years,
- plan location	- 10m east of Ocean Shores boundary,
- weir crest width	- 200m,
- weir crest level, fixed	- 1.9m AHD,
- dune level	- 2.3m AHD maximum, maintained by regular clearing,
- berm level (non-flood)	- 4.0m AHD approximately,
- berm level (flood)	- 2.3m AHD maximum, obtained by event activated clearing.

3.3 Coast Road

The present coast road runs parallel to and immediately behind the frontal dune at a level of around 3m AHD.

To minimise construction costs the Coast Road could be incorporated into the weir structure. The surface of the road when sealed could act as the fixed weir crest level at 1.9m AHD. Scour protection would be provided by gabions and reno mattresses along both faces of the weir. Rubble mound end walls at 6.0m AHD could link the weir into the existing dunes and prevent outflanking.

4. CREST LEVEL MAINTENANCE

4.1 Trial Period

Prior to construction of the outlet, and preferably as soon as possible, a trial area of beach berm in the vicinity of the proposed outlet should be lowered to below 2.3m AHD. This area could then be monitored to provide approximate infill rates related to seasonal, wind and wave conditions.

4.2 Accretion Rates

The rate of sand accretion at the outlet will be affected by tide, wave and weather conditions, as well as the height of the berm. However, given that the gross littoral sand transport in the area is high, around 175,000m^3/year (Section 2.3) and the natural berm height in the area is greater than 2.5m AHD, build up of the berm above 2.3m AHD could be expected within days or weeks.

4.3 Maintenance Program

To provide a maximum weir crest level of 2.3m AHD during floods, a combination of regular programmed clearing and event activated clearing will be required. To reduce wave overtopping of the dune and ocean inundation of the back beach area, it is proposed to maintain the foreshore berm on the alignment and level of the surrounding berm, approx. 4m AHD (see Figure E4). Between the foreshore berm and the outlet weir the dunes would be reduced to or below the 2.3m AHD level and maintained at this level by regular programmed clearing operations.

The higher foreshore berm would need to be removed prior to a flood event to allow the outlet to operate as a flood mitigation structure. To reduce reliance on the event activated clearing, it may be possible to maintain a staggered foreshore berm which would inhibit wave overtopping yet provide a lower level (2.3m AHD) flow path for flood outflow (Figure E6). The effectiveness of variations such as this could be tested in the trial period mentioned above.

Clearly, for event activated monitoring to be successful, a number of unknown and variable factors need to be considered. Many of those relating to beach processes could be answered by further detailed investigation and design associated with the trial clearing. However, the first and most important factor would be an ongoing commitment from the responsible authorities to make a clearing program work.

One or two plant operators and a suitable machine would need to be available on site at short notice. Also, an advance flood warning system would need to be developed. This could be activated by relatively minor rainfalls in the catchment because of the low level of disruption caused to the beach by the clearing operation. If a flood did not eventuate, as would be the usual situation, the beach berm could be re-formed to approximately 4.0m AHD or left to rebuild naturally. The actual mechanics of the clearing operation and the follow up work should be determined after further investigation.

Flood warnings could be given with the assistance of the Bureau of Meteorology or by an independent network. To ensure the maximum time possible was available for berm clearing operations, a suitable sand moving plant would need to be garaged in the area with operators on a rostered standby. The catchment has a response time of approximately eight hours. Therefore an event activated clearing operation should be feasible. However, detailed examination of the maintenance procedures, both programmed and event activated, would be required before final design and implementation.

Also required for final design is an evaluation of flood impacts, and a detailed assessment of the coastal processes. The flooding problem is largely addressed in the main report and is principally a matter of adequate design. The following sections provide a preliminary assessment of the coastal processes.

5. ELEVATED OCEAN INFLOWS

5.1 General

The design still water elevated ocean level for a 1% event is 2.6m AHD and for a 5% event is 2.3m AHD (Section 2.5). Therefore ocean storm events larger than the 5% event would have still water levels above the 2.3m AHD maximum weir crest level for floods. Further, for lesser events wave runup would occur above the 2.3m level.

The extent to which ocean inflow occurs as a result of elevated ocean levels or wave runup is complicated by a number of factors. These include the level of the beach berm and dune, the width of the beach from the berm to weir, the profile of the beach in the foreshore and nearshore zones, the wave height and wave period, etc.

5.2 Impacts

Based on a worst case scenario, with a narrow beach berm at 2.3m AHD and a 1% ocean level, ocean water inflow of the order of 500,000m^3 could be expected. This figure is based on still water flow over an eroding 2.3m AHD broad crested weir and would be affected by wave runup and the factors mentioned above. However, the figure is of a sufficient order of accuracy to conclude that even under this scenario, inundation of back beach areas would occur but without flooding properties or services.

With a foreshore berm level maintained at around the 4m AHD level (i.e. a level similar to the surrounding berm) ocean water inflow would be largely restricted to wave overtopping. Under these conditions there would be little or no adverse impact.

6. SAND LOSSES

6.1 Overtopping

The inflow of ocean water during periods of wave overtopping could bring with it large quantities of sand. Rendel (Reference 5) estimated that about 50 000m^3 of sand washed over the low dune just north of Brunswick Heads (Sheltering Palms) during three cyclones in 1975. The PWD (Reference 1) concluded that sand losses in the past due to wave washover were insignificant in terms of the gross transport rate. As with ocean inflow, determination of the quantity of sand likely to be carried over the outlet by waves is dependent on a large number of variable factors. However, any sand lost to the beach system by this process could be returned to the beach during flood events or by periodic dredging if necessary.

6.2 Aeolian Transport

A flood outlet cut and maintained through the dunes would remove the stabilising vegetation. Wind blown sand as well as accumulating on the foreshore berm would be deposited in the lake behind the weir. This sand would represent a loss from the beach system. However, this sand could be returned to the littoral system by periodic excavation and placement back onto the beach if necessary.

6.3 Flood Scour

Sand scoured from channel during floods is deposited offshore. If this sand remains within the beach system, there is no permanent sand loss. However, particularly with large catchments, the channel scoured by flood flows can extend into the nearshore region.

The Floodplain Management Investigation examined the extent of channel scour during a range of flood events. The Ackers and White sediment transport formula (Reference 6) was used to estimate the quantity of sand removed from the entrance. The results of this investigation for a 1% flood indicated that around 10,000m^3 of sand would be scoured from the outlet channel by the flood flows. This level would not be sufficient to form a major outlet channel. Therefore, loss of sand from the beach system by floods moving sand into deep water and out of the littoral system should be negligible.

7. STORM EROSION AND LONG TERM BEACH RECESSION

7.1 General

Using the North Ocean Shores Development boundary as a reference line, the existing dune crest is about 25m to the east and the proposed weir centreline is 10m to the east. That is, the weir has been positioned at the back of the dune system cognizant with the storm erosion and recession hazards. As outlined in Section 2.4, the shoreline at the outlet is subject to a design storm erosion bite of 200m^3 per metre length of beach and long term recession is 0.5m/year.

7.2 Storm Bite

The size of storm bite was assessed for both the natural dune cross-section and for a section cleared to 2.3m AHD. Both sections were used to provide an upper and lower bound to the erosion profile. During a storm event the lower area of the cleared outlet would be partially infilled as storm waves transported sand along the beach. Therefore the quantity of sand available for erosion and hence the storm bite, would be an average between the cleared outlet and the surrounding beach profile. The extent of storm erosion for an averaged value is shown in Figure E7. This shows that the storm bite remains at least 70m from the Ocean Shores Development boundary and well away from the weir structure.

7.3 Beach Recession

The beach is also subject to long term recession. Based on a recession rate of 0.5m/year and a 50 year design life for the outlet weir, an additional 25m setback is required to place the structure beyond wave attack. Figure E7 shows that even with this allowance for 50 years of beach recession, the weir structure remains beyond the predicted erosion line.

8. "GREENHOUSE" IMPACTS

8.1 General

Ocean water levels and storm frequency may be affected by the increasing level of "greenhouse" gases in the atmosphere. Current estimation of the impact (Reference 7) are for an increase in atmospheric temperature of 1 degree Kelvin over the next 50 years. The effect of atmospheric warming on ocean water levels varies greatly and no clear indication is expected within the next two decades. However, a commonly accepted level is between 0.2 and 0.4m (workshop proceedings Macquarie University Greenhouse Conference, July 1989). Another possible result of atmospheric warming is the movement of the tropical cyclone zone further south.

8.2 Maintenance

Any change in ocean water levels and storm activity has implications for the outlet. An increase in sea level could be expected to raise the natural beach berm level. This would then lead to an increase in both maintenance and flood clearing requirements for the outlet. However, any increase in maintenance would be gradual, and could probably be accommodated by experience and increased efficiency in the clearing operation.

8.3 Ocean Water Inflow

An increase in general ocean water levels would also raise the water level height of major events. These levels would be further increased by a change in storm activity. As a result wave overtopping and ocean inflow could increase in frequency and volume. Under current conditions elevated ocean levels are not a significant problem. However this situation could deteriorate.

To counter the effect of increased ocean water levels it may become necessary to raise the height of the foreshore berm. This would solve the problem of ocean overtopping but would require increased efficiency in the event activated clearing operation to maintain the required flood outflow level.

8.4 Beach Erosion

Increased ocean water levels and storm activity would result in increased beach erosion. The current calculation of storm bite is based on a single major event and so should remain at a similar magnitude to the present. However beach recession will be affected. Estimation of the effect depends on a large number of factors such as the change in nearshore slope, the effect of offshore reefs, etc. On available data an increased recession of between 7m and 17m over the 50 year life of the structure could be expected.

Recession of this amount would increase the possibility of the weir structure being subjected to wave attack although not outflanking (Figure E7). However, as the proposed design makes allowance for wave forces, wave attack would not be a problem.

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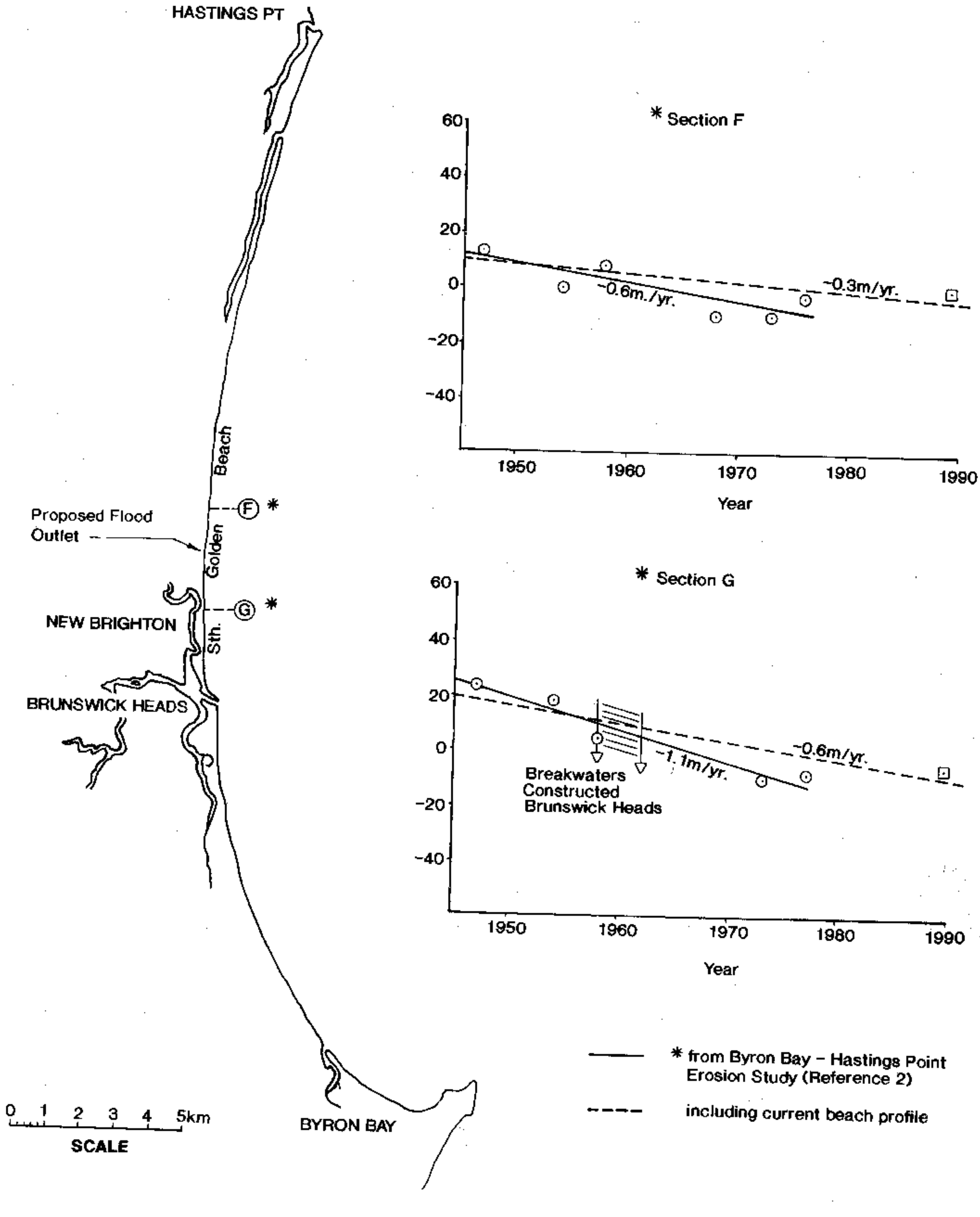
FIGURE E1

RECESSION RATES – SOUTH GOLDEN BEACH



RECESSION RATES – SOUTH GOLDEN BEACH

FIGURE E2

DESIGN ELEVATED OCEAN LEVEL

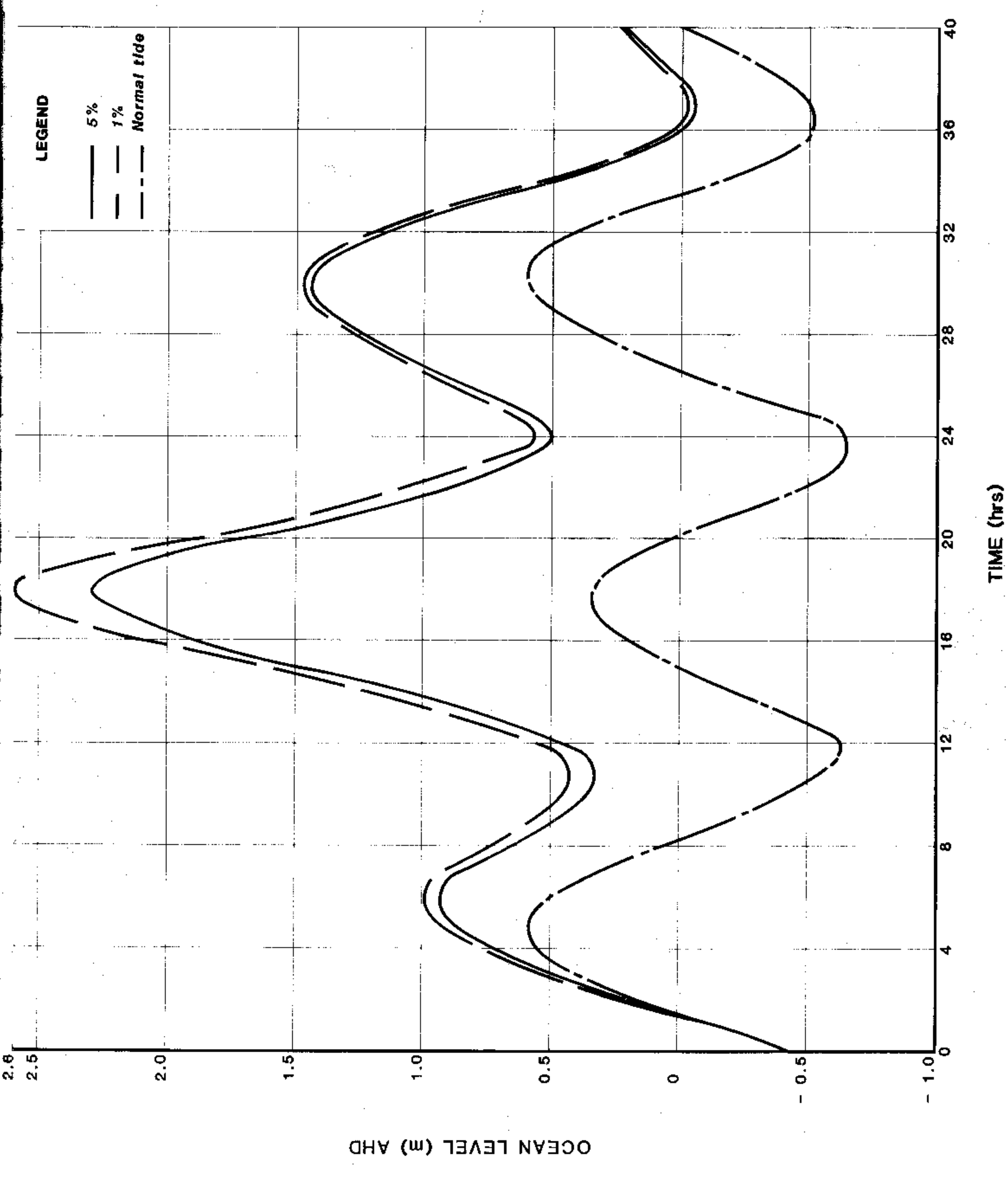
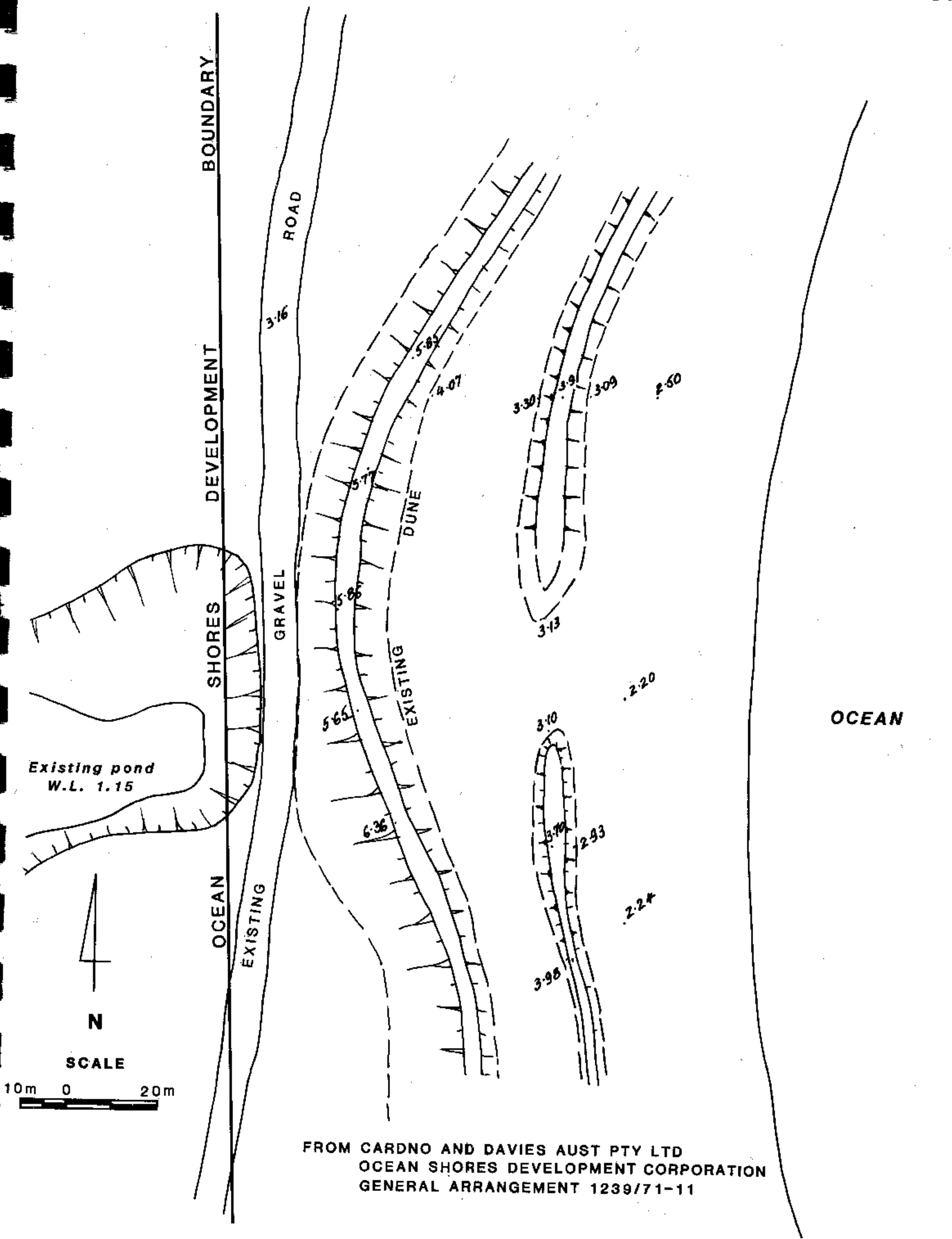


FIGURE E3

BEACH SURVEY NEAR PROPOSED OUTLET MAY 87



FROM CARDNO AND DAVIES AUST PTY LTD
OCEAN SHORES DEVELOPMENT CORPORATION
GENERAL ARRANGEMENT 1239/71-11

Fig E4

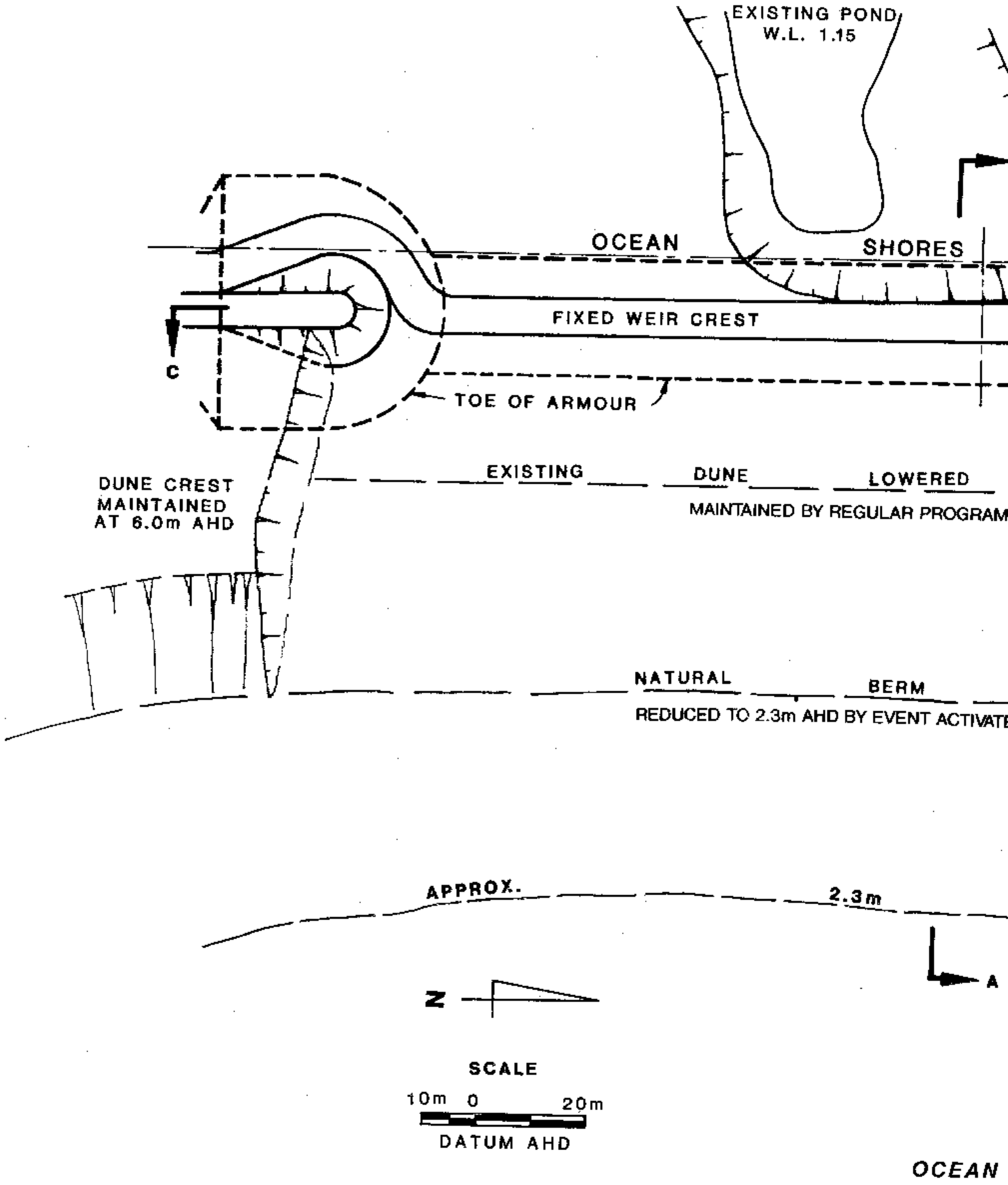

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Fig E4

FIGURE E4

OCEAN SHORES FLOOD OUTLET CONCEPT DESIGN PLAN

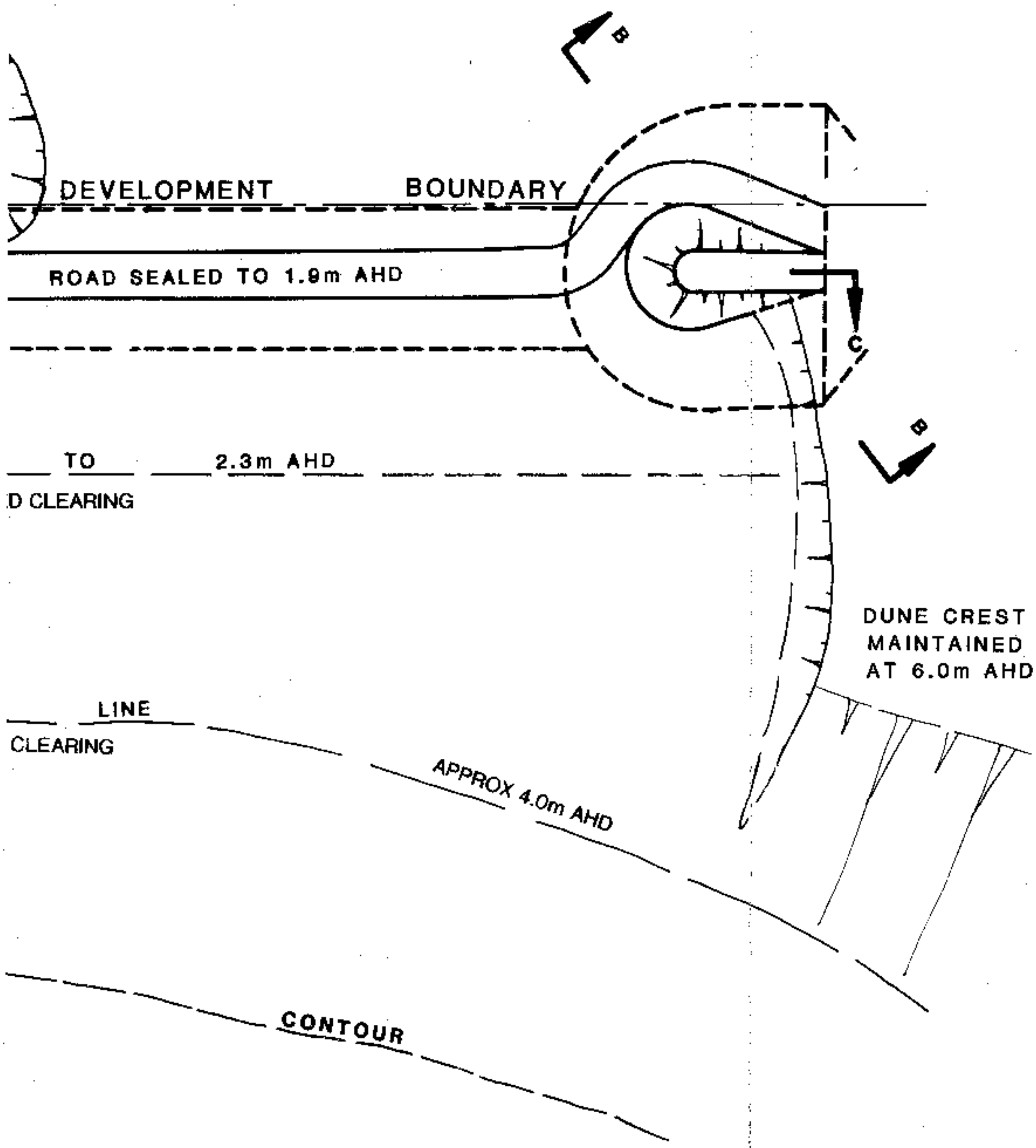
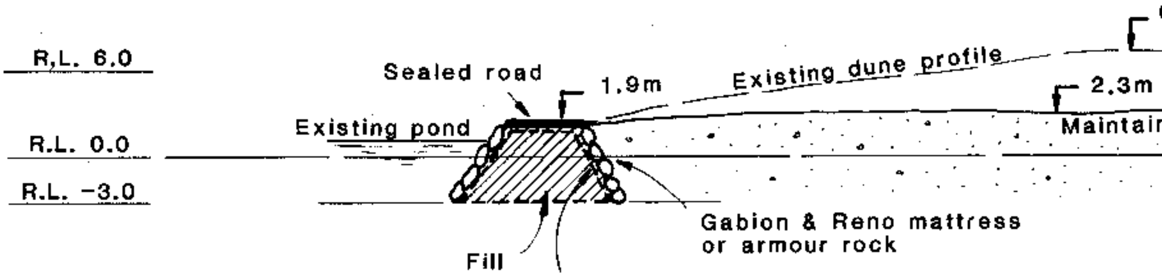
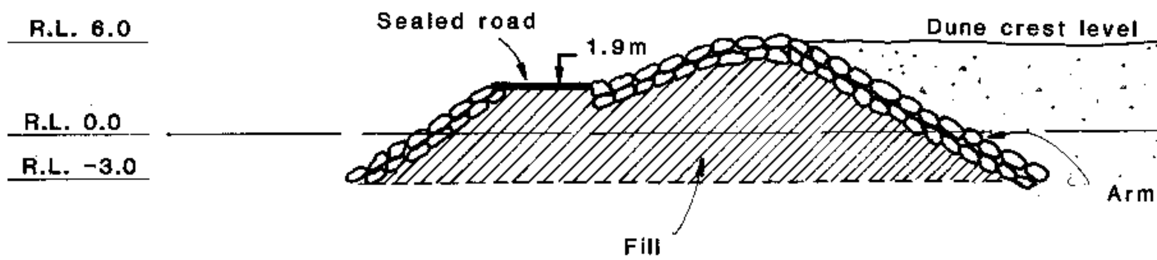


Fig E5



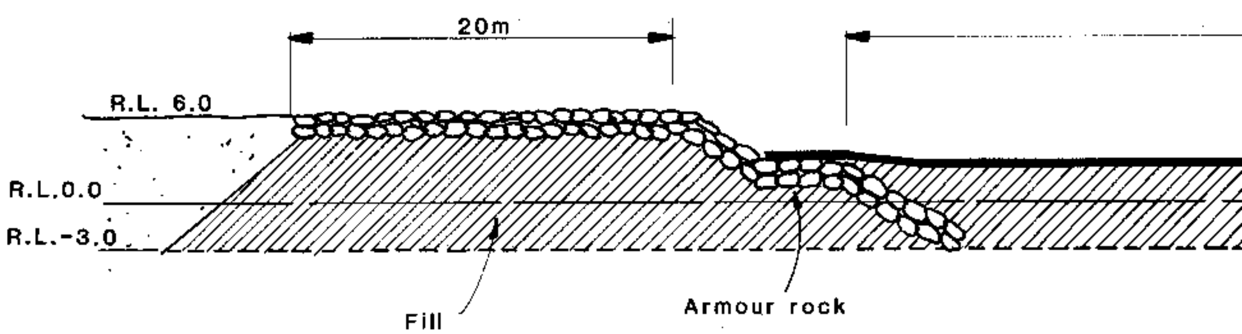
SCALE 1:5

SECTION AA



SCALE 1:5

SECTION BB

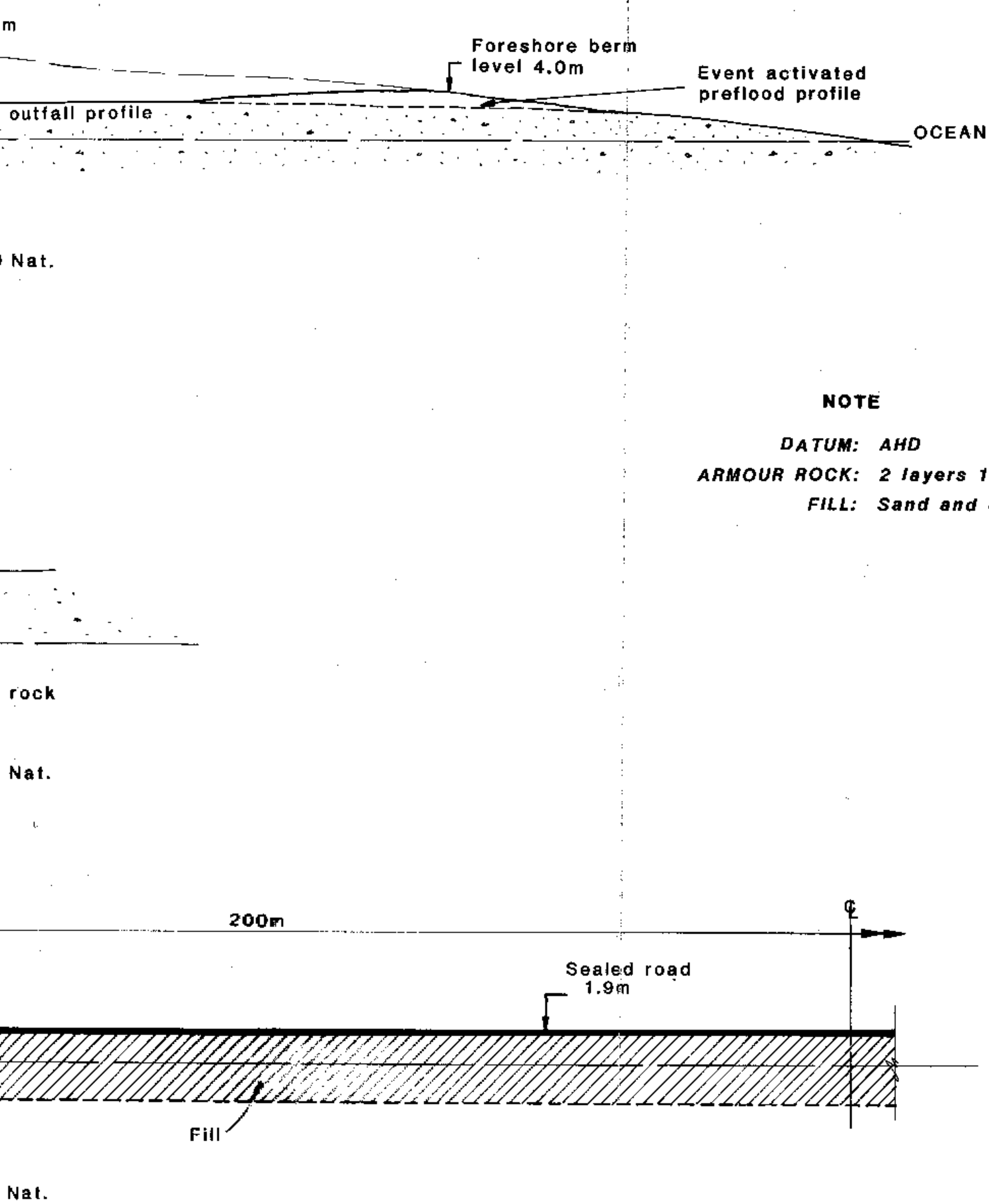


SCALE 1:5

SECTION CC

FIGURE E5

OCEAN SHORES FLOOD OUTLET
CONCEPT DESIGN
CROSS-SECTIONS



NOTE

DATUM: *AHD*
ARMOUR ROCK: *2 layers 1000kg*
FILL: *Sand and quarry run*

Fig E6

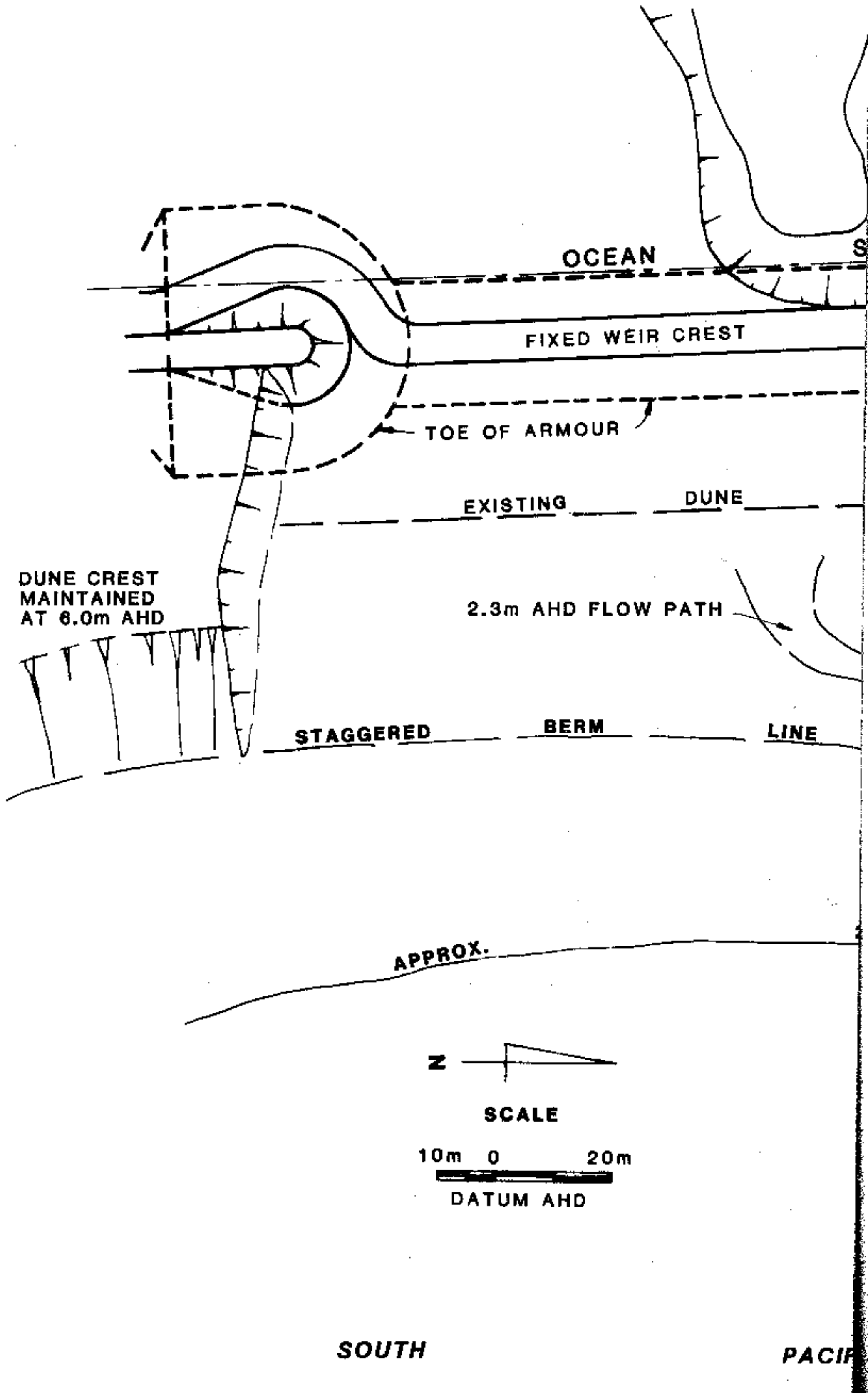

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Fig E6

FIGURE

OCEAN SHORE
FLOOD OUTL
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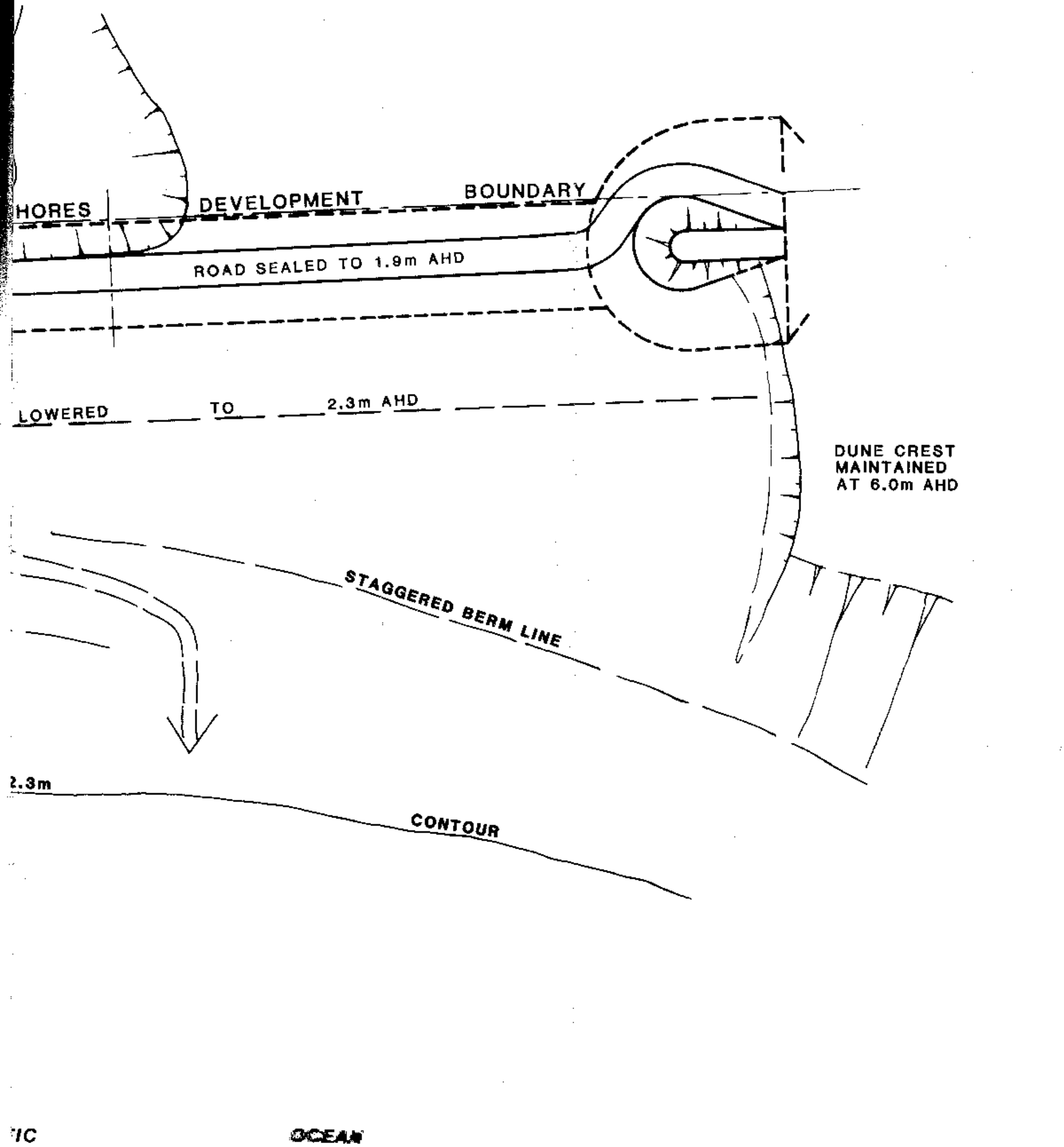

Fig E7

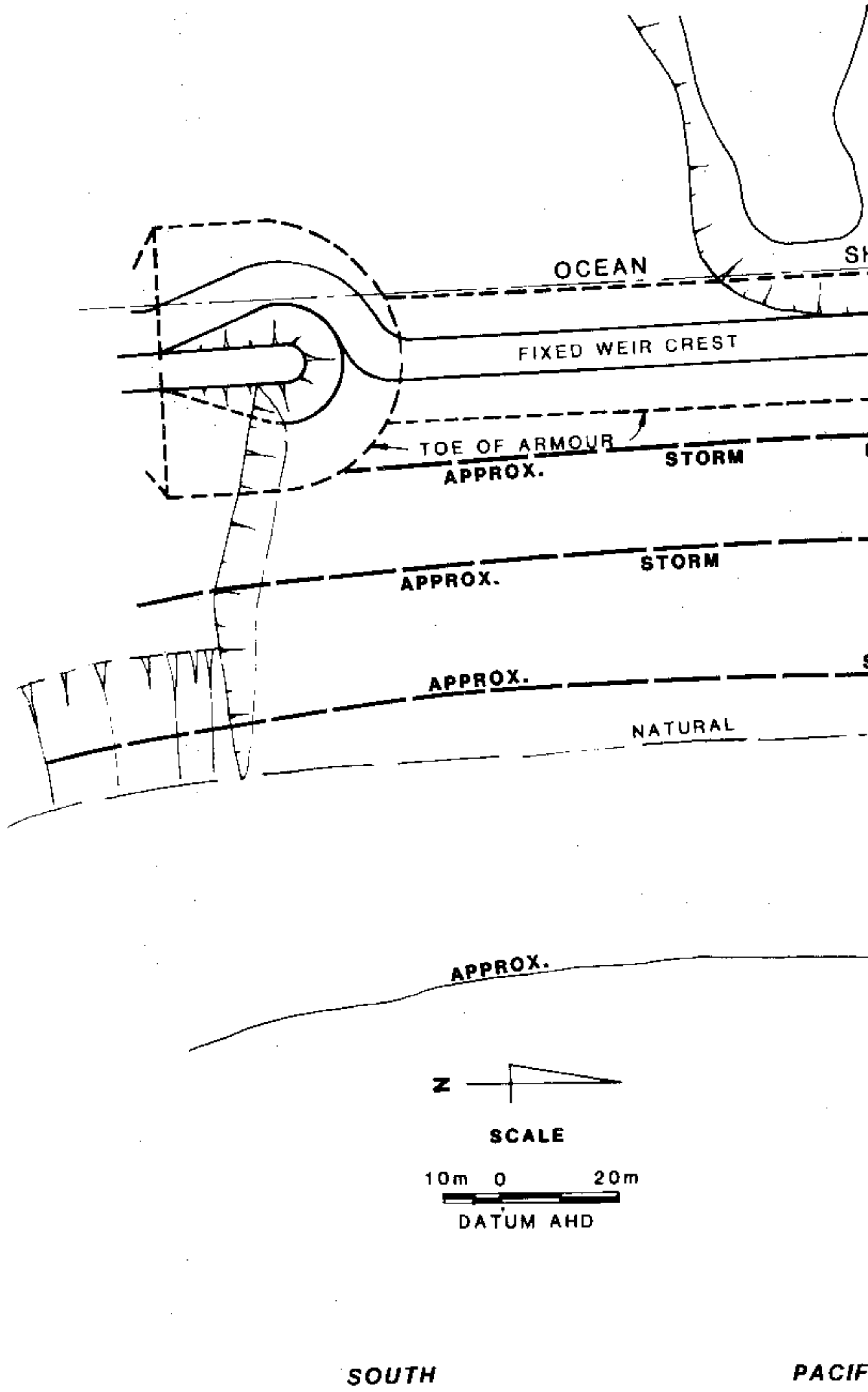

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Fig E7

FIGURE E

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